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MAPPING AND SURVEYING FOR LAND MAPPING, TITLING AND REGISTRATION PROJECTS

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I. General.

The Land Mapping, Titling and Registration (LMTR) project was designed to establish a more expeditious land titling and registration of land and evaluate the benefits of such a program to the tiller of the land. The land mapping aspect of this project is to evaluate the procedures presently employed and determine and recommend future procedures that will provide more expeditious production of cadastral base maps necessary for land titling and registration.

In accomplishing the afore mentioned goals, a survey of technical personnel and equipment inventory presently available within Directorate of Land Registration (DLR), Directorate General of Agraria (DGA), and also land use data and a limited terrain slope data evaluation was conducted. The land use data was obtained from the Directorate of Land Use (DLU), DGA. While the pertinent terrain data for primarily sawah areas (where distinctive boundaries are readily discernable from aerial photographs) has been approximated from a limited experience factor. Also training needs as well as availability of training facilities has been investigated and evaluated. Other parameters (private sector capability, etc) have also been considered.

From the evaluation of present procedures and the consideration of the other parameters, this report endeavors to provide recommendation and/or proposals that will expedite land mapping, for cadastral bases, establish cadastral survey standards including ties to the National triangulation net and present DLR organization structure.

II. Regulations Effecting Mapping.

There are a number implementing regulations under the Basic Agrarian Law (BAL) that apply to the LMTR program and a number of them specifically to the surveying and mapping to which this report will be addressed. These include ground survey, photogrammetry, monumentation and tie of local cadastral control to the national triangulation net. Specifically, key regulations include:

A. Minister of Agraria Regulation No. 6,7 September 1961, section I Article 1, which applies to establishment of basic control for cadastral base mapping and essentially states:

1. Before some areas are to be completed as land registration areas, they must have basic ground control established which will be used later as control net for position reference;

2. Basic ground control could be established by terrestrical and/or photogrammetric methods;

3. Considering efficiency, basic ground control could be established for one desa or for many desas;

4. Basic ground control established by the terrestrical method could be a net of three or more stations in a polygonal configuration.

5. Basic ground control could be tied to an established national triangulation net or to precise local coordinate system for temporary positioning.

B. Minister of Agraria Regulation No. 6,7 September 1961, section I, Article 31, which applies to the mensuration of parcels for land titling and registration and essentially states:

1. If measurement is to be done by terrestrical method, the measuring can be done desa by desa.

2. If measurement is to be done by photogrammetric method, the measuring can be done for several desas.

III. Discussion.

A. Present Mapping and Survey Procedures.

As specified by Agrarian Regulation No. 6, 7 September 1961, Articles 1 and 31, DPT employs both terrestrical and photogrammetric methods for establishing basic control for the cadastral surveys, and also uses the ground survey and the reproduced photogrammetric bases for parcel measurement. However, only measurements made from ground survey are presently accepted for Letter of Measurement and official certificate of title. Measurements from the photogrammetric bases are used for parcel area determination prior to posting of situation map in the Desa and the Kabupaten Agraria Office. Specific description of the terrestrical and photogrammetric procedure are provided as follows:

Terrestrical Method, Desa by Desa

The techniques, precision and accuracy described herein incorporate additional information and slight modification to some techniques described in the "Land Mapping, Titling and Registration (LMTR) in Indonesia - Systems Description and Recommendations Report", August 1982, by the LMTR team.

Basic Control Establishment.

The establishment of basic control on a Desa by Desa basis consists of several steps and levels of survey precision.

Initially a plan for control establishment is developed identifying general locations of stations along the primary, secondary and, as necessary, tertiary polygons.

The installation of a minimum of three primary control concreted monuments, constructed according to Letter of Decision by Head, DPT, No. 1, 8 January 1961, are placed in the preplanned locations around the general perimeter of the Desa at a maximum distance of approximately 2,000 meters apart along the primary polygon (a traverse that approximates the perimeter of the Desa). These primary control stations are tied together by a survey traverse along the primary polygon through intermediate stations, 100 to 200 meters apart, using an approximate United States (U.S.) F.G.C.C. third order, class I, survey procedures, with 1:10,000 precision in position closure, four (two direct and two reverse) horizontal angle observations and five second rejection limit from the mean horizontal angular reading. A one second theodolite is used for angular measurements and distance measurements between stations are accomplished using electronic distance measuring equipment taking a minimum of two observations (one forward, one reverse).

Secondary polygons are extended through the Desa with secondary concrete monumentation established at 400 to 800 meters apart and tied to each other and to the primary polygon control by survey traverse, using similar U.S. F.G.C.C. third order, class II, procedures with 1:5,000 in position closure, two (one direct and one reverse) horizontal angle observations and a five second rejection limit from the mean horizontal angular reading. A one second theodolite is used for angular measurements and an electronic distance measuring equipment is used for distance measurements taking two observations (one forward, one reverse).

As necessary, tertiary polygons within the Desa may be added to supplement the primary and secondary control in gap areas to densify the basic control to a 400 to 800 meter spacing. This tertiary control is also monumented in concrete, and tied to the primary and secondary polygon control through short survey traverses using one second theodolites with two angle observations, as in the secondary polygons, and a rejection limit of seven seconds from the mean horizontal angular reading. A 1:5,000 in position closure is also required. The electronic distance

measuring equipment is used for distance measurement taking two observations (one forward, one reverse).

Specifications of the primary, secondary and tertiary concrete monumentations as well a the quatenary monumentation (primarily for urban area) are enclosed as Attachment A (this enclosure is essentially a direct literal translation).

Parcel and Structure Measurements

Both parcel boundaries and building structures in the Desas are measured for the cadastral map bases. This is accomplished using the following procedures.

Direct observation and measurement using 20 second theodolites is the predominant procedure as in most instances the points to be measured are in direct sight of one of the nearest polygon stations. The measurements are made by first setting the theodolite on one polygon station and sighting on a second station (backsight station). A second surveyor or an assistant who may be a local resident sets a stadia rod at each point of boundary intersections or corners of structures and at each the theodolite operator sights on the stadia rod, recording the angular measurement between the backsight station and the point observed and the distance to that point from the observation station. When a boundary is irregular (i.e. it follows a contour and is curved) then several observations are made at significant points along that boundary . This provides a base for construction of that boundary for the Letter of Measurement and, if necessary, reconstruction of that boundary in the field.

When direct observations are not possible from a polygon station, the distance measurements normal (90 degrees) from a polygon line segment to the boundary or structure point is made using a survey tape. That point along the polygon line segment is then measured for distance from one of the terminal stations of that polygon line segment.

After the surveyor has completed his field measurement he returns to the office where he completes his computational work (determining distances along boundary or structure segments and areas of parcels and structures) and prepares his subsequent drawings (plats) and descriptions for the parcels or structure. These become the base for the Letter of Measurement.

Photogrammetric Method, Multiple Desas

This method utilizes aerial photography for preparation of map bases. The area usually covered is 10,000 or more hectares (as many as 50 Desas as in Pati, Central Java) and the aerial photography of the area is taken from about 760 meters above ground elevation at a scale of 1:5,000.

Basic Control Establishment

As with the terrestrical method, a plan is prepared identifying the distribution of the primary polygon control station (1500 to 2000 meters apart) around the perimeter of and across the middle of the area to be photographed. Secondary control stations are established and are spaced at 400 to 800 meters apart throughout the area. All of the primary and secondary control stations are concrete monumented according to specifications as required for the terrestrical and premarked (paneled/marked with an "X" or a cross) using material that can be easily identified on the aerial photography. The aerial photography is taken over preplanned flight lines normal to the primary polygon across the middle of the area.

The control stations of primary polygons are tied together using a partial equivalent U.S. F.G.C.C. third order, class I, survey procedures with 1:10,000 accuracy in position closure, four (two direct and two reverse) horizontal angle observations and five second rejection limit from the mean horizontal angular reading. A one second theodolite is used for angular measurements and an electronic distance measuring instrument is used for distance measurements with a minimum of two observations (one forward, one reverse).

The completion of the survey of the primary control stations may occur prior to or in coincidence with the flying of the photography.

Upon receipt of the aerial photography, DPT personnel select sharply distinct photographic image tie points common to successive photographs along a line of flight and also to photographs of the adjacent flight lines. They are marked by a fine pricked hole using precise point transfer equipment (PUG IV) on the film positive of each photograph. The tie points and the primary and secondary control stations on the photographs are measured on a precision photograpmmetric instrument (A-8) obtaining coordinates at photographs scale.

Through the common tie points all the photographs are tied together into one coordinate system at photographic scale and through the known local ground position (from the ground survey traverse) of the primary control station all

points on the photographs (tie points, primary and secondary control stations) are transformed simultaneously into the local ground coordinate system by a simultaneous mathematical aerial triangulation computer program.

The secondary control station coordinates, established with at least third order class II accuracy, are now part of the control base for the area.

Parcel Measurement

Using the tie points on the photographs the photography is rectified to a 1:1,000 scale onto a paper photographic base. This base is then used in the field for parcel identification, after which the parcel boundaries are drafted on a semi-stable drafting paper.

The parcels are then measured for area by either computing triangled segments of the parcel or by planimetric measurement of the parcel. These parcel drawings and measurements, at present, are used only for the Desa and Kabupaten posting and for issuance of a temporary certificate.

Boundary Monumentation

Boundaries of all parcels are monumented according to Minister of Agrarian Affairs Regulation No. 8, 7 September 1961 are enclosed as Attachment B (this enclosure is essentially a direct literal translation).

Boundary monuments are not generally placed in sawah area due to ground stability - Article 4 would cover this situation.

B. National Control Network.

BAKOSURTANAL

The Indonesian Government through Coordinating Body of National Survey and Mapping (BAKOSURTANAL) is the GOI Agency responsible for the overall coordination of geodetic, mapping and photographic operations in Indonesia. The Agency reviews and approves large scale to small scale aerial photographic and mapping projects for the military and various non-military agencies as well as plan and coordinate the establishment of the precise National first order monumented geodetic control net on the Indonesian Datum (ID) 1974 and the present National 1:50,000 scale mapping program. They maintain the national inventories or data banks of aerial photography, maps and

geodetic control but not those established by the individual agencies, i.e. Agraria. The National data are available to the various government agencies as needed.

Development of First Order Control Net

Development of this net is fundamental to providing a base for accurately positioning geographic features and relating these features in a graphical form on maps. The present control is primarily developed to meet the 1:50,000 scale National mapping program with plans to develop a 1:25,000 scale map series in the future; however, this control serves to relate in an absolute positioning all secondary control and mapping, accomplished by the various organizations for special purpose, by tying such secondary control and maps to it. Presently, the control stations are spaced at 30 to 60 kilometer intervals to control the 1:50,000 scale aerial photography taken over Java and Bali and the 1:100,000 scale aerial photography taken over the other islands, but it is planned to tighten this net to 15 to 17 kilometers to provide the necessary control for the 1:25,000 scale mapping. There is still a few areas and smaller islands where first order control must be established.

BAKOSURTANAL maintains all the records of the control which includes printouts of station identification latitude, longitude, UTM zone northing and easting (ID-74) and elevation. Records also include description, photographic identification and graphical representation (with tie to reference points) for the control. This control is available to the various organizations establishing local control for the special purposes to provide absolute positioning to that local control.

C. DPT Personnel and Equipment

It is the Sub-Directorates of Photogrammetry and Triangulation, and Terrestrical Surveying and Mapping that are responsible for survey and mapping for cadastral bases; however, once it has been determined in joint meetings of the Sub-Directorate Heads and the Director, DPT, what areas will be surveyed and the method by which it will be surveyed and mapped, the two Directorates proceed with their assignments independent of each other. Each plans its own control distribution for the surveying and mapping as well as the other individual operations in preparing a map base.

Technical Personnel

As of September 1981, the DPT technical staff consisted of the following:

Level IV	5
Level III	116
Level II	2.508
Level I	2.239
Total	4,868

Through the year 1982 it is estimated that there has been an increase of about ten percent in the technical staff enlarging it to over 5,300. Levels III and IV are mostly engineers, with a few Level III and all Level IV classified as senior staff.

Of these technical personnel, it is estimated that 3,500 to 4,000 are actual field surveyors (Levels II and I) assigned to surveying and measuring the parcels and structures for cadastral maps. At present an increase of approximately 250 surveyors a year is made to the staff. It was indicated that a field surveyor would require two days in the office (computations and plat preparations) for one day in the field.

Although a number of the engineers are qualified or trained in photogrammetry they are not involved in direct production operation. Present operational/personnel total ten at the photogrammetry laboratory and that staff will be increased at possibly two a year for at least the next one to two years.

Technical Equipment

As of December 1982, the DPT had on inventory the following equipment.

Survey Equipment

EDM's (three kilometer)	45
EDM's (one kilometer)	14
Theodolites (one-second)	366
Theodolites (ten-second)	251
Theodolites (20-second)	682
Stadia Rods (wooden)	2,528
Stadia Rods (metal)	474
Measuring Tapes (Chains)	1,455
Grid Plates	209
Metal Rules (one meter)	340
Planimeters	350
Compasses	1,373
Scales (desk size)	2,133

Pantographs	78
Prisms (for ground measurement)	299
Prisms (solar)	266
Mini-calculators	551
Stereoscopes (mirror)	110

Photogrammetric Equipment

Wild A-8	1
Wild B-8	1
Zeiss Planimat	1
Zeiss C-8 (need of repair)	1
Wild E-4	1
Zeiss SEG V	1
Zeiss GZ-1	1
Wild EK-22	1
Wild PUG-4	1
Klimsch Copy Camera	1
Miscellaneous Lab. Equipment	-

The Wild A8 and B-8 and Zeiss Planimat and C-8 are precise stereoplotting and stereotriangulation instruments. The Wild E-4 and Zeiss SEG V are precise aerial photographic rectification equipment. The Zeiss GZ-1 is the only orthophotographic reproduction equipment. The Wild EK-22 is an on-line coordinatograph to the Wild A-8. The wild PUG-IV is the only stereo point transfer equipment. The Klimsch Copy Camera is a varying scale film reproduction equipment. The miscellaneous laboratory equipment includes contact printer, processing tanks, print dryer, etc.

Primarily to meet urban mapping requirements, a wild analytical plotter and OR-1 orthophotographic equipment is to be delivered, tentatively by July 1983.

Productivity

Productivity of land surveying and mapping varies with the areas or situations. Based upon visitations to Agraria field offices and discussions with Central Agraria personnel it is estimated that a surveyor can cover six hectares per day for clear areas (sawah, dry land, plantation, alang-alang, fish ponds, lakes, marsh, etc.), one hectare per day for kampung and urban area and two hectares a day for transmigration areas. Considering that a surveyor works two days in the office for one day in the field and works six days a week, a surveyor can survey 600 hectare a year for the clear areas, 100 hectares a year for kampung/urban areas and 200 hectares a year for transmigration areas.

For the photogrammetric process of a 10,000 ha area (after DPT receives the photography from the aerial photographic contractor) selection and marking by point pricking of tie points on each photograph requires approximately 30 calender days, mensuration of the photograph requires approximately 20 calender days, rectification requires approximately 60 calender days and the drafting of the base and areal calculation approximately 90 calender days.

D. Contract Surveying and Mapping

Presently there are more than 60 private terrestrical engineering and survey companies varying in technical personnel from as few as a couple of dozen to 400 or more. These companies could alleviate the work load of DPT for polygon and parcel measurement surveys. Existing regulations, as presently known to the LMTR Team, do not indicate whether use of contract surveys can or cannot be used for cadastral purposes in either polygon or parcel mensuration. Presently contract survey companies have been or are to be used in polygon surveys for the Solok, West Sumatra, LMTR orthophoto test area and for the urban mapping project that includes 100 Indonesian cities. Two significant factors effect the use of the private sector of surveying and mapping - first, the available contractors to bid on a project at a given funding level and second the competency or ability to complete the project. For example, contractors eligible to bid on the orthophotographic reproduction of photography on the Solok, West Sumatera, project do not have the equipment or facility to produce orthophotraphic bases.

E. Training

DPT has a training program for surveyors, within its own organization. In addition to this DPT can draw from ITB, GMU, etc for surveyors; however the major number of surveyors added to the staff each year come from the ranks of the DPT trainees. Photogrammetric operational personnel generally come from the specialized training from ITB and GMU. Also the DPT engineers are graduates from both ITB and GMU. Several of the DPT staff, including the senior level group, have advanced or graduate level training at ITC in the Netherlands, and also in Switzerland and Germany.

The DPT has a highly technically trained staff at the mid to senior level, but this staff is small. There has been a significant increase in this staff in the last three to five years, but consideration for

more advanced and specialized training for these personnel and expansion on training programs in general is tantamount.

F. Standards for Surveys and Mapping in the United States (U.S)

In the U.S. a set of standards has been established for meeting the varying levels of accuracy requirements. These standards have been developed by the Federal Geodetic Control Committee (F.G.C.C.) under the direction of the Federal Geodetic Control Coordinator, National Oceanic Atmospheric Administration, Department of Commerce. In general, standards for traverse have been established for first order, second order class I, second order class II, third order class I and third order class II.

First order control stations are the primary control for a National network. These stations established through traverse precision procedures meet one part in 100,000 accuracy. In addition, second order class I stations established through traverse precision to a one part in 50,000 accuracy provides secondary control and thereby densifies a precise National control network.

Second order class II traverse station control established to a one part in 20,000 accuracy supplements the National control network and provides the base from which surveys requiring greater accuracies is conducted. This class of precision is used in such cases as urban areas and special development projects where the land values are high.

Third order, both class I (established to one part in 10,000) and class II (established to one part in 5,000), traverse control can provides the necessary base for surveys in lower valued land area, i.e. rural.

Since the standards for precision and accuracies for all levels (fist, second and third order) and procedures (triangulation, trilateration, traverse, azimuth and leveling) is described in detail in the F.G.C.C. publications (Attachments C and D) included with this report, further breakdown here is not necessary. Additional copies of the publications will be provided by the Department of Commerce through the BLM Washington Office.

IV. Conclusions

A. Accuracy of Ground Control.

There is not in general, a significant difference between the precision or accuracy employed in surveys and parcel measurement between the procedures employed by DPT and those of the United States Federal Geodetic Control Committee (F.G.C.C.) that specifies precision and accuracy for various types of land utilization in the U.S. Possibly precision may be less, but not so critical that the efficiency of land titling and land registration may be more effectively accomplished. Ultimately a precise national cadastral base must be considered but even under a present fragmented area by area basis it is being completed on a local but precise control network but on a local basis.

Procedures of precision in angular measurements for both terrestrical and photogrammetric basic monumental control is less than that precision specified by the F.G.C.C. For the primary polygon control on Desa by Desa, two sets of direct and reverse observation are made with a one second theodolite and the maximum angular variation tolerance compares favorably with the F.G.C.C. standard; but in the case of the secondary and tertiary polygons (Desa by Desa) DPT takes only one set of direct and reverse observations (using one second theodolite) compared to at least two sets specified by the F.G.C.C.

A second situation where precision may be less than desireable is in establishing the primary control for the photogrammetric area base which compares to the F.G.C.C. third order class I standards. Since there is an extensive area of approximately 10 kilometer by 10 kilometer, second order class II accuracy of one part in 20,000 should seriously be considered. This is already being done for the photogrammetric bases for the urban mapping program.

B. Tie to National Control Network

This is essentially being accomplished in all of DPT surveys, as in the Pati, Central Java, pilot site where several valid National triangulation station have been incorporated in or tied to the primary polygon for the 11,000 to 12,000 hectare area. Generally the polygons have been computed in a local system due to lack of National control - especially outside of Java - but, where at least two valid established National control stations are accessible, the polygons can be

mathematically transformed into the National coordinate system (ultimately the ID-74 UTM system).

As the expansion of both the National network and DPT cadastral base surveys continues, more opportunity to tie the DPT bases to National control will exist. As this takes place, all cadastral bases (past ones already computed on local coordinate systems, present and future) should be mathematically transformed into the National UTM coordinates system.

The mathematics are fundamental for transformations of the local systems into the National UTM system and the transformation parameters and coefficients for the mathematical formulation are available through BAKOSURTANAL. Transformation can also be made to latitude and longitude coordinates through BAKOSURTANAL formulations.

C. National Grid Net

On a long range bases the adoption of a grid net on a National bases could be considered for relating at least down to Desa level DPT data to a National identity. This has been done by land use offices within the Provinces, but it is an arbitrary grid that cannot be directly related to neighboring Provinces. The establishment of this grid would not replace the logical reference system of DPT records for DPT administration (presently records are maintained on a Desa basis with systematic identification of land parcel by type of rights, etc, and sequential numbering). Rather it would provide an identification to a base to which DPT data could be related to that of other data not necessarily maintained on a Desa basis.

As Indonesia covers seven UTM zones (a distance east to west greater than conterminous U.S.) a single square or rectangular grid base is not possible. A similar situation confronted the establishment of an offshore outer continental shelf grid by the U.S. which ultimately adapted the UTM grid system including zones. By adopting the UTM grid system used by many countries, as it is worldwide, Indonesia could be segmented into seven six-degree bands, each band extending in the north-south direction and presenting a flat representation of Indonesia.

The central meridian of each UTM zone is given an arbitrary false Easting coordinate of 500,000 meters. The equator is given a Northing coordinate of zero meters for measurement of all points north of the equator and an arbitrary false Northing of 10,000,000

meters for measurement of all points south - in both cases coordinates get larger in north direction.

Due to the narrowing of the longitude distances from the equator to the poles, the north-south grids of adjacent zones will not be parallel and therefore cannot directly correspond at the point of abutment. Since coordinates increase to the east, it would be preferable starting with the westernmost zone and extend the grid to the westernmost north-south grid of the next zone - this, however, will cause partial grids on the east edge of the first zone (this procedure was used for the U.S. outer continental grids) but it effectively relates any specific area that of any other area in Indonesia. This procedure would be followed across the seven zones. For the north-south, the change in value at the equator can be alleviated by setting a zero Northing at a point south of the most southerly point of Indonesia and adjust the grid values both south and north of the equator such that they will be sequentially increasing and such that one of the east-west grids is coincident with the equator for ease of adjustment.

A minimum grid interval of 10,000 meters would be appropriate. This interval is approximately six miles which is the length of the sides of a township in the U.S.

This grid on a large scale base for data reference, however, will still need considerable study and planning on an interagency as well as DPT, Agraria, basis. Coordination with BAKOSURTANAL should be effected.

D. DPT Organizational Structure

General

There is an obvious requirement to build up the DPT staff especially field surveyors for establishing basic ground control and complete parcel measurements. The area to be covered is far too extensive for the present staffs to accomplish the work in the near future. To establish a base to determine the staff needs, land use data from both Directorates of Land Use, DGA, and the Central Bureau of Statistics, was used to estimate the following areas in hectares. The figures are in millions:

Sawah, dry land farming and plantation	43.0
Alang-Alang	11.5
Fish ponds, salt beds, waste land, swamp and lakes	7.5
Kampung, urban	5.0
Forest	125.0
Total area	192.0

Any variation of the above data with that of the two sources should not significantly effect the projection of an increase in surveyor staff to expedite the cadastral mapping.

Assuming that the mapping of forest areas on a large scale cadastral base would be waived in favor of photogrammetric mapping with small scale aerial photography (except for those areas being developed for transmigration), approximately 62,000,000 hectares of sawah, dry land farming, plantation, alang-alang, fish ponds, salt beds, waste land, swamp and lake areas and approximately 5,000,000 hectares of Kampung and Urban area will need to be surveyed and mapped for cadastal and land registration (192,00,000 ha total area minus 125,000,000 ha forest area). At least 2,000,000 ha of these lands have been surveyed already, therefore the remaining could be approximately 60,000,000 ha open land and 5,000, 000 ha kampung and urban land. These are the figures used as a basis to establish a DPT staff increase.

Development of the Survey Staff.

Four types of situations exist for completion of surveys. They include transmigration and NES areas, both developed in existing forest areas, kampung/urban areas and other areas (sawah, dryland farming, plantation, alang-alang, etc.) that are outside of the forest areas. The number of hectares that can be surveyed per year by one surveyor varies with the type of area. Approximations for number of hectares that can be accomplished for one year by one surveyor, has been based upon visitations to the field offices and discussion with Central DPT staff. For example, a surveyor can cover approximately six hectares a day (sawah, etc) but will be in the field for only one day out of three (the other two days he is in the office working on his computations and plat preparation). Working six days a week, he will be able to cover 12 hectares a week and, with 50 weeks a year, he can survey an approximate 600 ha per year for sawah, dry land farming, etc. Based upon the approximation of 600

ha/yr/surveyor (sawah, dry land farming, plantation, alang-alang, fish ponds, salt beds, waste land, swamp and lakes, pencetakan sawah and NES) for Group I, approximation of 100 ha/yr/surveyor (kampung/urban) for Group II and approximation of 200 ha/yr (transmigration) for Group III, Chart A (Page 17) shows an initial assignment of a total 2,500 field surveyors for 1983 between Groups I and II (1,000 field surveyors in Group III) and periodic tentative increase of surveyors each year thereafter in Groups I and II has in developing a base to determine an optimum buildup in the total field survey staff. At present group III is considered to maintain a uniform 1,000 surveyors per year to meet a 200,000 hectare a year transmigration development.

Considering no increase and the distribution used on Chart A for the initial estimated field survey staff, it would take 100 years (based upon 600, 100 and 200 ha/yr/surveyor of Groups I, II and III) to complete the surveys of all the nonforest (essentially 67.000,000 plus hectares). If productivity is only 50 percent of the above estimate then it would take 200 years to complete the nonforest areas. By building the staff this can be reduced substantially, but such build up must be incremental as large scale immediate buildup is not feasible for quite obvious reasons - budget being primary, facilities for working space, availability of personnel, purchase of equipment and training. On Chart A an arbitrary but possible logical yearly increase of surveyors has been developed. This includes an increase of 250 surveyors a year (this is the present buildup) for the first four years to 1987 and 500 per year until 2000 (it is anticipated that the increase to buildup of 500 a year could be programmed by 1988). After the year 2000 any increase would have no significant effect on shortening the time period to complete the 67,000,000 hectares identified above. Based upon the Chart A with increases of 500 surveyors a year up through year 2000, it would take until the year 2005 to complete (a total of approximately 22,5 years) to complete the surveys of the nonforest areas and for any increase of surveyors after year 2000 the completion would still take to the year 2005. With only 50 percent of the estimated productivity (600, 100 and 200 ha/year/surveyor) it would take till at least year 2028 to complete the nonforest areas (a minimum of 46 years from 1983).

At this point consideration as to the optimum total buildup should be made. To facilitate such determination, Chart B (Page 18) was developed to

evaluate the number of surveyors versus number of years to complete the surveys. Chart B is ranged from years 1992 through 1998 and a total survey staff from 7000 to 10,000. With a buildup to 7,000 surveyors in 1992 it would take another projected 19 years to complete the surveys - a total of 29 years from 1983. In contrast with a continued buildup to 10,000 surveyors in 1998 a total of 23.5 years from 1983 would be required.

Chart A - Expansion of Staff by Year.

	GROUP I			GROUP II		
Year	Surveyors @ 600 ha/yr	Total Ha/yr. (Millions)	Accumulative Ha (Millions)	Surveyors 100 ha/yr	Total Ha/yr. Millions)	Accumulative Ha. Millions)
83	1000	0.6	0.6	1500	0.15	0.15
84	1250	0.75	1.35	1500	0.15	0.3
85	1500	0.9	2.25	1500	0.15	0.45
86	1750	1.05	3.3	1500	0.15	0.6
87	2000	1.2	4.5	1500	0.15	0.75
88	2500	1.5	6.0	1500	0.15	0.9
89	3000	1.8	7.8	1500	0.15	1.05
90	3500	2.1	9.9	1500	0.15	1.2
91	3500	2.1	12.0	2000	0.2	1.4
92	4000	2.4	14.4	2000	0.2	1.6
93	4500	2.7	17.1	2000	0.2	1.8
94	5000	3.0	20.1	2000	0.2	2.0
95	5500	3.3	23.4	2000	0.2	2.2
96	5500	3.3	26.7	2500	0.25	2.45
97	6000	3.6	30.3	2500	0.25	2.7
98	6500	3.9	3.42	2500	0.25	2.95
99	7000	4.2	38.4	2500	0.25	3.2
00	7000	4.2	42.6	3000	0.3	3.5
01	7500	4.5	47.1	3000	0.3	3.8
02	7500	4.5	51.6	3500	0.35	4.15

GROUP III - Uniform 1000 Surveyors each year at a rate of 200 ha/yr/surveyor

Chart B - Projected Survey Completion Based on Chart A

Year	Total Staff Groups I,II, III	1983 - on Lapsed Years	Projected Years	Total Years from 1983
1992	7,000	10	19	29
1993	7,500	11	16.5	27.5
1994	8,000	12	14	26
1995	8,500	13	12	25
1996	9,000	14	10.5	24.5
1997	9,500	15	9	24
1998	10,000	16	7.5	23.5

Based upon the productivity estimates of Chart A, consideration of an expansion of the field survey staff to the 10,000 level through 1998 would be desirable as it would allow for an estimated completion of surveys of existing nonforest areas by the year 2006. Such a buildup to 10,000 could lead to a possible overstaffing of field surveyors as the required cadastral survey nears completion. However, possible reassignment of surveyors from DPT to other staffs (Land Use, Land Reform, Agriculture, Public Works, etc) and attrition (advancement, moving laterally to other type assignments, retirement, etc by the surveyors) would alleviate this problem and the survey staff could be reduced to an appropriate level to meet the required surveys for additional development of new projects and survey maintenance of those areas already completed.

Development of the photogrammetry Staff

The photogrammetry staff is the less critical of the mapping staffs because the aerial photography coverage is limited each year. The areas that photogrammetric mapping can be used for boundary determination is limited to those areas where boundaries are significantly definitive on the aerial photography, and those areas that are economically feasible. Such areas are the large sawah areas which are relatively flat (an exception is the urban areas where a program for photogrammetric mapping of 100 cities is presently in progress), and at least 90 percent can be complete by rectification.

Sawah represents approximate seven percent of Indonesia or an area of approximate 13,500,000 ha. of which 700,000 to 800,000 ha. have been completed by photogrammetric mapping. However, on an average the photographic coverage is limited to approximately 60,000 ha per year for the rural areas. Possibly partial effort and budget presently expended on the urban mapping project could be shifted to the rural

areas upon completion of that project (estimated date 1985 or later). This could possibly double to triple the present average area coverage without need of increasing the present staff as they would shift assignments from urban to rural. However, the possible increase in acquisition of aerial photography over present urban and rural coverage will require an increase in the photogrammetry staff. It is conceivable that at least doubling the present staff may be needed to meet the increased photogrammetric budget, space and equipment and equally the increased ability of the aerial contractors to obtain additional overall coverage for all users.

Reorganization of Basic Survey Control Staff

As noted earlier, the planning and establishment of a basic ground control is split between the Terrestrical Survey Subdirectorate and the Photogrammetry and Triangulation Subdirectorate of DPT. Although it is true that both perform the parcel mesuration by different procedures, the establishment of the basic control requires essentially the same planning and ultimately will all have to be tied together to the National control net. Presently the staff of the two Subdirectorates are and would have to be responsible for these adjustments of their individual control nets.

Maximum utilization of those staff members of both Subdirectorates, maintenance of total effective coordination of control planning and assuring adjustments of local control nets to each other, as well as an ultimate adjustment to the National control net, is important. To accomplish this, consolidation of those functions combining staff members responsible for those functions from each of the two Subdirectorates into a single organization or office should be effected.

Two approaches could be considered for the formation of an office for basic control planning, coordination and computational work. First, is an office responsible to the Director, DPT, with coordination to each of the Subdirectorates. Second and probably most feasible is establishment of another Subdirectorate at the same level as the other Subdirectorates of DPT.

Following the second approach to a single office or organization for basic control surveys establishment, the "Subdirectorate of Basic Control Surveys" would be established and segmented into a minimum of two

sections. The first section would be the "Planning and Coordination" with responsibilities to maintain regular coordination with the other Subdirectorates of DPT on project areas and with BAKOSURTANAL on status of National control availability for preparation of control plans. The second section would be "Mathematical and Adjustments" with responsibilities for final computational work on project control nets and ultimate adjustments and transformation of local control nets to each other and to the National net.

Equipment

With expansion of the technical staffs, increased equipment purchases will be necessary. This will be far more significant with respect to survey equipment than photogrammetric.

Survey Equipment

Most significant requirement for survey equipment will be theodolites. Presently one theodolite is required for every three field surveyors (each surveyor is in the field one day and in the office two days). This, however, does not provide the staff with a margin to allow for routine maintenance and special project use of the theodolites - a reasonable approximate increase of 10 percent more theodolites should cover this situation. With a 10 percent increase, the ratio would be approximately one theodolite for every 2.7 field surveyors. Additionally some of the older theodolites could be used for training thus reducing again the availability of the one for three ratio. Therefore a ratio of one theodolite for every 2.5 field surveyors would be reasonable. With an increase to 10,000 field surveyors an inventory of 4,000 theodolites will be needed.

Other survey equipment in approximately the same proportion with theodolites as is presently in the inventory would also be necessary. The increase in theodolites with a buildup to 10,000 field surveyors is about three times, therefore at least three times as many EDM's will be necessary as will the rest of the various equipment.

Photogrammetric Equipment

The buildup of the photogrammetric staff could have less effect on equipment needs proportionately as the survey needs; however if the available aerial photographic coverage and the staff were to double at least one more rectifier, one more point transfer

equipment would be needed and the photographic laboratory would have to be expanded.

Facilities

Office and laboratory space will needed to proportionately expand to meet the increase in staff and additional equipment. Programming of funds to provide facilities for the expansion of personnel and equipment is as important as the expansion of the latter.

V. Recommendations/Proposals

A. Accuracy and Precision Standards of Basic Ground Control

Multiple Desa's (Photogrammetric Base)

Primary Polygon Control: Preferably second order class II accuracy
Distance between control stations: 1500-2000 meters
Distance accuracy: one part in 20,000 (presently one in 10,000)
Distance measuring precision: minimum of two observations
Angular accuracy: 5" maximum angular deviation
Angular precision: preferably four direct and four reverse readings (presently two direct and two reverse)
Secondary control:
Distance between secondary control: 400-800 meters
Position accuracy (aerial triangulation): one part in 5,000
Equipment: EDM (2,000-3,000meter), one second theodolite for the field survey work.

Desa by Desa (Terrestrical Base)

Primary Polygon Control: third order class I accuracy
Distance between control stations: 1500-2000 meters
Distance accuracy: one part in 10,000
Distance measuring precision: minimum of two observations
Angular accuracy: 5" maximum angular deviation
Angular precision: minimum of two direct and two reverse readings
Equipment: EDM (2,000-3,000 meter), one second theodolite
Second and Tertiary Polygon Control: third order class II accuracy
Distance between control stations: 400-800 meters
Distance accuracy: one part in 5,000
Distance measuring precision: minimum of two observation

Angular accuracy: 5" maximum angular deviation
Angular precision: two direct and two reverse readings (presently one direct and one reverse)
Equipment: EDM (1,000 meter), one second theodolite

B. Tie to National Control Network

Tie of all local basic cadastral control should be made to the National control net as indicated in "Discussions" and "Conclusions". To achieve this the following should be accomplished.

1. A liaison staff of possibly two or more be established to maintain regular liaison with BAKOSURATANAL regarding status of the National control net.

2. Liaison staff obtain as necessary the mathemathical transformation programs to convert local control systems to the National system.

3. Liaison staff coordinate DPT projects with BAKOSURTANAL for obtaining National coordinates as available in establishing the project control on a National system.

C. National Grid Net.

Adaptation of National grid net should be considered on a long range basis, but planning of the net for large scale data reference purposes still requires considerable study from an interagency standpoint. The BAKOSURTANAL should be the key coordination point as they have the technical capability and programs for development of grid systems. This should be one of the responsibilities of a DPT liaison staff, as identified with respect to local basic cadastral control to the National net.

As an initial consideration due to the size, east-west for Indonesia, the adaptation of the UTM (ID-74) system on a seven-band (south-north direction) from west to east in Indonesia should be evaluated. Similar approach was made by the Bureau of Land Management, Department of Interior, for the offshore outer continental shelf grid. Examples of the approach as described under "Conclusions" are not presently available for this report but can be provided at a later date.

D. Parcel and Structure Measurement for Cadastral Bases.

Procedures followed presently for parcel and structure measurement by terrestrical method is quite satisfactory. It is the fastest as far as ground survey is concerned and is accurate. The procedure should be continued for parcel measurements, but the need for accurate position of a structure on a parcel is highly questionable for cadastral survey purposes. For significant time reduction in cadastral surveys of kampungs, the measurement of structures should be eliminated as a standard practice and left to the owner to have done if he should feel it necessary.

The photogrammetric bases prepared from the accurately controled photography should be reproduced on stable base film from which parcel areal measurements on flat terrain should be able to be made within maximum three percent error accuracy for parcels greater than 0.1 ha. The photogrammetric bases would be used to reconstruct destroyed boundaries by direct measurements on those bases. These photogrammetric base measurements should be acceptable as valid measurements for Letter of Measurement.

If the photogrammetric bases are accepted as valid bases for Letter of Measurement then the slope in adjacent parcels should be less than 5 percent to use rectification process. If greater, then orthophotographic process should be use. However, until the Solok, West Sumatera, pilot site orthophotographic test is completed, the optimum use of the orthophotographic base and terrestrical survey base cannot fully be established. Therefore that test must be completed and evaluated for time/cost impact.

E. DPT Organizational Structure and Equipment

On the assumption that a buildup of 250 field surveyors could be accomplished for the years 1983 to 1987 and an expansion to 500 thereafter and also that a critical overstaffing is not encountered, a staff of 10,000 surveyors should be considered (this also considers the assumed productivity as specified in the conclusions). If a critical overstaffing of field surveyors will be encountered because attrition, reassignment, etc., cannot take place, then a fallback position to 9,000 or even 8,000 surveyors will have to be considered - this will have to be based upon the experience factor gained over the next several years.

With regard to the photogrammetric staff, a great amount of decision hinges on the total acceptance of

photogrammetric bases as valid Letter of Measurement base and to the extent orthophotograhic bases will be used. On the surface assumptions that there will be an increase of available aerial photgraphy for both rural and urban mapping (if nothing more than producing situation maps) by the aerial photographic contractors, then a buildup of possibly double or more of the staff should be considered.

The establishment of an additional "Subdirectorate of Basic Control Surveys" should be initiated. This Subdirectorate should be segmented into at least two sections. The first section would be the "Planning and Coordination" with responsibilities to maintain regular coordination with the other Subdirectorates of DPT on project areas and with BAKOSURTANAL on status of National control availability in order to prepare the control plans. The second section would be the "Mathematical and Adjustments" with responsibilities for final computational work on project control nets and ultimate adjustments and transformation of local control nets to each other and to the National net.

Program the purchase of one theodolite for every 2.5 field surveyors and, to approximately the same proportion of the present inventory of the other survey and associated equipment. The ratio of two one-second theodolites for every five 20 second theodolites to meet DPT needs will be required.

With a buildup to double the present photogrammetric staff and increase of aerial photographic coverage the purchase of at least one more rectifier, one more point transfer equipment will be needed and expansion of the photographic laboratory capability will be necessary.

Increased training capacity will have to be developed over the next four years to buildup to an expansion of surveyors from 250 a year to 500 a year.

Attachment A

Decree of the Director
of Directorate of
Land Registration.

Jakarta, January 8,1961
Number : 1/1961
Attachment: 5 exp of illustration
1 exp of explanation.

Head of Directorate of Land Registration

Considering: that with the effectiveness of Government Regulations No. 10, 1961 it is deemed necessary to arrange regulation and principal on the preparation of monuments of technical base points permanent mark.

In view of: Regulation of the Minister of Agraria No. 6/1961 article 3 point 4 article 4 point 5 and article 6

Decided

To stipulate: REGULATION AND PRINCIPAL ON THE PREPARATION OF PERMANENT MARKS OF TECHNICAL BASE POINT.

Chapter 1

The preparation of Monuments of Permanent Mark of Triangle Point.

Article 1

Primairy point monument of which form and size is arranged on the illustration of the attachment of this regulation and principal is made of concrete with mixture of 1:2:3 and constructed on its foundation of gravel mixed with sand of at least 2 dm thick. Part shown on the land surface is ± 1.4m height (see illustration 1 of the attachment).

Article 2

The secondary point monument of which form and size is arranged on the attach illustration of this regulation and principal is made of concrete with mixture of 1:2:3 and constructed on its foundation of gravel mixed with sand of at least 2 dm thick. Part shown on the land surface is ± 1.15m height.

Article 3

The tertiary point monument of which form and size is arranged on the attach illustration of this regulation and principal is made

of concrete with mixture of 1:2:3 and is constructed on its foundation of gravel mixed with sand of at least 2 dm thick. Part shown on the land surface is ± 0.9m height (see illustration on the attachment).

Article 4

Quarternary point monument of which form and size is arranged on the illustration of the attachment of this regulation and principal is made of concrete with mixture of 1:2:3 and constructed on its foundation of gravel mixed with sand of at least 2 dm thick. Part shown on the land surface is ± 0.61m high (see illustration 4 of the attachment).

Article 5

Subsidized material used for the preparation of triangle point according to the local condition is only permitted with the approval from the Head of Directorate of Land Registration.

Article 6

Monuments as explained above is mark with a signed of P.T. (Land Registration) while the underneath explained the level and number of the said monuments, such as:

P.T	P.T	P.T	P.T.
P10	S80	T100	Q200

Chapter II

The Preparation of Monument of Permanent Mark of Polygon

Article 7

1) The preparation of monument of Permanent Mark of Polygon Point is committed to the policy of K.K.P.P. (Head of Land Registration Office) and KKIDPT (Head of Land Registration Inspection Office) on its own area.

Article 8

This Letter of Decision is become effective on the date issued.

Based on the original Decree,
Head of Directorate of
Land Registration,

sgnd.

R. Hermanses S.H.

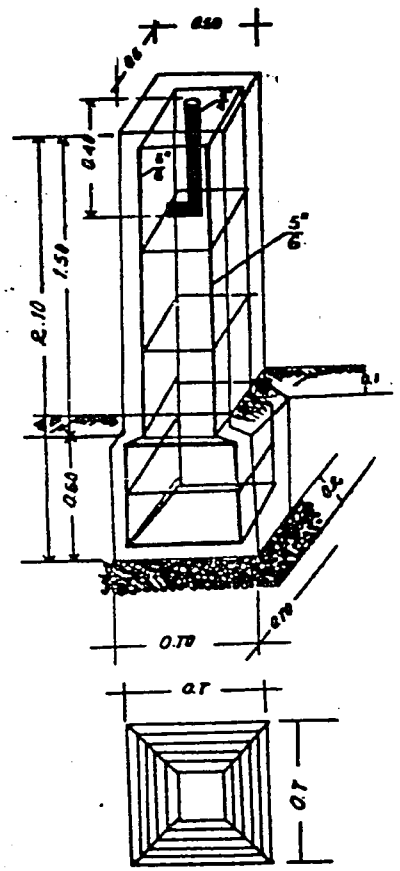
LAMPIRAN Surat Keputusan Kepala
Jawatan Pendaftaran Tanah
tgl. 8 Januari 1963 No.1/1963

BAB I.

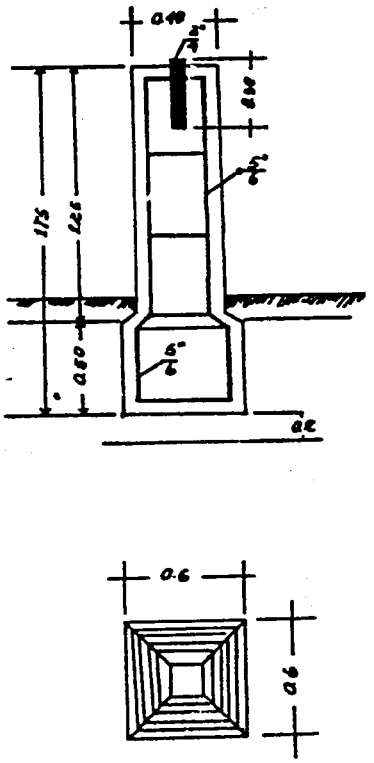
GAMBAR 1 dan 2

Tugu segi tiga Primair dan Secundair
(Pasal 1 dan 2)

Campuran 1 : 2 : 3



GAMBAR 1.



GAMBAR 2.

ATTACHMENT B

LAMPIRAN Surat Keputusan Kepala Jawatan Pendaftaran Tanah
tgl. 8 Januari 1963 No.1/1963

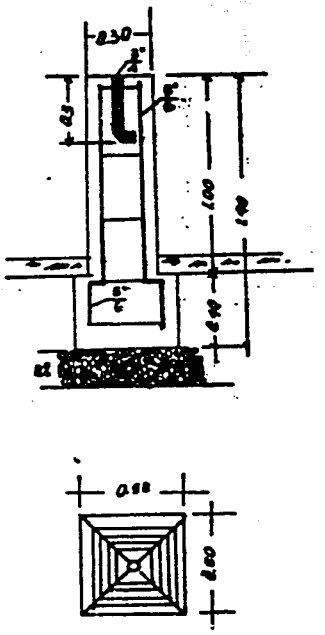
BAB I.

GAMBAR 3 dan 4

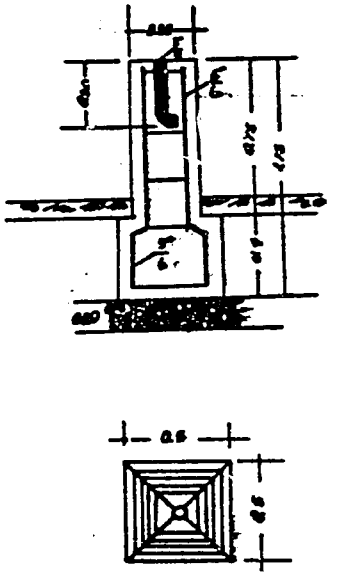
Tugu Segi Tiga Teriair dan Quartair
(Pasal 3 dan 4)

Campuran 1 : 2 : 3

GAMBAR 3.



GAMBAR 4.

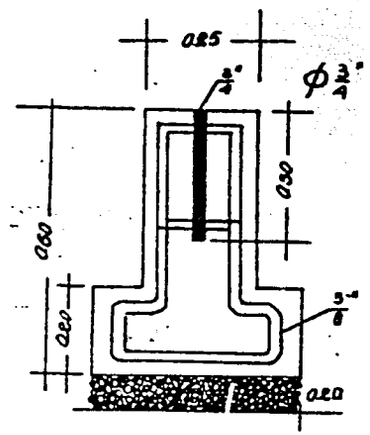


LAMPIRAN Surat Keputusan Kepala Jawatan Pendaftaran Tanah
tgl. 8 Januari 1963 No.1/1963

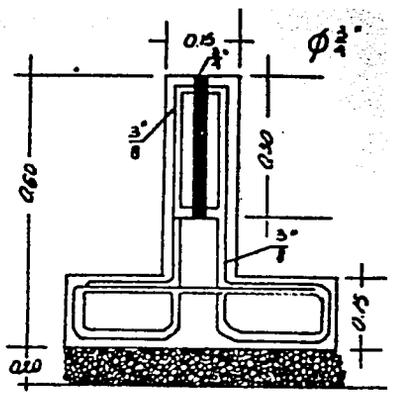
BAB II
GAMBAR 5 dan 6
Tugu Segi Banyak Primair
(Pasal 7)

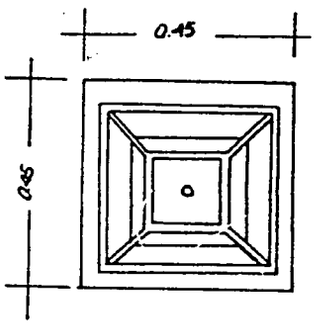
Campuran 1 : 2 : 3

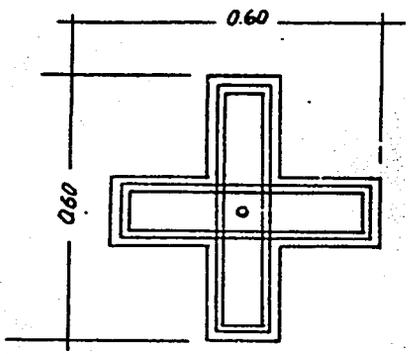
GAMBAR 5.



GAMBAR 6.



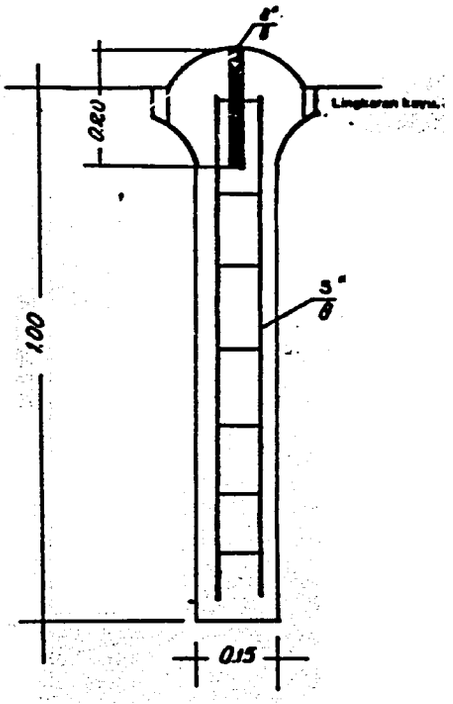


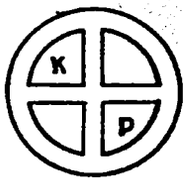


LAMPIRAN Surat Keputusan Kepala Jawatan Pendaftaran Tanah
tgl. 8 Januari 1963 no.1/1963

BAB II
GAMBAR 7
Tugu Segi Banyak Primair (Strauspaal)
(Pasal 7)

Campuran 1 : 2 : 3





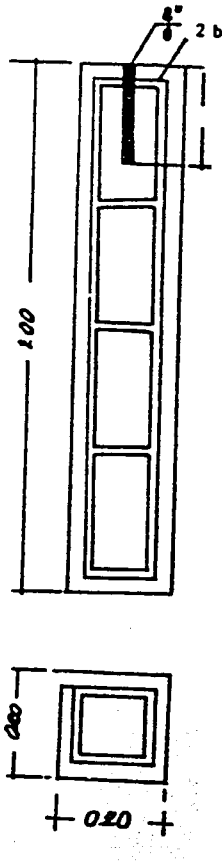
Lampiran surat keputusan Kepala Jawatan Pendaftaran Tanah.
tanggal 8 Januari 1963 No. 1/1963

BAB II

GAMBAR 8/9/10

Tugu Segi Banyak Secundair, Tertiair dan Quarir

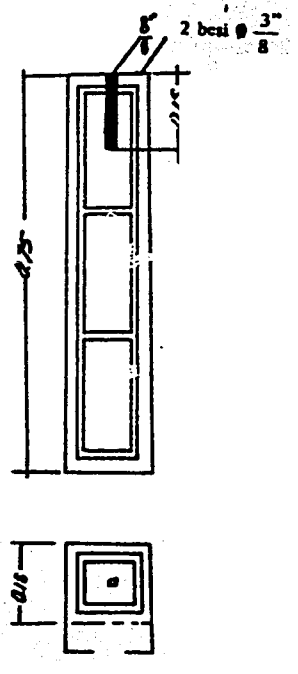
Gambar 8.



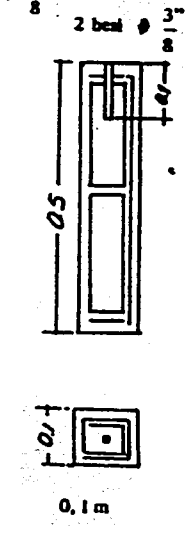
Perbandingan 1 : 10

Campuran 1 : 2 : 3

Gambar 9.



Gambar 10



0,1 m

Surat-keputusan Kepala Jawatan
Pendaftaran Tanah.

Jakarta, 8 Januari 1963

Nomor : 1/1963
Lampiran : 5 lembar gambar
1 penjelasan

KEPALA JAWATAN PENDAFTARAN TANAH.

Menimbang : bahwa dengan berlakunya PP.No.10/1961 dianggap perlu mengadakan peraturan dan pedoman tentang pembuatan tugu-tugu tanda tetap titik-titik dasar tehnik.

Mengingat : Peraturan Menteri Agraria no. 6/1961 pasal 3 ayat 4 pasal 5 ayat 1 dan pasal 6.

M E M U T U S K A N :

Menetapkan : Peraturan dan pedoman mengenai pembuatan tanda-tanda tetap titik dasar tehnik.

BAB I.

PEMBUATAN TUGU–TUGU TANDA TETAP TITIK SEGI TIGA.

Pasal 1.

Tugu titik primair yang bentuk dan ukurannya ditetapkan pada gambar dalam lampiran peraturan dan pedoman ini, dibuat dari beton bertulang dengan campuran : 1 : 2 : 3, dan dipasang dengan pondasinya atas dasar kerikil campuran pasir dengan tebal sekurang-kurangnya 2 dm. Bagian yang terlihat di atas permukaan tanah tingginya ± 1,4 m (lihat gambar 1 lampiran).

Pasal 2.

Tugu titik secundair yang bentuk dan ukurannya ditetapkan pada gambar dalam lampiran peraturan dan pedoman ini, dibuat dari beton bertulang dengan campuran 1 : 2 : 3 dan dipasang dengan

pondasinya atas dasar kerikil campuran pasir, dengan tebal sekurang-kurangnya 2 dm. Bagian yang terlihat di atas permukaan tanah tingginya ± 1,15 m (lihat gambar 2 lampiran).

Pasal 3.

Tugu titik tertiair yang bentuk dan ukurannya ditetapkan pada gambar dalam lampiran peraturan dan pedoman ini, dibuat dari beton bertulang dengan campuran 1 : 2 : 3 dan dipasang dengan pondasinya atas dasar kerikil campuran pasir dengan tebal sekurang-kurangnya 2 dm. Bagian yang terlihat di atas permukaan tanah tingginya ± 0,9 m (lihat gambar lampiran).

Pasal 4.

Tugu titik quarter yang bentuk dan ukurannya ditetapkan dalam gambar pada lampiran peraturan dan pedoman ini, dibuat dari beton bertulang dengan campuran 1 : 2 : 3 dan dipasang dengan pondasinya atas dasar kerikil campur pasir dengan tebal sekurang-kurangnya 2 dm. Bagian yang terlihat di atas permukaan tanah tingginya ± 0,65 m (lihat gambar 4 lampiran).

Pasal 5.

Mempergunakan lain bahan untuk pembuatan titik segitiga, berhubung dengan keadaan setempat, hanya dipergunakan dengan persetujuan K.D.P.T.

Pasal 6.

Tugu-tugu yang diuraikan di atas diberi tanda P.T. dengan di bawahnya tingkatan dan nomor dari pada tugu-tugu itu. Misalnya:

$\frac{P.T.}{10}$, $\frac{P.T.}{S\ 80}$, $\frac{P.T.}{T\ 100}$, $\frac{P.T.}{Q\ 200}$.

BAB II.

PEMBUATAN TUGU–TUGU TANDA TETAP TITIK SEGI BANYAK.

Pasal 7.

) Pembuatan tugu-tugu tanda tetap titik segi banyak diserahkan

kepada kebijaksanaan K.K.P.P. dan K.K.I.D.P.T. untuk daerahnya.

Pasal 8.

Surat-keputusan ini mulai berlaku pada hari ditetapkan.

Sesuai dengan surat keputusan asli.
Kepala Jawatan Pendaftaran Tanah,

ttd.

(R. Hermanses S.H.).—

Attachment B

BOUNDARY MARKS OF LAND

(Regulation of the Minister of Agrarian Affairs No. 8 of the year 1961 dated Sptember 7, 1961, T.L.N. No. . . . . .)

The Minister of Agrarian Affairs

Considering: It is deemed necessary to set up a regulation on boundary marks as to guarantee the boundaries of land-right

In view of
a) Article 19 of Basic Agrarian Law,
b) Article 3 point 7 and article 42 of the Government Regulation on Land Registration (Gov. Reg. No. 10 of the year 1961)

Decree

By revoking the State Gazette 1912 No. 497 (which is replaced by State Gazette 1918 No. 421 and State Gazette 1935 No. 54) and Regulation of the Minister of Agrarian Affairs No. 10 of the year 1951.

To Stipulate: REGULATION ON BOUNDARY MARKS OF LAND

Article 1

The boundary of every parcel of land should be indicated with boundary marks based on the provisions stated in this regulation

Article 2

For land parcels which is less than 25 Ha, boundary marks as mentioned in the following are used.

a. Iron pipe or bar-iron, of at least 1,00 m long with at least 0,03m diameter is burried into the land, while the remainder (0,20m) is covered and painted in red (minium). (See illustration 1 of this regulation's attachment), or

b. Stone wall monument of at least 0,30 sq.m. width and height of at least 0,40 m, in which part of it is buried into the land or foundation of at least 0,20m high and 0,40 sq.m wide (See illustration 2 of this regulation's attachment), or

c. Concrete monument or river stone is chiselled to at least 0,10 sq.m. and 0,50m long in which the 0,40m is buried into the land, If the said boundary mark is made of concrete then an iron nail is placed on the center of it (see illustration 2 of this regulation's attachment), or

d. For swamp areas we could use a kind of wood of at least 0,10 sq.m. and at least 1,50m long in which the 1 m is buried in land and the remainder is painted in red (minium). At around 0,20m from the bottom end at first we place 2 pieces of similar wood of at least 0.03 x 0.05 x 0.70 m in the form of a cross (see illustration 4 of the attachment of this regulation).

Article 3

For land parcels which is more than 25 ha, boundary marks as explained in the following are used.

a) Wall monument, at least 0.50 sq m. wide, height at least 0.60m high and placed on the foundation which is buried into the land with the size of at least 0.70 x 0.70 x 0.40 m (see illustration 5 of this attachment), or

b) Iron block steel rods of at least 3,00m long, 0.12m wide in which the 1.50m is buried into the land, at around 0.20m of the bottom end we place 2 pieces of iron bar which is formed as a cross with the size of 1.00x0.05x0.015m (see illustration 6 of this regulation's attachment), or

c) iron pipe of at least 3.00m long with outside diameter of at least 0.11m, in which the 1.50m is buried into the land; the upper side of the part shown on the land surface is covered with iron and painted in red (minium). At about 0.20m of the bottom end, 2 pieces of iron bar are placed which is formed as a cross whith the same size as boundary marks mentioned in point (b) of this article (see illustration 7 of this regulation's attachment), or

d) Waterproof wood could be used for swamp areas, such as iron wood of at least 0.15 sq.m, at least 3.00m long in which the 1.50 sq.m is buried into the land while part shown on the land surface is painted in red (minium). At about 0.30m from the bottom end we first place 2 pieces of the similar type of wood which is formed as a cross, with the size of at least 0.05x0.05x1.00m (see illustration 8 on the attachment of this regulation).

Article 4

If in any case the placement of boundary mark in one area will be too costly or because of the land condition the said boundary marks are considered as not good enough to guarantee boundary, then the Head of Land Registration Office concerned, with the approval from the Head of Land Registration Inspection Office, could indicate boundary marks in other form.

Article 5

Boundary mark mentioned in article 2 and 3 above should be placed on the boundary itself.

Article 6

Placement of boundary mark is carried-out on the efforts and responsibility of the party concerned, and if needed, according to the Instruction of the Head of Land Registration Directorate concerned.

Article 7

The Head of Land Registration Office will decide in which cases the boundary marks do not need to be placed.

Article 8

Matters which have not sufficiently regulated in this regulation, are to be arranged by the Head of Directorate of Land Registration.

Article 9

This regulation is in effect on the date of Land Registration as arranged on the Government Regulation on Land Registration (PP No. 1 of the year 1961) beginning to be implemented.

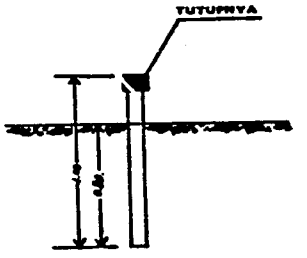
As to make people aware, this regulation will be attached on the additional State Gazette of the Republic of Indonesia.

Stipulated in Jakarta
On September 7, 1961
The Minister of Agrarian Affairs
sgnd.
(Mr. SADJARWO)

ATTACHMENT A

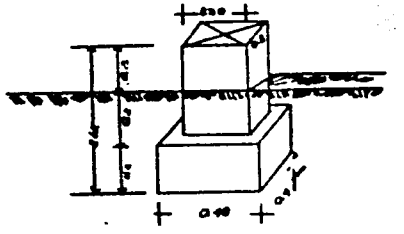
LAMPIRAN PMA No. 8/1961
TGL. 7 SEPTEMBER 1961.

PASAL 2 AYAT (1) a.



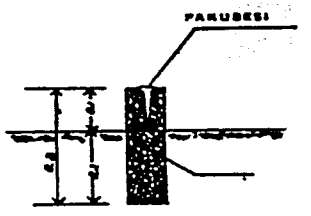
GAMBAR 1.

PASAL 2 AYAT (1) b.



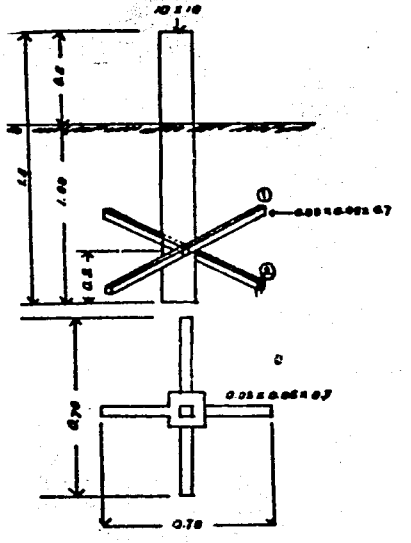
GAMBAR 2.

PASAL 2 AYAT (1) c.



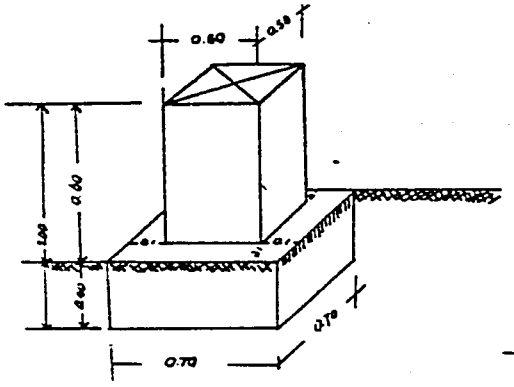
GAMBAR 3.

PASAL 2 AYAT (1) d.

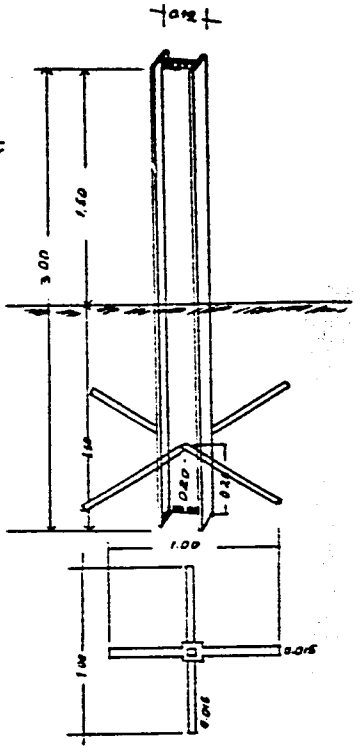


GAMBAR 4.

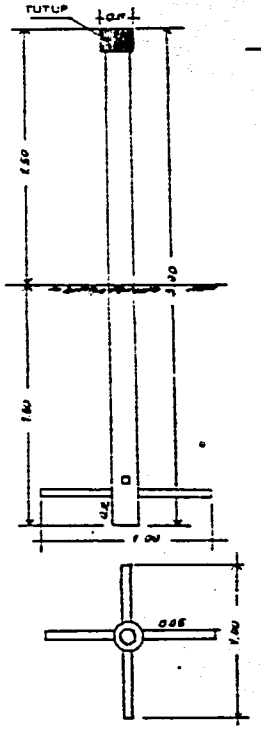
LAMPIRAN P.M.A. No. 8/1961
TGL. 7 SEPTEMBER 1961.



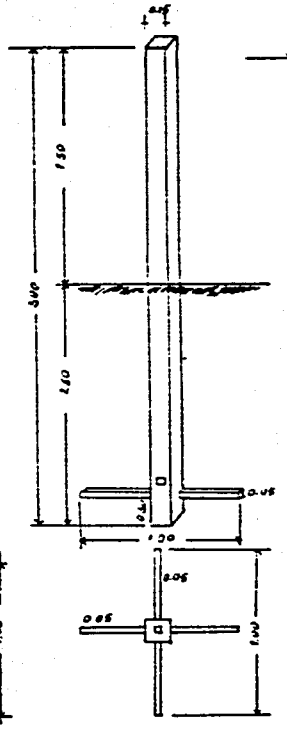
GAMBAR 5.



GAMBAR 6.



GAMBAR 7.



GAMBAR 8.

TANDA–TANDA BATAS TANAH–TANAH HAK.

(Per. Menag. No.8 Th. 1961 tg. 7 Sept. 1961 T.L.N. no. .....)

MENTERI AGRARIA

Menimbang : perlu diadakan peraturan tentang tanda-tanda batas untuk menjamin batas-batas tanah-hak.

Mengingat : a) pasal 19 Undang-undang Pokok Agraria.

b) pasal 3 ayat 7 dan pasal 42 Peraturan Pemerintah tentang Pendaftaran Tanah (P.P. No.10 tahun 1961).

M e m u t u s k a n :

Dengan mencabut Stbl 1912 No. 497 (yang diubah dengan Stbl 1918 No.421 dan Stbl 1935 No. 54) dan Peraturan Menteri Agraria No.10 tahun 1950.

Menetapkan PERATURAN TENTANG TANDA–TANDA BATAS TANAH–TANAH HAK.

Pasal 1.

Tiap-tiap tanah-hak batasnya harus dinyatakan dengan tanda-tanda batas menurut ketentuan-ketentuan dalam peraturan ini.

Pasal 2.

Untuk bidang-bidang tanah-hak yang luasnya kurang dari 25 ha, dipergunakan tanda-tanda batas seperti diuraikan di bawah ini :

a) Pipa besi atau batang besi, panjang sekurang-kurangnya 1.00 m dan bergaris tengah sekurang-kurangnya 0.03 m. dimasukkan ke dalam tanah, sedang selebihnya (0,20 m) diberi tutup dan dicat merah (meni) (lihat gambar 1 lampiran peraturan ini), atau

b) Tugu dari batu tembok sekurang-kurangnya besar 0,30 m. persegi dan tinggi sekurang-kurangnya 0,40 m, yang separoh

dimasukkan ke dalam tanah atau dasar sekurang-kurangnya tinggi 0,20 m dan besar 0,40 m persegi (lihat gambar 2 lampiran peraturan ini), atau

Tugu dari beton atau batu kali dipahat sekurang-kurangnya sebesar 0,10 m persegi dan panjang 0,50 m. yang 0,40 m dimasukkan ke dalam tanah; bila tanda batas itu dibuat dari beton di tengah-tengahnya dipasang paku dari besi (lihat gambar 3 lampiran peraturan ini), atau

Untuk daerah-daerah rawa dapat dipergunakan kayu yang tahan air, misalnya kayu besi, berukuran sekurang-kurangnya 0,10 m persegi dengan panjang sekurang-kurangnya 1,50 m, yang 1 m dimasukkan ke dalam tanah, sedang sebagian yang kelihatan di atas tanah dicat merah (meni). Pada kira-kira 0,20 m. dari ujung bawah terlebih dulu dipasang dua potong kayu sejenis dengan ukuran sekurang-kurangnya 0,03 x 0,05 x 0,70 m, yang merupakan salib (lihat gambar 4 lampiran peraturan ini).

Pasal 3.

Untuk bidang-bidang tanah-hak yang luasnya lebih dari 25 dipergunakan tanda-tanda batas seperti diuraikan di bawah ini:

Tugu dari tembok, sekurang-kurangnya besar 0,50 m persegi, tinggi sekurang-kurangnya 0,60 m dan berdiri di atas suatu dasar yang dimasukkan ke dalam tanah sekurang-kurangnya berukuran 0,70 x 0,70 x 0,40 m (lihat gambar 5 lampiran ini), atau

Besi balok atau rel kereta api sekurang-kurangnya panjang 3,00 m, lebar 0,12 m yang 1,50 m dimasukkan ke dalam tanah; pada kira-kira 0,20 m dan ujung bawah dipasang dua potong besi yang merupakan salib, yang berukuran sekurang-kurangnya 1,00 x 0,05 x 0,015 m. (lihat gambar 6 lampiran peraturan ini).

Pipa besi sekurang-kurangnya panjang 3.00 m bergaris tengah luar sekurang-kurangnya 0,11 m, yang 1,50 m dimasukkan ke dalam tanah; pada ujung di atas tanah diberi tutup dari besi dan dicat merah (meni; pada kira-kira 0,20 m dari ujung basah dipasang dua potong besi merupakan salib yang ukurannya sama dengan tanda batas yang dimaksud di bawah huruf (b) pasal ini (lihat gambar 7 lampiran peraturan ini).

d) Untuk daerah-daerah rawa dapat dipergunakan kayu yang tahan air, misalnya kayu besi berukuran sekurang-kurangnya 0,15 m persegi dengan panjang sekurang-kurangnya 3,00 m. yang 1,50 m dimasukkan ke dalam tanah sedang bagian yang kelihatan di atas tanah dicat merah (meni). Pada kira-kira 0,30 m dari ujung bawah terlebih dahulu dipasang dua potong kayu sejenis yang merupakan salib, dengan ukuran sekurang-kurangnya 0.05 x 0.05 x 1.00 m (lihat gambar 8 lampiran Peraturan ini).

Pasal 4.

Bilamana di sesuatu Daerah pemasangan tanda batas akan memakan biaya terlalu banyak atau disebabkan keadaan tanah, tanda-tanda batas itu dipandang tidak baik untuk menjamin batas, maka Kepala Jawatan Pendaftaran Tanah yang bersangkutan, dengan persetujuan Kepala Kantor Inspeksi Jawatan Pendaftaran Tanah, dapat menentukan tanda-tanda batas dengan bentuk lain.

Pasal 5.

Tanda batas yang dimaksud dalam pasal 2 dan 3 di atas, harus dipasang di atas batas.

Pasal 6.

Pemasangan tanda batas dikerjakan atas usaha dan tanggungan yang berkepentingan, dan bila perlu atas petunjuk Kepala Jawatan Pendaftaran Tanah yang bersangkutan.

Pasal 7.

Kepala Jawatan Pendaftaran Tanah menentukan dalam hal-hal mana tanda-tanda batas tidak perlu dipasang.

Pasal 8.

Hal-hal yang tidak diatur dalam Peraturan ini, diatur oleh Kepala Jawatan Pendaftaran Tanah.

Pasal 9.

Peraturan ini berlaku pada tanggal Pendaftaran Tanah sebagaimana

atur dalam Peraturan Pemerintah tentang Pendaftaran Tanah
'P.No.10 tahun 1961) mulai diselenggarakan.

Agar supaya setiap orang mengetahuinya, maka Peraturan
i akan dimuat dalam Tambahan Lembaran Negara Republik
donesia.

Ditetapkan di Jakarta
pada tanggal 7 September 1961
MENTERI AGRARIA

ttd.

(Mr. SADJARWO)

Specifications To Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys

By
Federal Geodetic Control Committee
John O. Phillips, Chairman

U.S. DEPARTMENT OF COMMERCE
Rogers C.B. Morton, Secretary

National Oceanic and Atmospheric Administration
Robert M. White, Administrator

Federal Coordinator for Geodetic Control and Related Surveys
David H. Wallace

National Ocean Survey
Allen L. Powell, Director

Rockville, Md.
July 1975

Specifications To Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys

PREFACE

The Office of Management and Budget published "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys" in February 1974. All Federal agencies are to comply with these standards in accordance with provisions of Circular A-16, Revised, "Coordination of Surveying and Mapping Activities," promulgated May 6, 1967. Specifically, all surveying activities financed in whole or in part by Federal funds must contribute to the National Networks of Geodetic Control when it is practical and economical to do so. In order to meet the geodetic control needs of Government agencies and the public at large, all sectors, both Government and private, are urged to adhere to the provisions of the Classification and Standards of Accuracy of Geodetic Control Surveys for all activities and to cooperate in assuring that all data are readily available to meet the public's need.

The Specifications have been prepared by the Federal Geodetic Control Committee to describe and explain the standards of accuracy and to provide guidance in methodology for surveying activities.

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Specifications To Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys

Introduction

To coordinate national mapping, charting, and surveying activities, the Board of Surveys and Maps of the Federal Government was formed December 30, 1919, by Execuive Order No. 3206. "Specifications for Horizontal and Vertical Control" were agreed upon by the Federal surveying and mapping agencies and approved by the Board on May 9, 1933. When this Board was abolished March 10, 1942, its functions were transferred to the Bureau of the Budget (now Office of Management and Budget) by Executive Order No. 9094. The basic survey specifications continued in effect. BOB Circular No. A-16, published January 16, 1953, and revised May 6, 1967, provides for coordination of Federal surveying and mapping activities. "Classification and Standards of Accuracy of Geodetic Control Surveys," published on March 1, 1957, replaced those of 1933. Exhibit C to Circular A-16, dated October 10, 1958, established procedures for the required coordination of Federal geodetic and control surveys performed in accordance with those classifications and standards.

The Federal Geodetic Control Committee (FGCC) was chartered December 11, 1968, and a Federal Coordinator for Geodetic Control and Related Surveys was appointed April 4, 1969. FGCC Circular No. 1, Exchange of Information, dated October 16, 1972, prescribes reporting procedures (vice Exhibit C to Circular A-16).

The evolution of economic and technological factors since 1957 made evident the need for greater flexibility in the design and performance of surveys. As a result, "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys" was published with the approval of the Office of Management and Budget in February 1974. This publication provides detailed specifications and general procedural guidance for surveys included in the National Geodetic Control Networks.

Geodesy (i.e., control surveys) is the common denominator that relates and coordinates the Nation's and the world's activities in a physical three-dimensional mode. The more commonly used geodetic data include latitudes, longitudes, elevations, deflections of the plumb line, and gravity values.

One of the multitude of uses for these data is the construction of maps and charts which, in turn, relate the horizontal and vertical positions between all cartographic features on a given map with those features on any other map of the Earth. The accuracy of cartographic presentations thus is directly related to the accuracy of the National Geodetic Control Networks.

From maps for tax assessment, construction, transportation system coordination, to maps of social and environmental data in a computer bank, adequate and accurate positioning is fundamental. These specifications are promulgated for use by both the public and private sectors to aid in ensuring that control surveying activities will be systematically accomplished to meet the Nation's needs.

Control surveys consist of horizontal, vertical, gravimetric, and astronomic surveys. Horizontal control surveys determine geographic positions referenced to a national datum and provide the basis for rectangular coordinate systems. Vertical control surveys determine elevations referred to a national datum referenced to tidal measurements.

Measurements of gravity and astronomy provide data for the establishment and adjustment of the national control networks. Surveys of large areas must take into account the curvature of the Earth. For small areas, such as a farm, a city lot or even a small

city, the curvature may be ignored. Larger areas must be surveyed by methods which recognize that the mathematical figure of the Earth is a slightly flattened sphere, or an ellipse of revolution.

Horizontal control is established by triangulation, trilateration, and traverse procedures.* Triangulation is a system of joined or overlapping triangles in which the length of an occasional side, known as a base line, is measured and the other sides are computed from angles measured at the triangle vertices. Trilateration is a method of surveying in which the lengths of the triangle sides are measured. Traverse is a method of surveying in which a sequence of lengths and directions of lines between points on the Earth are measured and used in determining positions of the points. Triangulation, trilateration, and traverse procedures may be used singly or in combination to obtain the accuracy and precision required operationally. Vertical control is established by spirit leveling of a high order of accuracy. It consists of the elevations of points with respect to each other and to a common datum. Geodetic leveling follows the geoid and its associated level surfaces which are irregular, rather than the mathematically determined ellipsoid.

The Classification and Standards of Accuracy provide the permissible tolerances for the indicated order and class of control. These Specifications provide the guidance to obtain (for horizontal control) the specified strength of figure, spacing of Laplace azimuths, number of observations, triangle closures, length checks, and station spacing, and (for vertical control) the line spacing, field procedures and section closures to be followed to satisfy the standards. They are a directive for surveys that are to be assimilated into or considered a part of the National Geodetic Control Networks. The Federal Geodetic Control Committee recommends that all other control or precise engineering surveys adhere to the Specifications and be properly referenced to the National Networks.

National Geodetic Control Networks

The Department of Commerce is responsible for establishing and maintaining the geodetic control networks to meet the needs of the Nation. This responsibility evolves from legislation dating back to the Act of February 10, 1807 (2 Stat. 413), which resulted in the creation of the "Survey of the Coast." The current authority is contained in United States Code, Title 33, USC 883a as amended, and specifically defined by Executive Directive, Office of Management and Budget Circular No. A-16, Revised.

The Federal Coordinator for Geodetic Control and Related Surveys, Department of Commerce, is assigned the responsibility for coordinating, planning, and executing national geodetic control surveys and related survey activities of Federal agencies, financed in whole or in part, to the end that:

1. The geodetic control needs of governmental agencies and the public at large are met in the most expeditious and economical manner possible with available resources; and
2. all surveying activities financed in whole or in part by Federal funds contribute to the National Geodetic Control Networks when it is practicable and economical to do so.

The Federal Geodetic Control Committee assists and advises the Federal Coordinator for Geodetic Control and Related Surveys.

Accuracy

The standards and specifications for the various orders and classes of accuracy are summarized in tables 2 and 3 of the "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys." Table 2 refers to horizontal control surveys and particularly to the procedures employed, i.e., triangulation, trilateration, and traverse. Table 3 refers to vertical control surveys. It should be noted that the standards of accuracy prescribed pertain only to field observations.

Specifications may be defined as the total effort deemed necessary to obtain certain results that are considered the basis for a prescribed standard. The precision of the effort expended contributes largely to the acceptance or rejection of the results; for *precision* may be defined as the degree of perfection *used*, and *accuracy* the degree of perfection *obtained*. Precision involves, among other considerations, the quality of instruments employed, the methods used, and the ability and experience of personnel.

Although there is no absolute guarantee that a particular standard will be met if all slated specifications are followed, it is reasonably certain that the closures in length and position will be about one-half that stated for a particular standard. One

* In special cases astronomic position determinations or positions from electronic navigation or satellite-based systems may be used.

of the factors and perhaps the major contributor to survey inaccuracies is undetected and/or uncorrected atmospheric anomalies. These anomalies, regardless of rigid adherence to the specifications, occasionally remain undiscovered until future surveys are extended from the stations involved, even though a careful and extensive evaluation of the observational data may have been made in the office.

In the final analysis, the published accuracies for primary and principal stations of the National Geodetic Control Networks are derived through comprehensive reviews of the specifications employed, the geometric design of the survey, and statistical evaluations determined from the adjustment. These accuracies are in relation to neighboring stations positioned to similar specifications, and take into consideration whether or not the points are directly connected. Further reference to accuracy in this document will often be described by the term "uncertainty" which implies in all cases, whether so stated or not, the relationship of adjacent or neighboring stations.

Horizontal Control

Generally, the density and accuracy of permanently marked control points are directly related to land values and local needs. Table I is a synopsis of the classification and standards of accuracy for the various components that are assimilated in designing the National Horizontal Control Network. The National Horizontal Control Network must be of such accuracy and precision that, everywhere on the North American Continent, control (both in density and accuracy) meets user needs or national interest. The "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys" was formulated on this concept.

Nationwide High-Precision Traverses

These traverses provide scale for the worldwide satellite triangulation network and upgrade the scale and orientation of the National Horizontal Control Network. They consist of a series of high-precision length, angle, and azimuth determinations coincident with geoidal profiles, running approximately east-west and north-south through the conterminous

TABLE I.—*Synopsis of horizontal control classifications*

ORDER, CLASS	SUPERIOR	FIRST-ORDER	SECOND-ORDER, CLASS I	SECOND-ORDER, CLASS II	THIRD-ORDER CLASS I, II
General title	Transcontinental control.	Primary horizontal control.	Secondary horizontal control.	Supplemental horizontal control.	Local horizontal control.
Purpose	Transcontinental traverses. Satellite observations. Lunar ranging.	Primary arcs. Metropolitan area surveys. Engineering projects.	Area control. Detailed surveys in very high land value areas.	Area control. Detailed surveys in high value land areas.	Area control. Detailed surveys in moderate and low land value areas.
Network design	Control develops the national network		Control strengthens the national network.	Control contributes to the national network.	Control referenced to the national framework.
Accuracy	1:1,000,000	1:100,000	1:50,000	1:20,000	1:10,000 1:5,000
Spacing	Traverses at 750 km. Spacing—stations at 15 to 30 km or greater. Satellite as required.	Arcs not in excess of 100 km. Stations at 15 km. Metropolitan area control 3-8 km.	Stations at 10 km. Metropolitan area control at 1-2 km.	As required.	As required.
Examples of use	Positioning and orientation of North American Continent. Continental drift and spreading studies.	Surveys required for primary framework. Crustal movement. Primary metropolitan area control.	Metropolitan area densification. Land subdivision. Basic framework for densification.	Mapping and charting. Land subdivision. Construction.	Local control. Local improvements and developments.

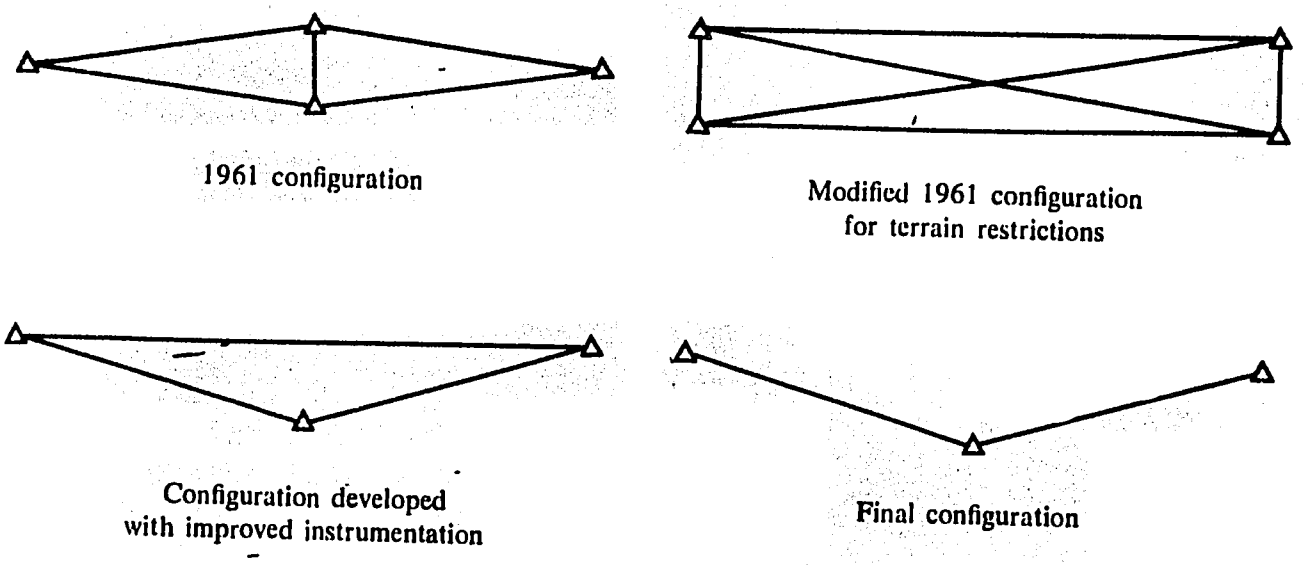

Fig. I—Configurations—National High Precision Traverses

States forming somewhat rectangular loops. Smaller loops and spur traverses are added to connect satellite triangulation stations and areas of special interest.

Complete specifications are not provided herein for high-precision traverses. The following outline of procedures indicates the care required to obtain the approximate 1 part in 1,000,000 accuracy. This order of work utilizes methodology and instrumentation to obtain the highest accuracy possible within the state of the art.

As specified originally (1961), the basic figure was an elongated, four-sided polygon, the long sides of which formed almost parallel traverses. The polygons were connected at the ends of the elongated figures, thus forming a continuous line or chain. Each polygon consisted of two slim triangles created by connecting the two nearby midpoint stations. These midpoint stations were selected to ensure significant differences between the length of the long sides and to maintain sufficient strength of figure to permit a good mathematical check between the measured lengths. The midpoint stations were usually located within 30 to 50 m of one another and generally consisted of a standard station monument and a monumented auxiliary point.

Horizontal angles, including closing the horizon, were measured to First-Order specifications on at least two nights by different observers. When the means of the two sets of angles differed by more than 1 second, a third set was taken on a third night. The average closure of the polygons could not exceed 0".7 and the maximum could not exceed 2".0. In general, the closures in seconds of the slim triangles were not to exceed 600 divided by the short length in meters. The short distances between the midpoint stations were measured using base line taping procedures.

Reciprocal vertical angles were observed over all traverse lines with ties to bench marks required every 4 to 5 polygons. Simultaneous reciprocal observations were required where the elevation differences exceeded 100 m. Where differences of elevations of stations exceeded 500 m, simultanous reciprocal observations were required both before and after the length measurement of each line. These observations were secured to correct the length observations for variations in atmospheric conditions.

Astronomical position and azimuth observations were required at the connecting stations of the polygons. If the distance between these stations exceeded 35 km, an astronomical position and azimuth determination was required at one of the midpoint stations. First-Order and modified First-Order (one night's observations) astronomic positions were specified. For azimuth determination, observed directions to all adjacent stations were generally included with the direction to Polaris. Azimuth ob-

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servations were taken on two nights with a different observer and instrument each night. The average difference between two nights' observations was not to exceed 2".5 and a combined standard error for the two nights not to exceed ±0".45. If either limit was exceeded, a third night's observation was required with two observers, using different instruments, taking succesive sets of 16 positions.

Distance measurements were made with electro-optical distance measuring instruments. At least two instruments were used over each line with the measurements made on different nights. Instrument frequencies were checked each week and recorded results were made part of the record of observations made during that week. In 1962 the offset mirror procedure was introduced. This technique involved making one or two measurements with the mirror centered over the point and two measurements offset ±0.4 m on the line. The check between the mean distances measured on different nights was not to exceed 17 mm plus 1 part per million (ppm). Distances projected through the slim triangles must agree with the measured lengths of the opposite sides within 25 mm. Meteorological observations consisted of temperature, barometric (altimeter), and humidity readings at instrument height with mid-line temperatures at line height obtained using remote reading thermometers supported by balloon. Angle and distance observations were made at least 10 m above the ground to avoid erratic temperature variations. Observers were alerted to the fact that unfavorable refraction conditions may exist on calm nights and that extreme care must be exercised.

Great care was used in collimating towers and in measuring eccentricities. When the wind velocity or direction changed during the night's observations, the tower was recollimated and the eccentricity remeasured.

With the adaptation of a laser light source to the electro-optical distance measuring instruments, the range was significantly increased. As a result of this technical improvement and a thorough evaluation of data secured over many years, several modifications in the original specifications were instituted. The original design was developed primarily to assure that blunders and ambiguities in measurement would be uncovered and resolved. Advanced instrumentation, observing techniques, and procedures have eliminated almost completely the possibilities of blunder.

When it was possible to measure the distance between connecting stations, only one midpoint station was required. The smaller angles of the resulting slender triangle could seldom be larger than 5° and could never exceed 10 . The closure could not exceed 1".7. The check between the measured long side and the projected value derived from the short sides was not to exceed 17 mm plus one part per million of the longest side.

Further modifications were made as the project progressed. Distance measurements were required on one night only. However, the sequence of observations was such that the distances involved in the projections were measured on different nights employing different instruments. In those cases where the figure consists of a single slender triangle, the short sides were measured on one night and the long side on another night with a different instrument. Also, the midline temperature requirements were deleted. Temperature observations were required at the end points only, provided readings were taken at minimum 10-m elevation above ground level, and the line of sight cleared all ground elevations by 10 m.

Continuing studies showed conclusively that when proper design is utilized and extreme care is exercised, single line traverse would approach the accuracy and internal consistency of past configurations. The network was completed using this form of traverse. All observations were made on at least two nights with different observers and instruments, and astronomical positions and azimuths were determined at each station with intermediate astronomic determinations required when the length of line exceeded 35 km. The various tolerances specified for these observations remained unchanged.

First-Order (Primary Horizontal Control)

The primary horizontal control of the National Network consists of monumented stations established with a positional accuracy specification so that the uncertainty between adjacent points should not exceed 1 part in 100,000. Higher accuracy specifications, such as those for the high precision nationwide traverses, have been described previously and may be used in lieu of these specifications when required by special circumstances.

Special surveys for the study of movements in the Earth's crust, for high precision engineering projects, and for the testing of defense or space equipment should also have an accuracy of at least 1 part in 100,000 and in some instances even 1 or 2 parts in 1,000,000.

Second-Order, Class I (Secondary Horizontal Control)

This class consists of monumented stations established with a positional accuracy specification that the uncertainties between adjacent points should not exceed 1 part in 50,000. This control, established between areas bounded by the high precision traverses or the primary horizontal control network, strengthens the network and provides a more advantageous spacing for local use.

In metropolitan areas, when there is a requirement for additional control points more closely spaced than those established under First-Order specifications, the additional points should be established by Second-Order, Class I procedures.

Second-Order, Class II (Supplemental Horizontal Control)

This class of control is established with positional accuracy specifications so that the uncertainly between adjacent points should not exceed 1 part in 20,000. The demands for reliable horizontal control surveys in areas which are not in a high state of development, or where no such development is anticipated in the near future, justify the need for this classification. Although points established for the network should be permanently monumented, there may be occasions where, for various reasons, a few points marked in a temporary fashion and interspersed within the survey system may be justified in this class of work.

In metropolitan area surveys, there is usually a requirement for a second breakdown of control points more closely spaced than those established to Second-Order, Class I specifications. This second breakdown may be performed at Class II standards. Depending upon the length of line and other considerations, the traverse specifications related to the number of angle measurements and azimuth closures may be relaxed somewhat. Specifically, modifications lessening the total effort by as much as one-half may be justified when the traverse courses are not longer than 800 m and the number of angle points between higher order control do not exceed 10.

See table 2 of the "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys" for the recommended number of observations and azimuth closure tolerances for these cases.

Third-Order, Class I and Class II (Local Horizontal Control)

Surveys of this order are established to specifications and by methods that are expected to provide relative accuracies between adjacent stations of not less than 1 part in 10,000 and 1 part in 5,000, respectively. Such control nets should be used only to support surveys of lower accuracy within areas of limited extent.

Third-Order surveys should be permanently marked, adequately described, and carefully connected to the National Network. Spires, stacks, standpipes, flagpoles, and other identifiable objects located to this accuracy have significant value for many surveying and mapping projects.

Specifications for surveys below Third-Order are not included herein.

Triangulation

The primary method of establishing horizontal control is triangulation, a procedure of determining the lengths of the sides of a system of joined or overlapping triangles by measuring occasional side lengths and computing the others from angles measured at the vertices. In table 2 of the "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys," under each order and class, there are three specifications pertaining to horizontal directions.

Horizontal Directions

First, the instruments must be of specified quality. First-Order instruments are defined as optical reading theodolites with micrometer readings of less than 1 second. The Kern DKM-3 and Wild T-3 are representative of this quality. Second-Order refers to a smaller optical reading theodolite with micrometer readings of 1 second, such as the Askania A2, Kern DKM-2, Wild T-2, or Zeiss Th-2. Good quality surveyor's transits or repeating theodolites are acceptable for Third-Order surveys but are not recommended because of the extra effort required to obtain the specified accuracies. The suggested number of observations for various types of transits is given in table II.

The second and third specifications relate to the number of positions to be observed and the rejection limits. These criteria, derived from experience and statistical evaluations, are based on the accuracy to be obtained, the instrument used, and to some extent the length of the lines involved. The specifi-

TABLE II—*Suggested number of observations using transit or repeating-type instruments*

The circle settings are given in table III.

Accuracy class	Transit*	No. of observations	No. of sets	Spread between D&R & sets not to exceed
Third-Order Class I triangulation	10"	6 D&R	2-3	4'
	20"	6 D&R	4-5	5'
	30"	6 D&R	6-8	6'
Traverse	10"	6 D&R	1-2	5'
	20"	6 D&R	2-3	6'
	30"	6 D&R	3-4	7"
Third-Order Class II triangulation	10"	6 D&R	1-2	5"
	20"	6 D&R	2-3	6"
	30"	6 D&R	3-4	7"
Traverse**	10"	2 D&R	1	5"
	20"	4 D&R	1	6"
	30"	6 D&R	1	7"

*When optical transits reading to 0.1 minute are employed, the "No. of Observations" may be reduced from 6 D&R to 4 D&R. For Third-Order, Class II Traverse, 2 D&R continues to be recommended. The "No. of Sets" and "Spread Between D&R and Sets Not to Exceed" remain unchanged. Use appropriate circle settings from Table III.

** The specifications given here are for traverses containing fewer than 10 courses between higher order points. For traverses containing more courses between higher order control, the "Number of Sets" in each case should be increased to two.

A set of observations consists of six (or otherwise specified) repetitions of the angle with the telescope in the direct (or reversed) position, followed by six (or otherwise specified) repetitions of the explement of the angle in the reversed (or direct) position.

cations tabulated for First-Order and Second-Order, Class I, must always be satisfied.

For Second-Order, Class II, or lower-order surveys, particularly traverses containing very short courses, the number of positions and rejection limits may be modified to fit the circumstances.

The observations should be made using procedures that minimize collimation, circle, and micrometer errors. Employing the circle settings (table III) will do much to minimize or eliminate these instrument errors.

When high precision is required, special care must be exercised in observing horizontal angles over inclined lines. The level of the instrument must be carefully maintained for lines that incline as much as 2°. For lines inclined over 5°, it is desirable to record striding-level or plate-level readings, using the procedures established for astronomic observations, and to correct the directions involved accordingly.

Observers should be aware of the importance of carefully centering the instrument and targets, eliminating phase in targets, protecting the instrument from vibrations caused by the wind, the heating effects of the sun, and the overriding necessity for stability of support. In addition, the scheduling of observations should take into account those situations where adverse horizontal refraction is likely to be present. In these cases, the time selected to make the observations should be when atmospheric conditions are best suited to obtain optimum results.

Procedures and precautions for use in high precision surveys described in detail in Coast and Geodetic Survey, "Manual of Geodetic Triangulation," Special Publication No. 247, pp. 9, 11, 17, 111, 131, and 141, are still valid. References to Third-Order traverse surveys will be found in Coast and Geodetic Survey, "The State Coordinate Systems, (A Manual for Surveyors)," Special Publication No. 235, and to Third-Order triangulation and traverse

TABLE III.—*Circle settings*

Two positions of circle

	10-Minute micrometer drum		
1	0°	00′	10″
2	90	05	40

Four positions of circle

	5-min micrometer drum			*10-min micrometer drum*			*Wild T-3** *Circle*		*Micrometer readings (units)*
1	0	00′	40″	0	00′	10′	0°	00′	10
2	45	01	50	45	02	40	45	00	25
3	90	03	10	90	05	10	90	00	35
4	135	04	20	135	07	40	135	00	50

Six positions of circle

1	0	00	10	0	00	10	0	00	15
2	30	01	50	30	01	50	30	00	35
3	60	03	30	60	03	30	60	00	50
4	90	00	10	90	05	10	90	00	15
5	120	01	50	120	06	50	120	00	35
6	150	03	30	150	08	30	150	00	50

Eight positions of circle

1	0	00	40	0	00	10	0	00	10
2	22	01	50	22	01	25	22	00	25
3	45	03	10	45	02	40	45	00	35
4	67	04	20	67	03	55	67	00	50
5	90	00	40	90	05	10	90	00	10
6	112	01	50	112	06	25	112	00	25
7	135	03	10	135	07	40	135	00	35
8	157	04	20	157	08	55	157	00	50

Twelve positions of circle

	5-min micrometer drum			*10-min micrometer drum*			*Wild T-3** *Circle*		*Micrometer readings (units)*
1	0°	00′	40″	0°	00′	10″	0°	00′	10
2	15	01	50	15	01	50	15	00	25
3	30	03	10	30	03	30	30	00	35
4	45	04	20	45	05	10	45	00	50
5	60	00	40	60	06	50	60	00	10
6	75	01	50	75	08	30	75	00	25
7	90	03	10	90	00	10	90	00	35
8	105	04	20	105	01	50	105	00	50
9	120	00	40	120	03	30	120	00	10
10	135	01	50	135	05	10	135	00	25
11	150	03	10	150	06	50	150	00	35
12	165	04	20	165	08	30	165	00	50

Sixteen positions of circle

1	0	00	40	0	00	10	0	00	10
2	11	01	50	11	01	25	11	00	25
3	22	03	10	22	02	40	22	00	35

TABLE III (continued)

Sixteen positions of circle

	5-min micrometer drum			10-min micrometer drum			Wild T-3* Circle		Micrometer readings (units)
4	33°	04′	20″	33°	03′	55″	33°	00′	50
5	45	00	40	45	05	10	45	00	10
6	56	01	50	56	06	25	56	00	25
7	67	03	10	67	07	40	67	00	35
8	78	04	20	78	08	55	78	00	50
9	90	00	40	90	00	10	90	00	10
10	101	01	50	101	01	25	101	00	25
11	112	03	10	112	02	40	112	00	35
12	123	04	20	123	03	55	123	00	50
13	135	00	40	135	05	10	135	00	10
14	146	01	50	146	06	25	146	00	25
15	157	03	10	157	07	40	157	00	35
16	168	04	20	168	08	55	168	00	50

Transit and repeating type instruments

Sets	Instrument** 10″ setting			Instrument*** 20″ setting			Instrument*** 30″ setting		
1	0°	00′	00″	0°	00′	00″	0°	00′	00″
2	90	05	30	90	10	20	90	10	30
1	0	00	00	0	00	00	0	00	00
2	60	03	30	60	06	20	60	06	30
3	120	07	00	120	13	00	120	13	00
1				0	00	00	0	00	00
2				45	05	20	45	05	30
3				90	10	00	90	10	00
4				135	15	20	135	15	30
1				0	00	00	0	00	00
2				36	04	20	36	04	30
3				72	08	00	72	08	00
4				108	12	20	108	12	30
5				144	16	00	144	16	00
1									
2							0	00	00
3							30	03	30
4							60	07	00
5							90	10	30
6							120	14	00
							150	17	30
1									
2							0	00	00
3							25	02	30
4							51	05	30
5							76	08	00
6							102	10	30
7							128	14	30
							153	17	00

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TABLE III (continued)

Transit and repeating type instruments					
	*Instrument*** 10" setting*	*Instrument**** 20" setting*	*Instrument**** 30" setting*		
Sets					
1			0°	00′	00″
2			22	02	30
3			45	05	00
4			67	07	30
5			90	10	00
6			112	12	30
7			135	15	00
8			157	17	30

The Kern DKM-3 theodolite is an example of an instrument where the micrometer drum has a range of 5 minutes. The Wild T-2 and the Kern DKM-2 theodolites are examples of instruments whose micrometer drums have a range of 10 minutes.

* For Wild T-3 theodolites, beginning with Serial Number 91084, the least graduation of the circle is 2 minutes. For these instruments, the micrometer readings shown in the tables as units would be seconds.

** Repeating theodolite with a 10-min circle.

*** Transit with a 20-min circle.

in Coast and Geodetic Survey, "Manual of Second- and Third-Order Triangulation and Traverse," Special Publication No. 145.

Triangle Closures

Specifications related to triangle closures are the simplest test available in the field to ascertain the accuracy of triangulation observations. However, this is but one of two tests that can be made and perhaps the least reliable for evaluating the network accuracies. The side check is the second test and probably furnishes the best overall analysis.

The triangle closure tolerances specified for First- and Second-Order triangulation should seldom be exceeded. For First-Order triangulation, it is generally expected that the average triangle closure will be about 0″.8 and the maximum closure, 2″.5.

Side Checks or Side Equation Tests

In addition to meeting specifications for average and maximum triangle closures, the lengths of the common sides of triangles in the figures as computed through various chains must agree within specified limits. To obtain a side check for a regular quadrilateral in which the triangles have been closed by applying one-third of the misclosure to each angle, the logarithms of the length of a common side, as computed through the R_1 and R_2 chains, should not differ by more than the tabular difference for 1 second of the log sine of the smallest angle involved, multiplied by the factor given below for a particular standard:

First-Order	Second-Order Class I	Second-Order Class II	Third-Order Class I	Third-Order Class II
1.5	1.5-2	2-4	4	10-12

In First-Order triangulation involving figures other than regular quadrilaterals, a comparable limiting value for side checks can be obtained through the following formula: 0.4 x Tab. Diff. x Σ A's. In this formula, "Tab. Diff. is the tabular difference for 1 second of the log sine of the smallest angle used in the computations and Σ A's is the total number of triangles involved. For Second-Order Class I and II and Third-Order Class I and II triangulation, the constant factor in the equation (0.4 for First-Order) is 0.5, 0.8, 1.0 and 3.0, respectively.

In cases where the triangles are computed using natural functions, the side check test is performed in a different and slightly more complex manner which is described in Leaflet-Table and Graph for Side Checks (1961), available from the National Geodetic Survey.

Side equation tests may also be applied and are recommended as a standard practice. The average correction to a direction is obtained by dividing the constant term of the equation by twice the sum of the tabular differences for 1 second of the log sines for the observed angles involved in the equation. The summation is made without regard to sign and the result of the division should not be greater than the value shown in table 2 of "Classification, Standards Accuracy and General Specification of Geodetic Control Surveys". Side equation tests should

be applied to all figures with excessive triangle and side closures as an aid in isolating the large errors.

In area triangulation, all side equations should be prepared either in a clockwise or counterclockwise order. When the signs of the constant terms are predominantly plus or minus, there is an error accumulation that should be investigated, regardless of the fact that the side check specifications have been satisfied. A careful review may indicate locations where repeat observations might improve the situation, but often there will be little evidence of the source of the problem. In this event, rather than expend a large effort, the measurement of a few lines in the affected locality will generally ensure that the specified accuracy is maintained.

Trilateration

Trilateration is a method of surveying in which the lengths of the sides are measured. The availability of electronic distancing equipment has made this procedure economically feasible and in many instances will provide accuracies superior to conventional triangulation and traverse.

The following specifications are a result of extensive field tests performed by the National Geodetic Survey. However, they must be viewed as provisional until further tests have been concluded under more diverse conditions, with varying length of lines and configurations, and utilizing other types of equipment. To date, most of the tests have been carried out with electro-optical instruments in projects involving lengths of 1 to 70 km.

The principal objections to trilateration in the past were the presumptions that complex figures were required to obtain the equivalent redundancies of triangulation and that additional astronomic azimuths were needed to control the orientation of such networks. While these presumptions may have had some validity, tests to date have not shown any need to increase the number of astronomic azimuths over that specified for triangulation. Furthermore, these evaluations show that the single geometric redundancy afforded by the conventional quadrilateral will suffice for arc-type surveys.

To maintain the strength of figure, the quadrilaterals should approximate a square and should seldom contain angles less than 30° and never less than 25° for First-Order and Second-Order, Class I surveys unless the accuracy of the lengths can be shown to be better by at least 50% of the specified standard. In no case, however, should any angles be less than 20° for these accuracy classes regardless of the improvement over the length accuracy standard.

Central point quadrilaterals with one measured diagonal provide two geometric redundancies for each figure and are also satisfactory, but for economic reasons should be avoided except in those cases where both diagonals of the standard quadrilateral cannot be measured. The angle restrictions would also apply to the principal chains, but angles as small as 5° are acceptable in one triangle involving the central point station. In general, however, the smallest angle in this triangle should seldom be less than 10°.

Four-sided and other multisided central point figures without diagonals provide a single redundancy and would be satisfactory for all accuracy classes. The smallest angle restriction specified for the conventional quadrilateral would generally apply, but it would be best to limit the smallest angle to 35° in cases where figures with 5 or more sides are involved in First-Order or Second-Order Class I surveys.

Rather limited investigations indicate that, for Second-Order, Class II and all Third-Order surveys, the recommended configurations can be modified to a large extent, provided the actual accuracies of the measured lengths are better than the specified standard by a factor of at least two. Elongated figures with angles as small as 10° (or less in some cases) can probably be employed for both classes of Third-Order work in these instances. However, it would be prudent to observe some of the larger angles in each figure.

A chain of single triangles has no check except when ties are made to control points or azimuths are observed along the route. This type of figure must be used with extreme caution and rarely, if ever, in surveys where accuracies better than Third-Order, Class I are anticipated. It is also prudent to observe at least one angle in each triangle.

Geometric restrictions, which limit the selection of locations in which stations may be established, serve to limit the utility of trilateration networks where the purpose is to place control at specifically needed sites. These restrictions also complicate the problem of selecting optimum high-ground locations to obtain the necessary instrument heights. The normal requirement to establish reference and azimuth marks and to determine the positions of intersected objects means that precise angle measurements are required at all stations. It is expected, therefore, that few network surveys will be performed as pure

trilateration projects. In recent years, however, lengths of many additional lines of the conventional triangulation networks have been measured, thus greatly strengthening the network and in some cases taking advantage of the trilateration technique to strengthen the angular measurements.

Electronic Distance Measurements

Distances measured using Electronic Distance Measuring (EDM) equipment are subject to errors arising from the instrumental components, calibration of the equipment, inaccuracies in the meteorological data, elevation discrepancies, and the centering of the instruments or reflectors. All these factors, in addition to the operating procedures, must be considered when deriving specifications for length measurements for a particular standard.

Instrumental errors usually are described in the manufacturers' specifications as a number of millimeters or centimeters plus or minus a number of parts per million (ppm). Various tests have indicated that these statements of accuracy are reasonably valid. However, it must be emphasized that these are average values obtained under average conditions, often at a single location, and may not be completely representative of results at varying sites. Nevertheless, these specifications alone, to a large degree, dictate how short a line may be measured with a particular instrument. Of equal or often greater importance in the measurement of long distances are the errors introduced by the inaccuracies of meteorological observations. For short-range measurements, centering the instruments or reflectors and determination of elevations for reduction of slope distances to the horizontal become especially critical.

Three types of EDM are presently in general use: electro-optical devices that use visible light, short-range equipment that employs infrared as the carrier wave, and those that utilize microwaves. The electro-optical instruments, especially those equipped with a laser light source, and microwave equipment, can measure distances of 100 km and more, but those using infrared have limited range, generally 2 km or less.

Several infrared instruments are available; some have direct readouts, others indirect. These instruments require a series of operations to obtain the slope distance. The stated accuracies of these instruments vary between 2 and 10 mm and most include 1 or more parts per million of the length in the statement. For crustal movement studies and for establishing calibration base lines, only those instruments whose stated accuracy is ±5 mm or less should be employed. To establish calibration base lines, two or more instruments should be employed and the measurements should be made in a variety of combinations over at least a 2-day period.

A complete measurement with direct readout electro-optical and infrared equipment consists of 10 readings. Some instruments are equipped with a built-in computer to correct the measurements for atmospheric conditions. The corrections should be checked periodically to ensure that the computer is not malfunctioning. Furthermore, experience has shown that more accurate and reliable results may be obtained by the usual reductions by formula.

One version of microwave distance measuring equipment can be used under most conditions for short distances (0 to 2000 m), with an accuracy compatible to most infrared devices, and for long distances by simply changing antenna horns. This instrument measures in one direction only, but does require an operator at both ends of the line. When used in the short-range mode, we suggest that the measurements be made from both ends of the line with a spread not to exceed 20 mm after the measurements have been corrected for meteorological conditions. When this instrument is used for short-range measurements, care must be taken to ensure that various forms of interference which adversely effect microwaves, such as radio transmitters, high voltage power lines, and the like, are not in the survey area or can be avoided.

One high-accuracy, short-range instrument has an xenon gas lamp emission source. This instrument has a range of 3000 m; its stated accuracy is about 1 ppm of the distance measured. Where optimum results are required, such as for special crustal movement studies, dam deformation studies, and high precision base line measurements, the premium cost for this instrument can be justified.

The accuracy of electro-optical instruments is considered superior to microwave equipment primarily because the effect of humidity on electro-optically measured lines is negligible. Humidity affects microwave measurements under some conditions by 70 ppm or more. Furthermore, an error of 1°C between wet and dry bulb temperatures obtained under normal conditions can produce errors of 10 to 15 ppm in distances measured using microwaves. For distances less than 2 km, the infrared devices usually are superior in accuracy and are little affected by humidity.

Excluding the effect of humidity, errors of 1°C in the air temperature and 3 mm in barometric pressure (about 30 m in an altimeter) produce inaccuracies of 1 ppm for most electronic distance measuring instruments presently in use. We highly recommend that meteorological data be obtained at both ends of the lines for distances measured in connection with all permanently monumented horizontal control surveys. Experience has shown that when the measurements are to be made at tripod height, temperatures taken about 10 m above ground level are generally more representative of the conditions along the ray path. However, when the line barely clears the ground, temperatures obtained at instrument height would perhaps be best. Meteorological equipment should always be shaded from the direct rays of the sun. Furthermore, when the highest accuracy is required, the instruments should be carefully protected from the elements (e.g., by using observing tents).

Number of Length Measurements

The number of length measurements commensurate with the various orders and classes of surveys are tabulated below, together with the recommended instrumentation and other specifications for each.

Commensurate Length Measurements

	First-Order	Second-Order		Third-Order	
		Class I	*Class II*	*Class I*	*Class II*
Base lines	A, B	B	C	D	E
Trilateration	B	C	D	E	F
Traverse	D	E	F	G	G

A. First-Order National Horizontal Control Network base lines (1:1,000,000)

Spacing: 6 to 12 figures (12 to 25 triangles) depending on geometric strength and configuration of segment of network. Generally located at junctions of intersection of arcs.

Instrument: *Electro-optical*

Complete measurement: Four observations—two concentric, two offset ± 0.4 (occasionally shorter offset bar may be employed).

Spread between observations: Not to exceed 40 mm.

No. of complete measurements: Two, at least two days/nights, employing different instruments.

Tolerances: Difference between mean of two complete measurements not to exceed 25 mm for lines less than 8 km and 17 mm plus 1 ppm of distance for lengths in excess of 8 km

B. Regular First-Order base line and First-Order trilateration (1:1,000,000).

Second Order, Class I base lines (1:900,000).

Base line spacing: As required to maintain the scale integrity of the framework, particularly in metropolitan areas usually adhering to R_1 criteria.

Instruments: *Electro-optical* recommended. *Microwave* only when electro-optical not available. *Infrared* for special projects.

Complete measurement: *Electro-optical*—same as specified for "A". Distances not less than 0.4 km.

Microwave—a full set of fine and coarse readings as recommended by manufacturer. For instruments that measure from both ends, the observations at one end should be completed prior to beginning observations at other end. Observations to be made in opposite direction when remote unit serves as slave only. Distances not less than 3 km.

Infrared—Indirect readout, three observations, 1 concentric and 2 offset ± 0.2 m to ± 0.4 m. Direct readout—10 readings for each measurement—3 observations, 1 concentric and 2 offset ± 0.2 m to ± 0.4 m.

Spread between observations: *Electro-optical* same as specified for "A".

Microwave — difference between observations made at both ends of line not to exceed 0.1 m after applying meteorological corrections.

Infrared — Spread between measurements seldom to exceed 7 mm and never to exceed 10 mm.

No. of complete measurements: *Electro-optical—one.*

Microwave—three on different days or nights or separated by at least 4 hours between each measurement.

Infrared—one.

Tolerances: *Microwave* — spread between

three complete measurements not to exceed 0.1 m, after correcting for meteorological conditions.

C. Second-Order, Class I trilateration (1:750,000)
Second-Order, Class II base lines (1:800,000)
Base line spacing: As required to meet R_1 specification.
Instruments: *Electro-optical* recommended for Second-Order, Class I trilateration. *Microwave* only for these survey classes when electro-optical not available. *Infrared* for special projects.
Complete measurement: *Electro-optical*—three observations one concentric, two offset ± 4 m. (Occasionally a shorter offset bar may be employed.) Distances seldom less than 0.4 km.
Microwave—same as specified for "B". Distances not less than 3 km.
Infrared—same as specified for "B", except two observations, one concentric and one offset ± 0.2 m to ± 0.4 m will suffice.
Spread between observations: *Electro-optical*—not to exceed 50 mm.
Microwave—same as specified for "B".
Infrared—not to exceed 10 mm.
No. of complete measurements: *Electro-optical*—one.
Microwave—same as specified for "B".
Infrared—one.
Tolerance: *Microwave*—same as specified for "B".

D. First-Order traverse (1:600,000)
Second-Order, Class II trilateration (1:450,000)
Third-Order, Class I base lines (1:500,000)
Base line spacing: As required to meet R_1 specifications.
Instruments: *Electro-optical*—for First-Order traverse recommended.
Microwave—may be used for First-Order traverse when electro-optical not available.
Electro-optical, Microwave, and Infrared—for other accuracy classes.
Complete measurement: *Electro-optical*—for First-Order traverse same as specified for "C". For other accuracy classes, two observations, one concentric and one offset ± 0.4 m. (Occasionally a shorter offset bar may be used.) Distances seldom less than 0.4 km.
Microwave—same as specified for "B". Distances seldom less than 3 km.
Infrared—same as specified for "C".
Spread between observations—*Electro-optical*—for First-Order traverse same as specified for "C". For other accuracy classes—60 mm.
Microwave—same as specified for "B".
Infrared—seldom to exceed 10 mm and never to exceed 15 mm.
No. of complete measurements: *Electro-optical*—one.
Microwave—two.
Infrared—one.
Tolerance: *Microwave*—difference between complete measurements not to exceed 0.1 m after correcting for atmospheric conditions.

E. Second-Order, Class I traverse (1:300,000)
Third-Order, Class I trilateration (1:250,000)
Third-Order, Class II base lines (1:250,000)
Base line spacing: As required to meet R_1 specifications.
Instruments: *Electro-optical, Microwave* and *Infrared.*
Complete measurement: *Electro-optical*—same as specified for other accuracy classes (D). Distances seldom less than 0.4 km for Second-Order, Class I traverse and seldom less than 0.3 km for other accuracy classes shown.
Microwave—same as specified for "B". Distances seldom less than 3 km for Second-Order, Class I traverse or 2 km for other accuracy classes noted.
Infrared—same as specified for "C".
Spread between observations—*Electro-optical* same as specified for other accuracy classes (D).
Microwave—for Second-Order, Class I traverse, same as specified for "B". For other accuracy classes difference between observations made at both ends of a line not to exceed 0.15 m after correcting for atmospheric conditions.
Infrared—same as specified for "D".
No. of complete measurements: *Electro-optical*—one.
Microwave—two for Second-Order, Class I traverse. One for Third-Order, Class I trilateration and Third-Order, Class II base lines.

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Infrared—one.

Tolerances: *Microwave* — for Second-Order, Class I traverse differences between two complete measurements not to exceed 0.15 m after correcting for atmospheric conditions.

F. Second-Order, Class II traverse (1:120,000)
Third-Order, Class II trilateration (1:150,000)

Instruments: *Electro-optical, Microwave,* and *Infrared.*

Complete measurement: *Electro-optical*—one concentric. Distances seldom less than 0.3 km.

Microwave — same as specified for "B". Distances seldom less than 2 km.

Infrared—one concentric.

Spread between observations: *Microwave*—same as specified for other accuracy classes (E).

No. of complete measurements: One for all instruments. A check measurement from an offset position (± 0.2 m) should be made for infrared instruments when decimal readings are near zero or in the high nines.

G. Third-Order, Class I traverse (1:60,000)
Third-Order, Class II traverse (1:30,000)

Instruments: *Electro-optical, Microwave,* and *Infrared.*

Complete measurement: Same as specified for "F".

Electro-optical—distances seldom less than 0.3 km.

Microwave—Spread between measurements seldom to exceed 0.2 m after application of meteorological corrections. Distances seldom less than 1 km for Third-Order, Class I and 0.5 for Third-Order, Class II traverse.

No. of complete measurements: Same as specified for "F"

NOTE: The specifications on complete measurements for electro-optical instruments are for those requiring calibration tables. Where those instruments which do not use calibration tables are employed and multiple observations are specified as a complete measurement, the number of observations required may be reduced by one.

Infrared equipment within the range of the instruments for measuring distances may be specified in special projects such as those concerned with crustal movement studies where First-Order accuracy is required. In these instances, a complete measurement will consist of the mean of three observations, one made with the reflector centered over the mark and two observations offset ± 0.4 on line. The spread between the three observations should not exceed 7 mm and should never exceed 10 mm.

Electro-optical and infrared measuring devices are recommended for use in subsidiary surveys made to Second-Order, Class I specifications in high-density population areas. Two complete observations are suggested, one made with the reflector centered over the mark and the second with the reflector offset ± 0.4 m. Since lines in such networks are generally rather short, the spread between the two observations, after reduction to the mark, should not exceed 20 mm. Electro-optical instruments are not recommended where distances to be measured are less than 0.3 km.

Secondary breakdown traverse distances in metropolitan areas accomplished to Second-Order, Class II specifications may be determined using electro-optical and infrared equipment. However, electro-optical instruments are not recommended for lines of 300 m or less. A single observation with either instrument should be sufficient The verification check described for use with infrared devices should be made when conditions warrant. When using infrared instruments, it is recommended that lines be measured from both ends as this procedure will substantialy reduce the possibility of blunders. Microwave equipment is not recommended in these situations except where the distances are in excess of 2 km.

For First-Order and Second-Order, Class I surveys, distances of less than 300 m should be taped using the precision methods outlined in the paragraphs on "Taped Base Lines or Traverses." Consideration should be given to measuring distances of less than 200 m, 100 m, and 50 m involved in Second-Order, Class II and Third-Order, Classes I and II surveys, respectively, using the taping procedures recommended for the particular order of work.

All electronic distance measuring devices should be checked frequently over lines of known distances, and serviced on a regular basis. Instruments should be recalibrated annually, or even better, every 6 months. Frequency checks are recommended every 3 to 4 months. Equipment must be handled with care and protected against the elements at all times.

EDM and related equipment and observing techniques are constantly being improved. Those who

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have a requirement for high accuracy electronic distance measurements are invited to contact the National Geodetic Survey for advice, for results of latest evaluations, and for lists of reference publications.

Taped Base Lines or Traverses

The general availability of EDM equipment has drastically reduced the use of taping procedures for the measurement of base lines or traverse lengths. However, there will probably always remain some requirements for the use of taping procedures for calibrating base lines, short traverse connections, engineering or scientific projects and for low-cost, low-order property surveys. There is little doubt that further development of EDM equipment, particularly those with a capability of measuring shorter distances with high accuracy, will further reduce the requirements for taping. The previous section on EDM measurements discusses those situations where taping should be performed to meet accuracy specifications.

Detailed procedures for taped measurements of high-order base lines and traverses are given in Coast and Geodetic Survey "Manual of Geodetic Triangulation," Special Publication No. 247. Procedures to be used for lower-order surveys will be determined by the accuracy required. As to a minimum, we recommend that stakes or taping bucks and standardized tapes, or those that have been compared with a standard length, be used. For Second-Order, Class II or higher, the tapes should be of low-coefficient nickel steel. Accurate differences of elevation between tape supports are required for the reduction of the measurements to the horizontal plane.

The reduction of the lengths must include corrections for the difference between the field observed temperatures and the temperature used in the standardization or comparison, for the true length of the tape and for the slope of the line, for the support condition, and, in most instances, to the ellipsoid (commonly, the sea level surface).

Astronomic Azimuths

The accumulation of angular errors in horizontal control surveys, and the tendency for a portion of these errors to be somewhat systematic (thus providing the opportunity for the development of a twist in the direction of a survey), makes it necessary to include astronomic azimuth observations at specified intervals. For precise surveys, the astronomic observations must include a determination of longitude which is used in determining the correction for the deflection of the vertical. This value is then used to compute the Laplace correction which converts the astronomic azimuth to a geodetic azimuth. Values for the deflection of the vertical can be interpolated from previously observed determinations for many areas of the country. These interpolations are considered sufficiently accurate except at some anomalous locations, for the correction of azimuth observations used to control Second-Order, Class II, and some higher order traverses. Department of the Army Technical Manual TM 5-442 "Precise Astronomic Surveys" and Coast and Geodetic Survey "Manual of Geodetic Astronomy," Special Publication No. 237, contain discussions of the Laplace correction and methods of determining azimuth.

Reconnaissance, Site Selection, and Property Ownership

Reconnaissance for horizontal control surveys should be performed well in advance of the arrival of the observing party. Instructions for the reconnaissance should define the area to be covered, the survey accuracy specified for the project, the desirable spacing of control points, connections to existing surveys, and any special requirements for station locations including, on occasion, those sites where particular base lines and astronomic observations are to be made. Information should be included on any contracts, negotiations, or preliminary planning involving other organizations.

As a general rule, reconnaissance personnel are responsible for satisfying all requirements for strength of figure, line of sight, and other technical specifications. They must secure permission of property owners or managers and negotiate any special agreements that may be required.

An office review of the proposed survey should then be made to ascertain that all requirements have been satisfied and to select sites where base lines are to be measured and astronomic observations are to be obtained. On those ocasions where the survey involves long packs or helicopter support to reach the station sites, simulated studies should be undertaken to assure that the desired standard will be secured most economically without sacrificing the specified accuracies. This approach is also recommended where the configurations may be geometrically weak or of minimum strength. However, in most cases, where the primary doubts are usually related to the

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sufficiency of scale or orientation of a survey, it is often simpler to specify that additional lines be measured or astronomic azimuths observed rather than to perform the computational studies.

Instructions for vertical control surveys usually provide adequate information so that the detailed reconnaissance can be performed by personnel assigned to the observing party working ahead of the surveying units. Site location, property ownership, logistical, and safety problems are resolved by the observing party.

Connections to Existing Surveys

When connecting First- or Second-Order, Class I surveys to established control, new measurements of previously observed angles must be made to check on the recovery of these stations. If the new observations fail to check the previously observed values by more than 2 seconds, the observations should be repeated and all field conditions that might cause such changes investigated. The National Geodetic Survey should be notified of any such difficulties, as it may be able to furnish justification for acceptance of check angles exceeding this maximum. The length of at least one previously observed line should be measured near the initial and terminal stations to furnish additional scale for the existing triangulation or as a further check on the recovery of the station. An integral part of the recovery of a station involves the reobservation and remeasurement of the directions and distances to the reference marks. The directions to the azimuth mark and objects such as water tanks, church spires, etc., which had been previous observed should be included in the reobservation program. When significant differences are found, the newly obtained values should be verified in the field and the National Geodetic Survey notified. The value for a significant difference may vary under certain circumstances. In a general sense, whenever the directions or distances to the reference marks cannot be checked within 3 minutes and 1 cm, respectively, and the directions to the azimuth marks and objects fail to agree to within 10 seconds, specific mention of these discrepancies should be noted in the recovery notes.

For Second-Order, Class II or lower-order surveys, it would be desirable to follow the above specifications. Costs or time factors may make this prohibitive; therefore, as a minimum, distances and directions to reference marks should be verified. If other stations or intersection points are visible, observations should be made on these points and compared with previously observed or computed values. It is much better to orient a survey through azimuths to other distant stations that were previously observed as part of the existing survey than to use a nearby azimuth mark.

New surveys should normally begin and terminate at control points of an accuracy equal to or higher than that required for the new network. This does not preclude connecting higher- to lower-order surveys in special cases. For example, in a scientific or engineering project, where the geographic positions are relatively unimportant, a high internal accuracy may be required. Whenever practical and economical to do so, all control surveying activities are to contribute to the National Geodetic Control Networks. In this case, prescribed connections should be made to the lower-order surveys, the published positions of one previously established point held, and a minimum constrained adjustment of the new data performed. On occasion, the new observations may be used to strengthen the lower-order surveys in the vicinity through a readjustment of the old observations. New observations may also be retained for future use.

Spacing of Control Points

Horizontal control points may be spaced at any required interval; such spacing is restricted only by the limitations of available instrumentation and technical procedures.

In triangulation and trilateration, the lengths of lines are usually governed by the requirements of the project instructions, by topographic conditions, and by strength-of-figure considerations. In general, National Network First-Order stations are seldom spaced less than 15 km apart; Second-Order, Class I, 10 km; and Second-Order, Class II supplemental control points, 5 km. Spacing in survey projects in areas of intensive economic development may be much closer, as indicated in the table of specifications. Project instructions issued by Federal agencies usually require that points be located at airports, colleges, towns, along waterways and principal highways, and that whenever practical, require that connections be made to horizontal and vertical monuments established by other surveying and mapping agencies. These may be main scheme stations, providing the strength of figure can be maintained, and, in the case of existing marks, the monumentation is satisfactory. Otherwise, new control should be established.

When selecting locations for horizontal control points, consideration should always be given to present or possible future control extension requirements. To this end, prominent manmade or natural objects should also be positioned by Third-Order intersection methods as a part of the observing schedule.

As a general rule, the spacing of traverse stations is restricted only by the minimum length-of-line specification. However, traverses per se have little or no geometric strength such as that inherent in triangulation in particular and to a lesser degree in trilateration. Weakening factors in the design of traverse systems must be held to a minimum. Specificaly, traverses, whether of a single line variety or interconnected networks, should be designed so that the routes between control stations are more or less straight lines with the distances between new stations (courses) of about equal length. Loop systems should be essentially square or rectangular. For First- and Second-Order, Class I traverses, any deviations greater than 20° from a straight line should be avoided; it is especially important that the courses be of equal length. In those cases where topography or other conditions make it necessary to introduce abrupt changes in the routes or shortened or expanded course lengths, additional astronomic azimuths in the vicinity of the abrupt changes, reobservations of the angles at these sites, or remeasurements of the distances on other occasions should be considered. These specifications for higher accuracy traverse networks are required to ensure that the components involved in position closures are directly related to the observational element involved in the particular component. For example, the closure in latitude in an east-west traverse should be directly related to the angulation or orientation, while the closure in longitude should be length related. Whenever abrupt changes or short courses are interspersed with longer lines, it is extremely difficult to evaluate a traverse network; for this reason, strict requirements must be specified.

The restrictions also apply to Second-Order, Class II traverses where the stations are spaced to National Network specifications and the purpose of the surveys is to supplement the National Network. However, some leeway is permitted for this survey class in areas where the topography requires the use of equipment not generally available or development is unlikely in the near future.

Where Second-Order, Class I and II standards are desired for metropolitan area subsidiary or breakdown surveys that involve station spacing at intervals of 200 to 2000 m, it would be best if the design patterns follow those just described. However, this will not always be possible, so modifications can often be instituted justifiably. The major problems encountered in traverses involving short lines are those related to the accuracy of available EDM equipment and orientation. Problems involving very short distance measurements can be resolved by employing taping procedures. Problems concerned with orientation can be reduced by instrumentation (e.g., the use of "forced centering targets") and the insertion of astronomic azimuths at more frequent intervals or the reobservation of the angles on at least one other occasion.

When closely spaced stations are included to meet project requirements rather than because of topographic limitations, it is often possible to measure between more widely separated stations to maintain overall accuracy and to locate the closer spaced points by closed loop traverses that include these stations.

Where somewhat parallel traverses or surveys established by other methods approach each other, they should be connected if at all practical. The "20% rule" should be enforced in these situations.

The 20% rule states that "whenever the distance between two unconnected points is 20% (or less) of the sum of the distances between directly connected stations involved in the shortest route between the two points, a connection between the points should be made where possible and practical." Ideally, multiple traverses in an area should be interconnected to form a somewhat rectangular pattern, and all should be included in a single adjustment.

Strength of Figure

The strength-of-figure determination is an expression of the comparative precision of computed lengths in a triangulation net as determined by the size of the angles, the number of conditions to be satisfied, the distribution of base lines, and lengths determined in previous adjustments. Strength of figure in triangulation is not based on an absolute scale but rather is an expression of relative strength.

Although the strength-of-figure concept remains an important consideration in performing triangulation reconnaissance, the reliance once placed on its use in determining the need for base lines has been relegated to a less prominent role. With the increasing use of electronic distance measuring equip-

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ment and, hence, the ability to measure distances almost anywhere, this essentialy economic reliance has been almost totally eliminated. Furthermore, modern survey networks are becoming a mixture of triangulation, trilateration, and traverse; other evaluation measures, much more complex than the simple strength-of-figure test, are required to ascertain the acceptability of the design. Nevertheless, the strength-of-figure tolerances are still an integral part of the specifications for triangulation, which for the immediate future continues to be the means for establishing primary National Network control.

Elevation of Horizontal Control Points

Whenever distance measurements are made in the course of performing control surveys to provide scale, or, as is the case in trilateration and traverse, a fundamental element in the positioning of points by these methods, some means are required for reducing the measurements to the horizontal and to the reference ellipsoid (commonly, the sea level surface). For First- and Second-Order, Class I surveys, the use of elevations is the preferred method. Although accurate elevations are desirable, the accuracy of the differences of elevation is of greater concern, especially where these differences are responsible for slope corrections in excess of 0.1 m. The reductions are computed using the elevations of the points at which the measurements were made plus the accurately measured heights above the stations of the instruments or reflectors.

It is preferable that elevations be determined by spirit leveling methods, either directly, that is, the stations also serve as bench marks in a leveling line, or by connections to bench marks. However, because of the locations of most horizontal control stations, this will seldom be the case, so most of the elevations for these points are determined by trigonometric leveling using zenith distance observations, a form of vertical angles, in the computations.

Although the use of elevations in reducing measurements involved in Second-Order, Class II and lower-accuracy surveys is highly recommended, particularly if trilateration is employed, vertical angles may be utilized to make the reduction to the horizontal. Several words of caution must be offered when the cosine of the vertical angle is used to reduce slope measurements to the horizontal. If the observations are made at one end of the line only, the correction for curvature and refraction should be applied as a general practice, whether or not it significantly affects the cosine. This correction is not necessary when the observations are made at both ends of the line and a mean vertical angle is used. In addition, any appreciable difference (and this can be as little as 1 foot) in the heights at which the vertical angles and the length measurements were secured must be taken into account as another correction to the observed vertical angle.

Table 2 of "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys" provides a limitation on the number of figures over which trigonometric leveling can be carried before a check on the elevations is to be obtained. This check may be made against previously determined trigonometric elevations, using similar specifications, or elevations may be obtained by spirit leveling. For those projects where the elevations are required only to reduce isolated base lines, and making connections to points of known elevation would involve a considerable effort, the elevation of one of the base line stations may be scaled from a good quality map or, in some cases, determined by a well-adjusted barometer or altimeter The difference of elevation between the stations at each base line site could then be determined by spirit leveling or reciprocal vertical angles.

Elevations are also required to reduce horizontal directions observed at the higher elevations and mountainous regions and to reduce astronomic observations to the reference surface.

The observational and computational procedures for vertical angle measurements in triangulation are given in Coast and Geodetic Survey "Manual of Geodetic Triangulation," Special Publication No. 247, pages 15, 103, 127, 139, 143, and 184 and "Elevations from Zenith Distances," Publication G-56. Although procedures for traverse are essentially the same, special emphasis must be placed on observations over the short or steep lines more common to these surveys and extra care must be exercised because of the lack of redundant observations.

The number of vertical angle observations required is indicated under *Vertical Angle Observations*, in table 2 of "Classification, Standards of Accuracy and General Specifications of Geodetic Control Surveys." One observation consists of one direct and one reverse pointing. If the permissible spread between the observations in a set is exceeded, the observations will be repeated until a satisfactory set is obtained. The heights of the instrument and target reflectors above the marks must be accurately measured. Special care must be given where the differ-

ence in elevation divided by the distance exceeds 0.03. When this ratio approaches 0.1 and the distances are less than 300 m, consideration should be given to leveling between the points. Refraction is a major consideration in obtaining accurate vertical angle observations, and observations should be made when atmospheric conditions causing vertical refraction are the most stable. In most areas the best time period for vertical angle observations will be between 11 a.m. and 4 p.m.

Other Sources of Error

The specifications presented herein include requirements for instrumental accuracy, the number of observations necessary, and the several computational closure requirements that must be satisfied to give reasonable assurance that the stated standard will be met. In some sections, additional precautions are mentioned. If these precautions are not taken, this lack of action may prevent the successful completion of a project even though all major requirements have been met. The following are presented as additional precautionary suggestions rather than as a part of the formal specifications.

Collimation and Eccentricity

These are related problems that affect the accuracy of both angle and distance measurements, and may be generally categorized as *centering* of instruments and targets. The effect of erroneous centering on distance measurements is direct and rather obvious. For angulation, however, the importance (especially on short lines) must not be overlooked. With reasonable care, angles can be measured to a precision of less than 1″.0 over lines of any length and an accuracy (3δ) approaching this value for lines in excess of 400 m. With special care, in controlled situations, angles accurate (3δ) to 0″.5 and less are possible. This is meaningless if the instrument or targets are not centered over the marks and sufficient measurements are not obtained to reduce the observations to the true points.

As a general rule, instruments and targets can be positioned within about 1 mm of the true center. An instrument or target may be deliberately or accidentally operated or observed in an eccentric position, usually a rather short distance. Accurate measurements of the angle and distance involving the eccentric and true points must be obtained to reduce the observations to the true point or to maintain the relationship of the points. The following examples give some indication of the magnitude of angular errors caused by small miscenterings. A 3-mm error in centering a target will cause a 0″.4 error at 1500 m and 4″ at 150 m. For traverse points spaced at 150 m, if both targets and the instruments are miscentered by 3 mm, the angle error could exceed 16″. The following formula can be used to determine the maximum error resulting from miscentering a signal by 1 mm: $E'' = 206/S$, where S is the distance in meters.

Stability and Rigidity of Supports

In precise surveys, towers, stands, and tripods must be substantial. The use of driven stakes or some type of quick setting cement for tripod leg support may be required. Catwalks, supported away from the tripod legs, may also be necessary under some soil conditions to ensure satisfactory results. Regular instrument tripods should rarely, if ever be used in First- or Second-Order, Class I Surveys.

Phase

Phase error in horizontal angle measurement is attributable to apparent displacement of a target. Such displacements can be caused by unequal daylight illumination of a target or by lighted targets not pointed directly toward the observer. For precise surveys, the use of carefully pointed illuminated targets is recommended for both day and night operations. The error effect of phase is the same as described for centering errors except that it is often not readily apparent and, even when detected, is not easily corrected mathematically.

Refraction

A ray of light bends as it passes obliquely through strata of air with different densities. Most of this kind of refraction is in the vertical plane, but frequently there is a measurable component in the horizontal plane. This type of refraction is one of the most uncertain factors encountered in the measurement of horizontal angles. Night observations have occasionally been found to be in error by 5 or 6 seconds and daylight observations by two or three times that amount. Corrections for refraction cannot be applied to observations, but a careful reconnaissance to avoid suspect topographic features and the selection of the best available observing conditions will usually eliminate or substantialy reduce the effect of this condition. Horizontal refraction is discussed in Coast and Geodetic Survey "Manual of Geodetic Triangulation," Special Publication No. 247, p. 135.

Shelter

The protection of instruments from uneven thermal expansion and from wind vibration is required for precise surveys. Minimum protection should consist of a large umbrella to block the Sun's rays and a wind screen if needed. A complete tent is recommended for lengthy observing schedules under adverse weather conditions, for the protection of equipment and the comfort of the personnel.

Monumentation and Description

All First- and Second-Order, Class I horizontal control stations must be monumented and described. The monumentation includes, in addition to the station mark, an underground mark where possible, two or more reference marks, and an azimuth mark. Another horizontal control point or, occasionally, natural or manmade objects visible from the ground, may be substituted for an azimuth mark. Although observations to all or to a representative grouping of manmade or natural objects visible from tripod height at a station should be included with the station description, this does not negate the necessity for establishing a monumented azimuth mark. Many of these objects are subject to repairs or rebuilding in more-or-less the same location, displacements caused by aging, and, in the case of radio masts and other guyed structures, movement caused by replacing or adding guys or sections. For these reasons, such objects should seldom be used in place of a monumented azimuth mark. However, where positive identification can be made, observations to substantial distant natural or manmade objects whose azimuths are known or computable are preferable to those to nearby azimuth marks for orienting surveys.

These specifications are applicable to stations established to Second-Order, Class II standards, but as noted previously, an occasional point marked in a temporary fashion, particularly in traverses, is permissible in some circumstances. These occasions should be few since it is neither economical nor good practice to obtain quality results without providing proper monumentation and descriptions for future use.

Third-Order monumented stations should be located in protected areas, when at all possible, and should be described with sufficient care so that unmistakable recovery is probable in the future. Manmade and natural objects determined to Third-Order accuracy by intersection or traverse connection methods should be described with adequate care and detail to give reasonable future assurance that the points may be positively identified.

When constructing marks, take into account the problems related to settlement in soft soils, possible movement caused by freezing and thawing, likely long-term erosion and wave action, and other natural or cultural effects.

A witness post or a sign should be placed near each monument where local conditions permit as an aid in recovery and as a protection against accidental destruction.

The station description should include information on how to reach the general location of the station site from a town or some prominent feature normally shown on road or other maps, and a detailed description of each monument in relation to nearby topographic features. Distances and directions to the reference marks relative to the station mark should be included.

These observations and measurements should be determined with sufficient accuracy to aid in the recovery of the underground mark should the surface mark be destroyed, to replace the station in its original location, to estabilsh a new station nearby, and, where necessary, to permit using a reference mark as a substitute station.

Recommended methods of constructing monuments, and marking and designating stations are included in Coast and Geodetic Survey "Manual of Geodetic Triangulation," Special Publication No. 247 and "Specifications for Horizontal Marks," *ESSA Technical Memorandum* C&GSTM-4.

Vertical Control

Elevations of bench marks of the National Geodetic Control Networks are currently published as linear distances ("orthometric heights," Coast and Geodetic Survey "Manual of Leveling Computation and Adjustment," Special Publication No. 240, p. 155) above or below the "National Geodetic Vertical Datum of 1929" (Federal Register, May 16, 1973, Vol. 38, No. 94, p. 12840). Lists published prior to September 1973 use the designation "Sea Level Datum of 1929." This datum was established by constraining the combined interconnected U.S. and Canadian networks of First-Order leveling, as it existed in 1929, to conform to mean sea level of various epochs, as determined at 26 long-term tidal stations distributed along the Atlantic, Gulf of Mexico, and Pacific coasts (21 in the United States and 5 in Canada). Spirit level instrumental observed differences of elevation, start-

ing at the various local mean sea levels, as defined at the 26 tidal stations held fixed, were interconnected into a network consisting of approximately 103,000 km of First-Order level lines. These observations, after orthometric corrections, corrections for other definable systematic errors, and simultaneous adjustments for loop misclosures, provided a set of orthometric elevations for all bench marks in the adjusted system (in 1929). This operation is referred to as the "General Adjustment of 1929," and the resulting elevations defined the datum to which (with a few exceptions) all subsequent geodetic leveling in the United States has been adjusted. By 1974 a total of approximately 350,000 km of First-Order lines had been adjusted to this datum.

The present vertical control network consists of a hierarchy of interrelated nets throughout the Nation, all of which have been adjusted to compatibility with the elevations of the bench marks defined by the General Adjustment of 1929. The primary network in this hierarchy comprises two interdependent networks: (1) a basic set of large circuits at a spacing of 100 to 300 km, and (2) a subsidiary primary subdivisional set at spacing of 50 to 100 km. A secondary densification network with line spacing of 10 to 50 km provides additional vertical control points to slightly lower accuracy standards. These primary and secondary networks together establish a reference base for all other leveling to meet user needs for compatible elevations referred to a common nationwide datum—the National Vertical Datum of 1929.

First-Order (Primary Network)

The basic elements of the primary National Network consist of a set of bench marks spaced about 2 km apart along the lines of leveling except in areas of intense development where the spacing is reduced to 1 km with line spacing as required. The basic set is observed with the requirement that double-run leveling between successive bench marks agree within 3 mm $\sqrt{K}$; the subsidiary subdivisional set is double-run leveling between succesive bench marks that agree within 4 mm $\sqrt{K}$ (K is the section length in kilometers). For the purpose of section agreements, only those sections originating and terminating with permanent bench marks will be considered. Sections less than 1 km long should be avoided in developing the basic network. However, when use of a short section is unavoidable, it is recommended that misclosures be combined with adjacent sections for check purposes so that the check will be for a kilometer or more. Also included in the First-Order classification is any leveling run, under special instructions, to accuracy higher than Class I.

Second-Order, Class I (Secondary Network)*

This class consists of bench marks established 2 km apart along the lines of leveling spaced 20 to 50 km apart. Densification of vertical control in intensely developed areas should also be established by this standard.

Second-Order, Class II (General Area Coverage)*

This class consists of control densification to provide elevations of bench marks on lines spaced 10 to 25 km apart throughout the Nation. This leveling is usually single run except for spur lines to new marks and long lines. Vertical control for local engineering projects and topographic mapping should be observed to specification for the standards of this class.

Third-Order (Local Vertical Control)*

This order is established to met local needs by specifications that provide for an agreement between National Network bench marks or internal loops of 12 mm $\sqrt{K}$. Spacing of bench marks along these lines of leveling should not be greater than 3 km.

Closure Requirements (Allowable Misclosures)

Allowable misclosure limits are given in table 3 of "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys." These misclosures are listed for section (forward and backward), loop, and line. For sections less than 0.25 km in length and for sections consisting of a single instrument setup, the following apply:

First-Order,	Class I	1.5 mm
First-Order,	Class II	2.0 mm
Second-Order,	Class I	3.0 mm
Second-Order,	Class II	4.0 mm

Single setup, plane-parellel plate micrometer 0.5 mm

Single setup, three-wire 1.0 mm

Sections should be 1 km or longer on First-Order lines, but because of the location of bench

* In certain areas where the network lines are far apart, mapping organizations may need to extend lines of Second- or Third-Order leveling over longer than recommended distances. In all such cases, the lines should be double-run, and for lines longer than 50 km, Second-Order, Class I is recommended. Where ties are made to existing single-run, Second-Order, Class II lines, misclosures up to 8 mm $\sqrt{K}$ can be expected.

marks in place along established lines, some short sections may be necessary.

Each double run section of a line of levels must contain a forward running and a backward running that agree with specified limits. When multiple runnings of a section are made, measurements that fall outside the indiscriminate mean by the following limits should be rejected.

First-Order,	Class I	$4\sqrt{K}$ mm
First-Order,	Class II	$6\sqrt{K}$ mm
Second-Order,	Class I	$8\sqrt{K}$ mm
Second-Order,	Class II	$9\sqrt{K}$ mm

A running suspected of containing an error because of uncertainty about a procedure, or a running seen by inspection to be far outside the average difference, should be rejected before determining the indiscriminant mean.

On First-Order, Class I leveling, use only sets of forward and backward runs unless the refraction correction is computed for each running.

The mean difference of unrejected multiple runnings for a section is determined by meaning the mean of the forward runnings with the mean of the backward runnings.

Generally, it is considered efficient to have the leveling operations conducted with such care that sections requiring additional runnings to obtain specified checks between forward and backward runnings be less than 5%. When the amount of rerunning exceeds 5%, improvements in field procedures should be investigated. Exceptions to this would be the instances where desert terrain or marshy ground are involved and where observing conditions are unavoidably difficult.

Reconnaissance—Obtaining Permission To Survey in Special Areas

The plans for new leveling should include the proposed route plotted on maps. The maps available to those preparing the instructions for leveling may not show all the existing field conditions. Consequently, as soon as posible, and preferably before the party arrives in the locality of a proposed survey, a reconnaissance should be made to determine if the most feasible routes have been proposed. If alternate routes for some of the lines would better develop the net or would facilitate the field work, these routes should be followed unless prohibited by the requirements. In considering alternate routes, the proper spacing of lines must be maintained within reasonable limits.

Permission will be obtained locally to operate along railroad rights-of-way, private roads, and turnpikes. This should be done well before actual field work commences along such routes. When leveling along busy highways, the chief of party should inform the State highway patrol of his plans and safety precautions to be taken.

Connections to Existing Surveys

The descriptions of all bench marks established by all organizations along the routes of the proposed leveling should be obtained before new bench marks are set. On a new leveling line, at least two bench marks and preferably three or more, which show no change in their relative elevations, must be connected at each end of the line and at intermediate junctions with previous leveling. These connections must check the elevation difference of the established bench marks within the prescribed limits of the original leveling. Checking the difference of two bench marks located on the same structure or so close together that both may be affected by the same localized disturbing influence is not considered a proper check. Checks should be obtained between two or more bench marks in two directions from the junctions of new and old lines where the old bench marks are widely spaced or where several old bench marks near the junction have been destroyed. This is more reliable than connecting the leveling lines that tie between two widely separated marks. Ties at both ends of the check line must check overall with each other. Usually, a line of leveling should not begin or end on a line of a lower order of accuracy. However, appropriate ties to these lines should be made at intermediate junctions.

Instrumentation

The leveling instrument used for geodetic surveys must be of good quality. For First-Order leveling, an automatic level with a parallel plate optical micrometer as an integral part of the instrument is generally preferred.* The Zeiss Ni-1 is representative of this instrument. However, spirit levels such as the Wild N-III will provide excellent First-Order results. For Second-Order leveling, representative instruments are the Zeiss Ni-2, the Wild NA-2 (automatic), and the Kern NK-3 (spirit). The properties of a geodetic level are stability, good

* Other systems have been developed for reading the rods exactly. When proven commensurate in accuracy to the parallel plate, optical micrometer, these systems will be acceptable.

optics, components for achieving precise horizontality, precision-made smoothly operating parts, durability, and a convenient arrangement of manipulative parts.

The *level vial* or tube of a spirit level should be of uniform curvature and be sufficiently sensitive to enable horizontality within ¼ to ⅓ second. For instruments having the bubble centered by optical coincidence and a bubble image magnification of 2 or 3 power, a sensitivity of 8 to 10 seconds per 2 mm is adequate. The level tube should be built into the instrument so that it will be shielded from direct sunlight.

In First-Order and other leveling, when instruments with exposed level vials are used, *sunshades* are necessary.

For *automatic levels*, the compensator substitutes for the sensitive spirit tube and the tilting screw. The compensator should have the capacity to achieve horizontality with the same ¼-sec accuracy. The compensator should not be oversensitive to vibrations such as those caused by wind of passing motor vehicles.

The telescope should have a large objective lens to insure brightness and good definition of the image. The eyepiece is usually nonerecting on spirit levels to allow for greater brightness. However, with coated optics now used in surveying instruments, an erecting lens is not objectionable. The magnification power of the telescope may vary from 30 to 50 diameters. The reticule should be designed with the markings sandwiched between glass plates to insure a dustfree image.

Stability

It is essential that the leveling instrument be stable. Stability may be accomplished by having a leveling base that is wide relative to the height of the instrument above the base, and by keeping the area exposed to wind small and streamlined. Since the tripod is a part of the instrument in use, it must be strong, rigid, and heavy enough to insure stability. The legs should be long enough to permit the observer to stand erect. Tripods with telescoping legs should not be used for precise leveling, although some have been successfully used with automatic levels on Second- and Third-Order leveling. The shoes of the tripod should be pointed and tapered to a size large enough to give good stability in sand or soil.

Leveling Rods

First,- Second-, and Third-Order leveling employ precision leveling rods with graduated scales on invar or similar low expansion nickel-steel strips. The scales should be calibrated and standardized by a reputable testing laboratory. The graduations should have an overall accuracy of 0.1 mm. Rods are graduated in either foot, yard, meter, or half-meter units. Rod scales on the invar strips are usually in the form of black and white rectangles or line marks. Rods with rectangular markings are used with levels that require the observer to interpolate the last integer of the reading; and rods with line markings are used with levels having optical micrometers. The numbers identifying the scale divisions are on the rod frame.

Most precise rods have two parallel scales for increasing the accuracy and especially for checking the readings and recording. Levels and rods may be designed by the manufacturer for use as units, but if the scales of the rod and the micrometer match, they can generally be interchanged. The rod frames are made of wood or a lightweight metal. The metal rods are usually lighter and more durable. The invar strips fit into guides or grooves and are fastened to the base with a constant tension applied at the top.

Turning Pins

Rod turning pins are generally prefered to turning plates except on pavement or packed soil. Tests indicate the pins are more stable. Turning plates should not be used on turf during First-Order surveys.

Leveling Procedures

Two rodmen should be used on First- or Second-Order leveling. During the progress of the leveling, the backsight and the foresight are observed alternately on successive setups. This procedure is accomplished if the same rod is observed first at all setups. After the completion of the setup, the rear rodman moves forward to become the forward rodman for the following setup, and the forward rodman remains in place to become the rear rodman.*

The maximum length of sight allowed for levels used as 3-wire instruments under favorable conditions is 70 m. When using instruments with the plane-parallel plate optical micrometer or other similar micrometer systems, the maximum length of

* Another recommended procedure for First-Order, Class I, is (1) backsight, left-scale, (2) foresight, left-scale, (3) foresight, right-scale, and (4) backsight, right-scale. (With Zeiss Ni-002 automatic level, turn mirror 180° after step 2.)

sight allowed is 50 m for First-Order, Class I leveling. For First-Order, Class II and Second-Order, Class I leveling, 60-m sights are permissible. The lengths of sight should be kept within distances that permit clear images of the rod for good interpolations of the wire readings and good settings of the optical micrometer. However, undue caution and very short sights will slow progress without producing a compensating increase in leveling accuracy. Under most favorable weather conditions and topography, the sight lengths should usually approach the maximum.

The lowest permissible reading on the rod is 0.5 m. The line of sight should not come nearer than 0.7 m to a crest between the instrument and rod and not nearer than 0.5 m to the ground at any point.

In balancing sights, the maximum difference between the backsight and foresight for a single instrument station is 5 m for three-wire leveling. For levels with a critical focus (Zeiss Ni-1) and instruments with optical micrometers the maximum unbalance should not exceed 2 m. If possible, unbalanced sight lengths should be corrected by unbalancing the sight lengths in the opposite direction during the following setup. The length of the two unbalanced setups should be equal. The accumulation of the difference in total lengths of backsights and foresights for a setup or within a section is given in table 3 of "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys." When sight lengths are carefully balanced and the unbalance for each section does not exceed 5 m, weekly determinations of the collimation error (level error) "C" should be sufficient. However, "C" must be determined any time there is reason to believe the level has been disturbed. Also, all instruments newly acquired, repaired, or removed from storage should immediately be tested for "C". Every 2 weeks, a test must be made of the adjustment of the *circular* (*plumbing*) *levels* on the rods. Refer to manuals such as U.S. Coast and Geodetic Survey, "Manual of Geodetic Leveling," Special Publication No. 239, and Lake Survey Center Miscellaneous Paper MP 69-4, ". . . Levels of High Precision Using the Zeiss Ni-2 Automatic Level."

Sources of Error

The errors inherent in leveling fall into three general classes: accidental, constant, and systematic. In addition to the inherent errors, mistakes result from carelessness, oversight, or poor judgment. Large mistakes are easily detected; small mistakes usually are detected by repeated observation of loop closures. Small systematic and constant errors must be eliminated so that only accidental (random) errors remain. The errors inherent in leveling include the following:

a. Limitations of the instruments (minimum reading, horizontality, unbalanced sight);
b. nonstandard instruments (rods not matched, scale inaccuracies);
c. lack of stability of level setup, and rod turning points;
d. environment (temperature, refraction, lighting, and ground surface features);
e. limitations of the observer (inability to use the instruments precisely, inability to interpolate);
f. lapses in recording (including transposition); and
g. astronomic effects (luni-solar, etc.).

The graduations of rods may vary from 0.1 mm overall for some types to 0.2 mm or more for others. Even if the rods are matched, an error is introduced on steep slopes when one rod is read low and the other high on the rod scales (minimized when standardization constants are used to compute corrections). Considering rods with scales that are too long by 0.05 mm per meter, the correction for the difference in elevation between two bench marks, in millimeters, would be +0.05 times the difference in elevation in meters. A source of error (probably accidental under the method of leveling prescribed) is the inertia of the liquid in the level vial of spirit levels. The observer should bring the bubble to rest before making the rod readings.

An error may be caused by slightly refocusing the rod image between sights at an instrument station. This may change the relation between the axis of the bubble and the line of sight of the telescope.

Because of the environment, there is usually some systematic error in a single running of a line, the size and sign of which probably depend upon the direction of the running with respect to the sun, the time of day, the movement of the air, or possibly upon all three combined. All types of leveling are affected by the varying vertical refraction on the backsights and foresights. Two runnings of a section on a clear day agree more closely when the wind is strong than when it is very light. When the air is turbulent the temperature is more likely to be uniform throughout the line of sight, and the refraction should be more nearly the same for the back-

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ward and forward sights. The errors that may affect a single running in a systematic manner can be made to act more like accidental errors by leveling the second running under different atmospheric conditions. For instance, if the first running has been made in the early forenoon after the ground has become warm (and refraction is minimal) and the second leveling near noon when the refraction is a maximum, the total error might be lower but forward and backward checks might be poor. The proper way to handle this problem is to level under as nearly similar conditions as possible and apply a refraction correction. Ideally, the section leveled in the early forenoon should be leveled backwards in the late afternoon when the air and ground temperature relationship is similar. Also, the temperature gradient should be observed and the refraction correction applied when large differences of elevation are involved. On cloudy days, the systematic errors due to atmospheric conditions will be small. There probably is a small error in the mean of the two measures of a section where one running is made on a clear day and the other on a cloudy day, unless refraction corrections are applied.

Systematic errors due to the environment are more likely to occur on steep grades if the observing is done when the surface of the ground is cooler than the air. This sets up a wave motion in air of uniform density and errors up to a millimeter can occur in a single sight. The condition is most likely to occur shortly after sunrise and just before sunset.

If the air and the surface of the ground have different temperatures, the layer of air in contact with the ground will assume a temperature nearer that of the ground, and consequently the density of the air changes with distance from the ground surface. This is true for only a few meters above the ground. If the ground is level, there will be no difference in the amount of refraction on the two sights (F & B), but if the ground is sloping, the sight upslope will pass through more layers of air of different densities than the sight downslope. The effect of the refraction on the rod reading on the first sight will be of one sign if the surface of the ground is colder than the air and of the opposite sign if the surface is warmer than the air. These errors are smaller when the wind is blowing and the air is mixed. On cloudy days, the temperature of the air and the surface of the ground are more nearly the same; consequently, the refraction error on slopes will not be as large as on clear days. The refraction error on a clear day will be at a maximum during the several hours near midday. It should be a minimum about 1 hour after sunrise and 1 hour before sunset. During the night, refraction is about half the daytime value and is of the opposite sign. Scintillation is most severe from about 10:30 a.m. to 2:30 p.m., which can make accurate leveling difficult.

As stated above, this refraction error should be nearly absent if the ground is level or nearly so, and the backsights and foresights at instrument stations are equal in length. It should reach a maximum on grades where the line of sight up the slope comes closer to the bottom of the rod. This error may be reduced by never letting the line of sight come near the ground. The minimum reading recommended is 0.5 m.

Because of limitations of the observer, accidental and systematic errors will be present under any method of operation and will increase when leveling progress becomes abnormally rapid. Speed and accuracy must be balanced. Also, special care must be exercised to guard against mistakes. In single-run leveling, unlike double-run leveling, errors are more likely to escape detection, which could necessitate the releveling of a complete line instead of a single section. With the three-wire method, the greatest source of accidental error is estimating the millimeter graduations on the rod.

A mistake in a rod reading is most likely to occur at a decided change in grade or when changing at a bench mark from the forward to the backward running or the reverse. However, to prevent transpositions or misreadings, a single setup may not be used on both a forward and backward running of a section. The instrument height should be changed by several centimeters. While leveling on a long, even grade, both the observer and recorder should be on the alert for a change in grade because the reading for the foresight, and possibly also for the backsight, may fall on a different meter space on the rod from where it had fallen for a number of previous stations. A mistake of a whole meter, or a half meter with half-centimeter rods, is the one most likely to occur at the change in grade. The backs of the older type rods are graduated in feet. By reading the backs of the rods at each station on at least one running, the probability of an undetected mistake of a meter or decimeter is greatly reduced. The kind of graduation used for the half-centimeter rod and the double scale should likewise prevent a mistake.

The transposition of backsight and foresight readings will probably remain undetected except rarely

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in double-run leveling. On single-run work, prevention of such transpositions is imperative. Under no circumstances should any section of a single-run line in which a transposition is even remotely suspected be allowed to stand without another running over the section, preferably in the opposite direction. On double-run leveling, a running that has been corrected for a transposition may be used, but it alone will not serve as a check. At least two runnings of the section must be free of transpositions that were not immediately discovered and corrected before the instrument was moved.

Monumentation and Description

Permanent bench marks should be established at 1- to 2-km intervals. The average distance between bench marks along a line should be 1.5 km. When transportation of materials is particularly difficult and there are no rock outcrops or suitable structures in which to set marks, the 4-ft (or longer) copper rod type of bench mark with base plate, or pipe mark with base plate, is economical to use. Much of the leveling done today is releveling old lines where the existing bench marks are mostly concrete posts. In such cases, set new bench marks of the short rod or pipe type with base plate. In determining the density, all bench marks of other organizations adequately set and described may be used in complying with the above requirements. Other types of marks, such as chiseled squares, bolts, rivets, unmarked points, etc., are not to be considered in the spacing requirements. In areas for study of rapid earth movement, it is good practice to space the bench marks ¾ km or less along each line. Since the movement of the bench marks should be a measurement of earth movement, a sufficient number of marks should be set to define the areas of movement and to insure the continuation of the records.

At locations where junctions exist or are planned, three special bench marks should be established within a radius of 1 km and spaced so that not all will be affected by a local disturbance. Bench marks set in bedrock and deep marks set in a stable earth stratum or solid rock should be used for First-Order junctions. Tie leveling to bedrock bench marks located up to 8 km from a line junction is highly recommended. Bench marks set in stable buildings and structures, or copper rods driven to refusal or substantial resistance, may be adequate for Second-Order junctions.

Except in unusual cases, the permanent bench marks should be established several weeks before the first leveling of the line to allow time for initial settling to take place. If it is necessary to set new marks during the leveling, those set in buildings and other structures not likely to settle significantly are best. Of course, bench marks may be set in rock outcrops or large embedded boulders at any time. Deep marks founded in bedrock or other stable strata should be stable almost immediately. Every new permanent bench mark must be marked by a metal tablet, secured in rock, brick, stone, concrete, or to the top of a metal rod or pipe. Exceptions are certain special deep marks where the identification may be cast in a concrete or metal cover.

When bench marks are emplaced, a witness sign should be set nearby. These signs will aid in future recoveries and may prevent unnecessary destruction of the mark during construction work. Omit the witness sign if it would be unsightly or a hazard to traffic.

When a permanent bench mark is established or brought into the leveling net for the first time, a description report should be prepared. Descriptions should be complete to locate the site and the bench mark and should include the distance and direction from the witness sign. A sufficient number of reference points should be included in the description to allow for changes at the site.

Gravity in Vertical Control Surveys

The equipotential surface, which is everywhere normal to gravity and has the same geopotential as the average at sea level over the world oceans, is known as the geoid. As the Earth is not homogeneous the geoid is an undulating surface. For computing gravity anomalies, an ellipsoid that best fits the geoidal surface is used. With adequately spaced gravity stations, anomalies can be used to compute gravity at any bench mark where actual gravity was not observed.

As the level bubble, or compensator of automatic levels, indicates the local direction of gravity, the surface of reference in leveling is the geoid and not the geodetic ellipsoid. Gravity is the resultant of gravitational attraction and the centrifugal force caused by the rotation of the Earth. The attraction force varies by a small amount over the Earth's surface, but the centrifugal force varies from zero at the poles to a maximum at the Equator. At different elevations, the rotational forces are different which causes level surfaces in the north-south direction to be nonparallel.

Because level surfaces at different elevations are not parallel, it is necessary to use one of two principal approaches to compute misclosures and to adjust extensive leveling nets. The first approach, used in the General Adjustment of 1929 and currently in the reduction of subsequent leveling to the National Geodetic Vertical Datum of 1929, was to apply a correction to the north-south range of leveling lines at significant average elevations above or below the geoid to reduce the elevations to consistency with the reference surface. The correction is called the "orthometric correction." Theoretical (normal) gravity at the mean latitudes of the level lines or sections was used for this method. The effect of this method was to account for the nonparallelism of level surfaces and to somewhat distort the elevations in the north-south direction. The distortion causes some problems in hydraulics. Because this procedure does not take gravity anomalies into account, it is inexact.

The second procedure is to compute and adjust on the basis of the geopotential differences. The geopotential difference between two points is the product of the actual mean gravity and the measured metric height difference ($g\Delta h$). This procedure is exact and the resulting elevations in geopotential units can be converted to orthometric values by one of several different methods. The requirement is that true gravity at bench marks be known, as given in table 3 of "Classifications, Standards of Accuracy, and General Specifications of Geodetic Control Surveys." The elevations in geopotential units (GPU) are the same for a given point or level surface regardless of the routes taken by lines of the net (except for the errors of leveling). Therefore, adjusted elevations in geopotential units give the true relationships (hydrostatic head) between bench marks. Although this problem was recognized many years ago, no effort was made to solve it at the time because the cost of determining gravity differences using pendulum instruments was prohibitive.

Low-cost determinations of gravity differences can now be obtained using readily available gravity meters. Actual gravity values are used to compute geopotential differences. These differences are used in the adjustment and the resulting elevations in geopotential units can be converted into orthometric (geometric) elevations. In a systematic releveling of the primary network of vertical control surveys, both orthometric elevations and geopotential units for the bench marks should be published.

Marine Geodesy

National programs for the development of food and mineral resources in the Continental Shelf and Great Lakes areas and for broader oceanic research have identified the need for special geodetic support. There are major requirements for position fixing of points along seaward boundaries, for markers or special instruments on the sea floor, and for ships at sea making various geophysical measurements. There are requirements for relating the geographic positions of remote islands to continental datums and even to a worldwide Earth-centered system. There are requirements for precise determination of ocean heights for tidal research and for more accurate geoid determinations.

These requirements emphasize the global characteristics of the science of geodesy. The extension of geodetic control into the environment of the sea has now become a major national task. The development of technology to meet these requirements is being undertaken by Federal agencies and special research institutes. Very few of the classical techniques used for the development of basic geodetic networks are suitable for work at sea.

For inshore hydrographic surveying, many reliable electronic systems are available. If the distances from shore do not exceed 50 km, positions can be easily determined with accuracies of 5 to 10 m. These surveys could be classified as Third-Order. For bathymetric mapping such accuracies may be entirely adequate, but for some boundary purposes higher precision may be needed. Various techniques, some of which are modifications of land-based methods, have been used with reasonable success in establishing offshore control or geodetic ranges.

For position fixing in the deeper oceanic regions, Loran C and the navigational satellite system are available, but neither fully satisfies the instantaneous velocity requirements associated with the measurement of gravity at sea. A program of continuing research, in which position, velocity, and gravity are considered as interrelated physical measurements, is essential.

Until more precise methods and reliable techniques are available, it is not feasible to prepare standards and specifications for marine geodesy. It is essential, however, that the nature of the tasks be described and the eventual need for standards be recognized.

Geodetic Data Publications

National Network Geodetic Control data are

widely distributed to all Federal, State, and local governmental organizations without cost. Other users are charged a nominal fee to cover cost of reproduction and distribution. Data are furnished through an automatic mailing service or on an individual order basis. Several different formats are now in use in horizontal and in vertical data publications.

The standard format, on which about 75 percent of the horizontal control data have been published, is a loose-leaf publication with all descriptive and numerical data regarding a station reproduced on a single sheet of paper. The stations within a quadrangle, 30 mintues of latitude by 30 minutes of longitude, are assembled as a unit and each station is assigned a unique number.

Vertical control data for many areas of the country have been assembled in the same 30-min quadrangle units. A brief history of the leveling in the quadrangle and lists of elevations on the first pages are followed by descriptions of the bench marks.

The eight 30-min quadrangles of horizontal and vertical data encompassed by the boundaries of the standard Geodetic Control Diagram, usually 1° latitude by 2° longitude, are further assembled and indexed as a unit.

Details of distribution procedures and exceptions to the formats described above will be furnished on request.

Horizontal Control Data sheets and Vertical Control Data Quadrangles contain information on the establishing organization and year of establishment, the order of accuracy, the datum used, and archival information for internal use.

The descriptive portion of the horizontal control sheets are reproduced directly as received from the field party or establishing organization. Angles or directions and distances to reference marks are included to aid in recovery or reestablishment of the station and for possible use as a substitute station. Descriptions of vertical control bench marks are usually edited and are maintained in the line sequence in the lists of elevations.

Maintenance of Survey Marks

Maintenance of marks is an integral part of the overall mission in establishing control. All organizations must be alert to maintaining control—both the physical condition of marks as well as their descriptions, for their own continued operations and for the service of others. The maintenance of control networks is actually the responsibility of all that utilize the control.

For the National Geodetic Control Networks the National Geodetic Survey has assigned specific geographic areas to field representatives whose sole mission is to prevent the destruction of these monuments through coordination for maintenance or by actually performing the work themselves. This is accomplished by relocating endangered station and bench marks to safe locations and repairing damaged station marks. With marks in place, new or replacement reference or azimuth marks are established when necessary. Witness signs are emplaced to protect the marks. In addition, the descriptive information for the stations is updated to enable users to locate the control points more easily.

All organizations are urged to be active in this maintenance program.

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References

Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys, *Federal Geodetic Control Committee*, February 1974 (reprinted May 1974), 12 pp. Available from the National Geodetic Survey, NOS, NOAA, Rockville, Md. 20852.

Coordination of Surveying and Mapping Activities, *Office of Management and Budget Circular* A16 Revised, May 6, 1967, 4 pp. Available from OMB, Washington, D.C. 20503.

Leaflet—Table and Graph for Side Checks (for use where natural functions have been used in the triangle computations), 1961, 2 pp. Available from the National Geodetic Survey, NOS, NOAA, Rockville, Md. 20852.

Manual of Second- and Third-Order Triangulation and Traverse, *USC&GS Special Publication* No. 145, 1935, Revised 1957, 226 pp.*

Precise Astronomic Surveys, *Department of the Army Technical Manual* TM 5-442, August 1970, 287 pp. Available from the Department of the Army, Adjutant General Publication Center, 1655 Woodson Road, St. Louis, Mo. 63114.

Baker, L. S., Specifications for Horizontal Control Marks, *ESSA Technical Memorandum* C&GS TM-4, 14 pp.*

Berry, R. M., Experimental Techniques for Levels of High Precision Using the Zeiss Ni 2 Automatic Level. *U.S. Lake Survey Miscellaneous Paper* 69-4, Detroit, Mich. 48226, 1969, 23 pp.

Dracup, J. F., Suggested Specifications for Local Horizontal Control Surveys. *ACSM Technical Monograph* No. CS 1, Revised 1973, Washington, D.C. 20005, 24 pp.

Dracup, J. F., *Tests on Trilateration Surveys to be Made in Field*, 1975, 27 pp.**

Gossett, F. R., Manual of Geodetic Triangulation, *USC&GS Special Publication* No. 247, 1950 (reprinted 1971), 344 pp.*

Hoskinson, A. J., and Duerksen, J, A., Manual of Geodetic Astronomy, *USC&GS Special Publication* No. 237, 1952, 205 pp.*

Mitchell, H. C., Definitions of Terms Used in Geodetic and Other Surveys, *USC&GS Special Publication* No. 242, 87 pp.

Mitchell, H. C. and Simmons, L. G., The State Coordinate Systems (A Manual for Surveyors), Revised June 1974, *USC&GS Special Publication* No. 235, 1945, 62 pp.*

Poling, A. C., Elevations from Zenith Distances (Vertical Angles), *USC&GS Publication* G-56. Available from the National Geodetic Survey, NOS, NOAA, Rockville, Md. 20852, 1947 (reprinted 1975), 30 pp.

Rappleye, H. S., Manual of Geodetic Leveling, *USC&GS Special Publication* No. 239, 1948, 94 pp.*

Rappleye, H. S., Manual of Leveling Computation and Adjustment, *USC&GS Special Publication* No. 240, 1948, 178 pp.*

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* Available through the Superintendent of Documents, Government Printing Office, Washington, D.C. 20402.

** Available from the National Geodetic Survey, NOS, NOAA, Rockville, Md. 20852.

Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys

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Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys

Introduction

The Government of the United States makes nationwide surveys, maps, and charts of various kinds which must be referenced to national datums. These are necessary to provide basic information for the conduct of public business at all levels of government, for planning and carrying out national and local projects, for programs relating to the development and utilization of natural resources, for National defense, and for development of the country. Requirements for geodetic control surveys are most critical where intense development is taking place; included are offshore areas, where the surveys are used in the development and exploitation of the marine resources and in the delineation of state and international seaward boundaries.

State and local governments and industry regularly cooperate in various parts of the total surveying and mapping program. In making surveys and maps of large areas, it is first necessary to establish frameworks of horizontal and vertical control surveys. These provide a common basis for all surveying and mapping operations, and so ensure a coherent product.

Geodetic surveys of large areas are affected by and must take into account the curvature of the earth, astronomic observations, and gravity determinations.[1] Geodetic surveys, executed with high precision, are used to control mapping and charting operations as well as engineering projects. "Geodetic survey" and "control survey" are terms used almost interchangeably.

Control surveys are of two types: horizontal and vertical. Horizontal control surveys determine latitudes and longitudes referenced to a national datum and provide the basis for rectangular coordinate systems. Horizontal geodetic surveys are adjusted to the mathematical figure of the earth applicable to the national datum. Vertical control surveys determine elevations referred to a national datum that has been referenced to tidal measurements. Vertical geodetic surveys are adjusted with respect to the geoid, an equipotential surface of the earth approximated by mean sea level. These surveys provide permanently marked and properly described stations.

Horizontal control is established by triangulation, trilateration, and traverse procedures. Triangulation is a system of joined or overlapping triangles in which the length of an occasional side, known as a base line, is measured and the other sides are computed from angles measured at the triangle vertices. Trilateration is a method of surveying in which the lengths of the triangle sides are measured. Traverse is a method of surveying in which a sequence of lengths and directions of lines between points on the earth are measured and used in determining positions of the points.

Vertical control is established by leveling of a high order of accuracy. It provides elevations of marked points along lines that form closed circuits and is accomplished by measuring differences in elevation between consecutive bench marks.

These classifications and standards have been prepared by the Federal Geodetic Control Committee (FGCC) and have been reviewed by the American Society of Civil Engineers, the American Congress on Surveying and Mapping, and the American Geophysical Union. The opinions of other organizations and individuals were also requested and received. After consideration of all comments, the original draft was revised in this, the present form.

These new classifications and standards replace those approved March 1, 1957 (and referred to in Exhibit C to the Office of Management and Budget Circular A-16 dated October 10, 1958). The classic First-, Second-, and Third-Order nomenclatures have been retained. However, classes within the orders have been revised to reflect a requirement for greater flexibility in designing surveys and in recognition of rapidly changing requirements for higher accuracy surveys. Improvements in the technology in recent years make these changes desirable and practical.

[1] The standards of accuracy for gravity determinations and observations of astronomic latitude and longitude are not contained herein as they depend on the particular application involved; they are described in separate publications such as Coast and Geodetic Survey Special Publications No. 237, "Manual of Geodetic Astronomy, Determination of Longitude, Latitude and Azimuth," 1947, GPO, Washington, D.C.

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The specifications in the attached tables show the permissible tolerances on the indicated quantities as a function of order and class; these must be very closely followed to insure that the overall standards are achieved. Detailed specifications have been prepared by the Federal Geodetic Control Committee to supplement this publication. Some important although seemingly minor factors which do not fit into the tables could cause a survey to fail to attain the required closures even though the specified requirements were met. It is emphasized that closure is not the sole criterion for classifying a survey into a particular order, as the closure is only part of the requirements and must be considered together with the other specifications in classification.

These classifications and standards are a directive for surveys which are to be considered a part of the National Geodetic Control Networks. Densification and extension of control is also provided by analytic photogrammetric triangulation and satellite systems (e.g., Doppler). It is highly desirable that these and all other geodetic or precise engineering surveys be properly referenced to the National Networks and adhere to these standards whenever possible.

Horizontal Control

First-order (Primary Horizontal Control)

The primary framework for National Horizontal Control Network consists of arcs of triangulation as well as trilateration and traverse, spaced about 100 kilometers apart in each direction. To maintain satisfactory mathematical consistency within the contained area networks between these arcs, this primary framework should have an accuracy of at least 1 part in 100,000. The study of gradual and secular ground creep movements in the earth's crust in areas subject to seismic or tectonic activity, the testing of defense and scientific equipment, and use in high-precision engineering projects require control of this same accuracy. These surveys that develop the National Network, prepare for metropolitan expansion, and serve these scientific and engineering purposes are designated First-Order.

Second-Order, Class I (Secondary Horizontal Control)

This class includes the area networks between the First-Order arcs and detailed surveys in very high value land areas. Surveys of this class strengthen the National Network and are adjusted as part of the network. Thus, this class also includes the basic framework for further densification. The internal closures of this class of survey should indicate an accuracy of at least 1 part in 50,000.

Second-Order, Class II (Supplemental Horizontal Control)

The demands for reliable horizontal control surveys in areas which are not in a high state of development or where no such development is anticipated in the near future justify the need for this classification. This class is used to establish control along the coastline, inland waterways, and interstate highways and is recommended for controlling extensive land subdivision and construction. The control contributes to the National Network and is published as part of the network. The minimum accuracy allowable in Class II of Second-Order is 1 part in 20,000.

Third-Order Class I and Class II (Local Horizontal Control)

Surveys of this order are used to establish control for local improvements and developments, topographic and hydrographic surveys, or for such other projects for which they provide sufficient accuracy. This order of survey is based on higher order control, carefully connected to the National Network, and also should be permanently marked and adequately described. Spires, stacks, standpipes, flag poles, and other identifiable objects located to this accuracy also have significant value for many surveying and engineering projects. The work should be performed with sufficient accuracy to satisfy the standards listed in the tables.

Although the specifications for the triangle closure and side checks are better than the minimum required for the standards, the specifications are logical and obtainable with judicious procedures; adherence to the standards is recommended.

Standards for surveys below Third-Order are not included in these specifications.

TABLE 1.—STANDARDS FOR THE CLASSIFICATION OF GEODETIC CONTROL AND PRINCIPAL RECOMMENDED USES

Horizontal Control

Classification	*First-Order*	*Second-Order*		*Third-Order*	
		Class I	*Class II*	*Class I*	*Class II*
Relative accuracy between directly connected adjacent points (at least)	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000
Recommended uses	Primary National Network. Metropolitan Area Surveys. Scientific Studies	Area control which strengthens the National Network. Subsidiary metropolitan control.	Area control which contributes to, but is supplemental to, the National Network.	General control surveys referenced to the National Network. Local control surveys.	

Vertical Control

Classification	*First-Order*		*Second-Order*		*Third-Order*
	Class I	*Class II*	*Class I*	*Class II*	
Relative accuracy between directly connected points or benchmarks (standard error)	0.5 mm $\sqrt{K}$	0.7 mm $\sqrt{K}$	1.0 mm $\sqrt{K}$	1.3 mm $\sqrt{K}$	2.0 mm $\sqrt{K}$
	(K is the distance in kilometers between points.)				
	Basic framework of the National Network and metropolitan area control. Regional crustal movement studies. Extensive engineering projects. Support for subsidiary surveys.		Secondary framework of the National Network and metropolitan area control. Local crustal movement studies. Large engineering projects. Tidal boundary reference. Support for lower order surveys.	Densification within the National Network. Rapid subsidence studies. Local engineering projects. Topographic mapping.	Small-scale topographic mapping. Establishing gradients in mountainous areas. Small engineering projects. May or may not be adjusted to the National Network.

TABLE 2.—CLASSIFICATION, STANDARDS OF ACCURACY, AND GENERAL SPECIFICATIONS FOR HORIZONTAL CONTROL

TRIANGULATION

Classification	*First-Order*	*Second-Order*		*Third-Order*	
		Class I	*Class II*	*Class I*	*Class II*
Recommended spacing of principal stations	Network stations seldom less than 15 km. Metropolitan surveys 3 km to 8 km and others as required.	Principal stations seldom less than 10 km. Other surveys 1 km to 3 km or as required.	Principal stations seldom less than 5 km or as required.	As required	As required
Strength of figure					
R_1 between bases					
Desirable limit	20	60	80	100	125
Maximum limit	25	80	120	130	175
Single figure					
Desirable limit					
R_1	5	10	15	25	25
R_2	10	30	70	80	120
Maximum limit					
R_1	10	25	25	40	50
R_2	15	60	100	120	170
Base measurement					
Standard error (1)	1 part in 1,000,000	1 part in 900,000	1 part in 800,000	1 part in 500,000	1 part in 250,000
Horizontal directions (2)					
Instrument	0".2	0."2	0".2	1".0	1".0
Number of positions	16	16	8 or 12	4	2
Rejection limit from mean	4"	4"	5" or 5"	5"	5"
Triangle closure					
Average not to exceed	1".0	1".2	2".0	3".0	5".0
Maximum seldom to exceed	3".0	3".0	5".0	5".0	10".0
Side checks					
In side equation test, average correction to direction not to exceed	0".3	0".4	0".6	0".8	2"
Astro azimuths (3)					
Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of Obser./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	0".6	0".8	3".0
Vertical angle observations (4)					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"

Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Closure in length (5) (also position when applicable) after angle and side conditions have been satisfied, should not exceed	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000
		TRILATERATION			
Recommended spacing of principal stations	Network stations seldom less than 10 km. Other surveys seldom less than 3 km.	Principal stations seldom less than 10 km. Other surveys seldom less than 1 km.	Principal stations seldom less than 5 km For some surveys a spacing of 0.5 km between stations may be satisfactory.	Principal stations seldom less than 0.5 km.	Principal stations seldom less than 0.25.
Geometric configuration (6) Minimum angle contained within, not less than	25°	25°	20°	20°	15°
Length measurement Standard error (1)	1 part in 1,000,000	1 part in 750,000	1 part in 450,000	1 part in 250,000	1 part in 150,000
Vertical angle observations (4) Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths (3) Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of obs./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	0".6	0".8	3".0
Closure in position (5) after geometric conditions have been satisfied should not exceed	1 part 100,0[illegible]	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000

TRAVERSE

Classification	First-Order	Second-Order		Third-Order	
		Class I	Class II	Class I	Class II
Recommended spacing of principal stations	Network stations 10-15 km Other surveys seldom less than 3 km.	Principal stations seldom less than 4 km except in metropolitan area surveys where the limitation is 0.3 km.	Principal stations seldom less than 2 km except in metropolitan area surveys where the limitation is 0.2 km.	Seldom less than 0.1 km in tertiary surveys in metropolitan area surveys. As required for other surveys.	
Horizontal directions or angles (2)					
Instrument	0".2	0".2	0".2	1".0	1".0
Number of observations	16	8 or 12*	6 or 8*	4	2
Rejection limit from mean	4"	4" or 5"	4" or 5"	5"	5"
Length measurements		* May be reduced to 8 and 4, respectively, in metropolitan areas.			
Standard error (1)	1 part in 600,000	1 part in 300,000	1 part in 120,000	1 part in 60,000	1 part in 30,000
Reciprocal vertical angle observations (4)					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of stations between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths					
Number of courses between azimuth checks (7)	5-6	10-12	15-20	20-25	30-40
No. of obs/night	16	16	12	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	1".5	3".0	8".0
Azimuth closure at azimuth check point not to exceed (8)	1".0 per station or 2" $\sqrt{N}$	1".5 per station or 3" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 2".0 per station or 3" $\sqrt{N}$	2".0 per station or 6" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 4".0 per station or 8" $\sqrt{N}$	3".0 per station or 10" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 6" per station or 15" $\sqrt{N}$	8" per station or 30" $\sqrt{N}$
Position closure (5)(8) after azimuth adjustment	0.04m $\sqrt{K}$ or 1:100,000	0.08 $\sqrt{K}$ or 1:50,000	0.2m $\sqrt{K}$ or 1:20,000	0.4m $\sqrt{K}$ or 1:10,000	0.8m $\sqrt{K}$ or 1:5,000

NOTE (1)

The standard error is to be estimated by

$$\sigma_m = \sqrt{\frac{\Sigma v^2}{n(n-1)}}$$

where σ_m is the standard error of the mean, v is a residual (that is, the difference between a measured length and the mean of all measured lengths of a line), and n is the number of measurements.

The term "standard error" used here is computed under the assumption that all errors are strictly random in nature. The true or actual error* is a quantity that cannot be obtained exactly. It is the difference between the true value and the measured value. By correcting each measurement for every known source of systematic error, however, one may approach the true error. It is mandatory for any practitioner using these tables to reduce to a minimum the effect of all systematic and constant errors so that real accuracy may be obtained.

"Manual of Geodetic Triangulation," Revised edition, 1959, for definition of "actual error."

NOTE (2)

The figure for "Instrument" describes the theodolite recommended in terms of the smallest reading of the horizontal circle. A position is one measure, with the telescope both direct and reversed, of the horizontal direction from the initial station to each of the other stations. See FGCC "Detailed Specifications" for number of observations and rejection limits when using transits.

NOTE (3)

The standard error for astronomic azimuths is computed with all observations considered equal in weight (with 75 percent of the total number of observations required on a single night) after application of a 5-second rejection limit from the mean for First- and Second-Order observations.

* See page 267 of Coast and Geodetic Survey Special Publication No. 247,

NOTE (4)

See FGCC "Detailed Specifications" on "Elevation of Horizontal Control Points" for further details. These elevations are intended to suffice for computations, adjustments, and broad mapping and control projects, not necessarily for vertical network elevations.

NOTE (5)

Unless the survey is in the form of a loop closing on itself, the position closures would depend largely on the constraints or established control in the adjustment. The extent of constraints and the actual relationship of the surveys can be obtained through either a review of the computations, or a minimally constrained adjustment of all work involved. The proportional accuracy or closure (i.e. 1/100,000) can be obtained by computing the difference between the computed value and the fixed value, and dividing this quantity by the length of the loop connecting the two points.

NOTE (6)

See FGCC "Detailed Specifications" on "Trilateration" for further details.

NOTE (7)

The number of azimuth courses for First-Order traverses are between Laplace azimuths. For other survey accuracies, the number of courses may be between Laplace azimuths and/or adjusted azimuths.

NOTE (8)

The expressions for closing errors in traverses are given in two forms. The expression containing the square root is designed for longer lines where higher proportional accuracy is required.

The formula that gives the smallest permissible closure should be used.

N is the number of stations for carrying azimuth.
K is the distance in kilometers.

Vertical Control

First-Order

Leveling of this order is used in developing the basic framework of the national vertical net in the United States (Basic Nets A and B) so that few points in the country will be more than 50 km from an established First-Order bench mark. All lines close upon First-Order leveling to form circuits. The lines are divided into sections 1 to 2 km in length, and each section is leveled forward and backward. The difference in the two levelings must not exceed 3.0 mm $(K)^{1/2}$ for Class I (Basic Net A), or 4.0 mm $(K)^{1/2}$ for Class II (Basic Net B), where K is the distance in kilometers. The same criteria are recommended for use in establishing primary networks of leveling in metropolitan areas, except that the lines should be closely spaced.

Actual gravity values at the bench marks are needed to compute geopotential differences. If the gravity is not already available with the required accuracy, it shall be measured at sufficient number of bench marks so that the gravity uncertainty computed for any interval will not affect the accuracy of the geopotential difference by more than 0.2 x 10^{-3}gpu[1].

Second-Order, Class I

Leveling of this class should be used in developing the secondary net of the national vertical network and in densifying precise control in metropolitan areas. The leveling should connect to leveling of equal or greater accuracy to form closed circuits. All lines should be divided into sections 1 to 2 km in length, and each section should be run forward and backward, the two runnings of a section not to differ more than 6 mm $(K)^{1/2}$, where K is the length of the section in kilometers.

Second-Order, Class II

This class should be used in subdividing loops of First-Order and Second-Order, Class I leveling to establish general area coverage. The leveling should form closed circuits with leveling of equal or greater accuracy, and should rarely extend more than 50 km unsupported in this manner. Single-run leveling for short distances is acceptable, but for distances greater than 25 km double-run leveling is recommended. For double-run leveling, the line should be divided into sections of 1 to 3 km, and the forward and backward running of each section should differ by not more than 8 mm $(K)^{1/2}$, where K is the distance in kilometers.

Third-Order

Third-Order leveling may be used in subdividing loops of First- and Second-Order leveling, where additional control is required for local development. Third-Order lines may be single-run, but must always be loops or circuits closed upon lines of equal or higher order with a check of 12 mm $(K)^{1/2}$, or better, where K is the length of the line in kilometers. It is recommended that single-run lines be limited to 10 km in length, and double-run, to 25 km. Exceptions would be control for topographic mapping at a scale of 1:24,000 or smaller, and leveling in mountainous areas, where accuracy requirements may permit Third-Order lines 50 km long.

Leveling of Lower Order

Trigonometric leveling, barometric leveling, and fly leveling may be considered as Fourth-Order, or less; standards for these surveys are not included in these classifications. Elevations are normally published as part of other data.

Instruments and Procedures

For First-Order leveling, an automatic or tilting level with parallel plate micrometer, or equivalent, should be used. It should have horizontal sensitivity of 0.25 second of arc, or better, and should have high-quality optics that will permit repeat reading of 0.2 mm on a geodetic rod at a distance of 50 m under normal atmospheric conditions. The instrument should remain stable in a moderate breeze (up to 20 km/h), and should be temperature compensated. The rod should be composed of an invar scale under tension on a wood or metal frame equipped with a bull's-eye bubble. The scale should be accurate overall to 0.1 mm. The rods are used in pairs, each rod alternating as the forward and backward rod, and the same rod is always read first regardless of position. That is, one rod will be used for the back readings on odd-numbered instrument stations, and for the forward readings on even-numbered stations. The lengths of sights should not exceed the criteria given in table 3, and should be shortened if refraction or scintillation is troublesome. Balancing of forward and backward sights also shall conform to the limits given in table 3.

[1] 1 geopotential unit (gpu) = 1 kilogalmeter (10^5 cm^2 sec^{-2})

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TABLE 3.—CLASSIFICATION, STANDARDS OF ACCURACY, AND GENERAL SPECIFICATIONS FOR VERTICAL CONTROL

Classification	First-Order: Class I, Class II	Second-Order: Class I	Second-Order: Class II	Third-Order
Principal uses	Basic framework of the National Network and of metropolitan area control Extensive engineering projects Regional crustal movement investigations Determining geopotential values	Secondary control of the National Network and of metropolitan area control Large engineering projects Local crustal movement and subsidence investigations Support for lower-order control	Control densification, usually adjusted to the National Net. Local engineering projects Topographic mapping Studies of rapid subsidence Support for local surveys	Miscellaneous local control; may not be adjusted to the National Network. Small engineering projects Small-scale topo. mapping Drainage studies and gradient establishment in mountainous areas
Recommended spacing of lines				
National Network	Net A; 100 to 300 km *Class I* Net B; 50 to 100 km *Class II*	Secondary Net; 20 to 50 km	Area Control; 10 to 25 km	As needed
Metropolitan control; other purposes	2 to 8 km As needed	0.5 to 1 km As needed	As needed As needed	As needed As needed
Spacing of marks along lines	1 to 3 km	1 to 3 km	Not more than 3 km	Not more than 3 km
Gravity requirement;	0.20×10^{-3}gpu	.....	.....	.....
Instrument standards (1)	Automatic or tilting levels with parallel plate micrometers; invar scale rods	Automatic or tilting levels with optical micrometers or three-wire levels; invar scale rods	Geodetic levels and invar scale rods	Geodetic levels and rods
Field procedures	Double-run; forward and backward, each section	Double-run; forward and backward, each section	Double- or single-run	Double- or single-run
Section length	1 to 2 km	1 to 2 km	1 to 3 km for double-run	1 to 3 km for double-run
Maximum length of sight	50 m Class I; 60 m *Class II*	60 m	70 m	90 m
Field procedures (2)				
Max. difference in lengths				
Forward & backward sights per setup	2 m Class I; 5 m *Class II*	5 m	10 m	10 m
per section (cumulative)	4 m Class I; 10 m *Class II*	10 m	10 m	10 m
Max. length of line between connections	Net A; 300 km Net B; 100 km	50 km	50 km double-run 25 km single-run	25 km double-run 10 km single-run
Maximum closures (3)				
Section; fwd. and bkwd.	3 mm $\sqrt{K}$ Class I; 4 mm $\sqrt{K}$ Class II	6 mm $\sqrt{K}$	8 mm $\sqrt{K}$	12 mm $\sqrt{K}$
Loop or line	4 mm $\sqrt{K}$ Class I; 5 mm $\sqrt{K}$ Class II	6 mm $\sqrt{K}$	8 mm $\sqrt{K}$	12 mm $\sqrt{K}$

NOTE (1)

See text for discussion of instruments.

NOTE (2)

The maximum length of line between connections may be increased to 100 km for double run for Second-Order, Class II, and to 50 km for double run for Third-Order in those areas where the First-Order control has not been fully established.

NOTE (3)

Check between forward and backward runnings where K is the distance in kilometers.

For Second-Order leveling, parallel plate or similar micrometers on the instruments are not required but are desirable. Otherwise, three-wire geodetic spirit or automatic levels with horizontal sensitivity of 0.5 seconds of arc are recommended. Stadia should be read for keeping the sight lengths balanced and for adjustment purposes. The rod scale should be accurate overall to 0.2 mm. For Third-Order leveling, it is necessary to read only the middle wire when using the three-wire geodetic spirit level.

National Networks of Geodetic Control

There is a continuing basic program for establishing, upgrading, and maintaining of the National Geodetic Control Networks described in these classifications to provide adequate spacing as well as sufficient strength and accuracy to meet the needs and satisfy the requirements of engineers and scientists engaged in the development and conservation of the resources of the United States. The accuracy and spacing of the National Network control must be based on the most stringent requirements to be placed upon it; this accuracy should be the best achievable considering economic advantage and the capabilities of instruments and technology.

Horizontal Control Network

National overall accuracy is derived through the establishment of a fundamental backbone of the high-precision traverses, with accuracies of 1:1,000,000 integrated with satellite triangulation. Primary horizontal control (First-Order, accuracy of 1:100,000) establishes the principal network with arcs or traverses at a spacing not in excess of 100 km. Secondary horizontal control (Second-Order, Class I, accuracy of 1:50,000), together with supplemental work (Second-Order, Class II, accuracy of 1:20,000) breaks down the principal network and strengthens the whole. Local horizontal control (Third-Order, Class I, accuracy of 1:10,000, and Third-Order, Class II, accuracy of 1:5,000) is referenced to the network. To meet these accuracy criteria and for optimum utilization of the network, approximate station spacing, in general, along First-Order arcs is 15 km, with a breakdown to 10 km for Second-Order, Class I and 5 km for Second-Order, Class II. In areas of high land value, the station spacing for First-Order is a maximum of 3 km and Second-Order, Class I, 2 km. Second-Order, Class II and all Third-Order, Class I and Class II are as required for local usage.

In addition to the above spacing of stations, control shall be established at all airports, towns of 2,000 or more population, colleges, and at a 6- to 8-km spacing along coastlines and interstate highways. Although wider spacing may suffice for federal topographic mapping, closer spacing is required for surveys of property, highways, transmission lines, reclamation projects, and numerous other engineering activities. Such stations should be situated so they are readily available to local engineers and surveyors. Frequency, stability, recoverability, and accessibility are factors to be considered when emplacing marks (including underground marks, reference marks, and witness posts when appropriate).

Nationwide High Precision Traverses

These traverses provide scale for the worldwide satellite triangulation network and upgrade the scale and orientation of the National Network of Horizontal Control. They consist of a series of high-precision length, angle, and astronomic azimuth determinations running approximately east-west and north-south through the conterminous states, forming somewhat rectangular loops. Smaller loops and spur traverses are added to connect satellite triangulation stations and areas of special interest.

Standards of accuracy for high-precision traverses are not given herein. Supporting specifications approved by the Federal Geodetic Control Committee outline procedures indicating the care required to obtain the approximate 1 part in 1,000,000 accuracy.

The high precision traverses provide a geodetic reference framework of continental extent which, when remeasured in the future, will provide data for studies of deformation of the earth's crust, including continental drift and spreading.

Triangulation, Trilateration, Traverse, and Bases

Methods of establishment of horizontal control are triangulation, trilateration, and traverses, either separately or in combination. This publication does not differentiate between the methods used. Appropriate instrumentation and procedures are to be selected to satisfy the standards in the tables and to obtain the precision required.

The general availability of electronic distance-measuring equipment has practically eliminated the use of taping procedures for the measurement of

base lines or traverse lengths. Accuracies are comparable or superior to those obtained with invar tapes.

Vertical Control Network

In leveling, inasmuch as survey errors propagate at least as the one-half power of the distance surveyed, and because of the great continental distances in the United States, the precision of the primary network measurements must be of high order.

The national vertical control network provides for this by a framework of high-precision basic control supporting a secondary network, which in turn supports area control of a density convenient for users.

The framework consists of two interrelated systems, designated as Basic Nets A and B, covering the country. Basic Net A is composed of lines of First-Order, Class I leveling, forming more or less rectangular circuits 100 to 300 km across, and is the fundamental reference system. Basic Net B is composed of lines of First-Order, Class II leveling, subdividing the circuits of Net A to provide an overall spacing of 50 to 100 km. The Secondary Network consisting of lines of Second-Order, Class I leveling spaced 25 to 50 km apart densifies the national network. The more closely spaced Second-Order, Class II area control is referenced to the network.

The surface of our continent is constantly changing because of tectonic and other physical forces. Large areas are undergoing subsidence owing to removal of ground water or petroleum resources, regions are still emerging from glacier recession, elevations are changing because of natural movement of the earth's crust, and seismic activity is continuous. Engineers and planners need up-to-date elevations to cope with new water levels and crustal changes.

One of the most important factors in the development of a control level net is to establish marks that will remain stable. In some areas, the causes of ground movement are so deep-seated that it is difficult to establish a mark that will remain stable indefinitely. The usual practice is to establish, at 1- to 3-km intervals along lines of First- or Second-Order leveling, bench mark disks set in concrete posts, stable structures, and outcropping bedrock, or secured to rods and pipes driven to refusal (or a stable stratum). At each intersection of lines of the basic framework, a cluster of three "super" marks is established; this cluster consists of deep isolated-pipe marks or marks in bedrock. Intersections of lines of the Secondary Network are marked by clusters of three driven-rod marks. In both cases, the next adjacent regular bench mark on each level line is the driven-rod or outcropping rock type. This pattern permits, to a degree, a check on the stability of marks for future extensions.

Earth Movement Surveys

Surveys for the measurement of horizontal and vertical movements of the earth's crust are undertaken in areas of known or suspected subsidence or seismic activity, where the safety of the inhabitants and the economy of the region are involved. These surveys consist of periodically repeated precise measurements to provide information relative to crustal distortions and strain buildup for use in geophysical studies and in engineering design and maintenance. Similar surveys, particularly vertical, are required in areas of subsidence caused by withdrawal of underground resources. These repeat surveys are necessary to maintain the quality of the networks and to correlate between changes in local mean sea level and crustal distortion. The economic and engineering impact, and the rate of movement, dictate the period between surveys.

TABLE 4.—NATIONAL GEODETIC NETWORKS

HORIZONTAL

Classification	Nationwide high precision traverses—Satellite Control	*First-Order*	*Second-Order*		*Third-Order*	
			Class I	*Class II*	*Class I*	*Class II*
Network component	Basic horizontal framework (control establishes the National Network)	Primary horizontal network (control develops the National Network)	Secondary horizontal control (control strengthens the National Network)	Supplemental horizontal control (Control contributes to the National Network)	Local horizonta control (control is referenced to th National Network)	
Nominal accuracy or precision between adjacent points	1 part in 1,000,000	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part ir 5,000
Recommended density of control	Traverses and satellite stations at 900-1200 km. Stations at 15 km. to limit of technical and geometric restraints	Arcs not in excess of 100 km. Stations at 12-20 km. Urban control 3-8 km.	Stations at 10-13 km. Urban control 1-3 km.	As required	As required	

VERTICAL

Classification	*First-Order*		*Second-Order*		*Third-Order*
	Class I	*Class II*	*Class I*	*Class II*	
Network component	Basic Vertical Network A (control establishes the National Network)	Basic Vertical Network B (control establishes the National Network)	Secondary Vertical Network (Control develops the National Network)	Supplemental Vertical Control (Control contributes to the National Network)	Local vertical control
Nominal accuracy *between points**	1.5 mm$\sqrt{K}$	2 mm $\sqrt{K}$	3 mm $\sqrt{K}$	4 mm $\sqrt{K}$	6 mm $\sqrt{K}$
Recommended density of *lines*	100-300 km	50-100 km.	25-50 km	10-25 km	As needed

* One-half of permissable closure

Specifications To Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys

By
Federal Geodetic Control Committee
John O. Phillips, Chairman

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Survey
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Attachment D

Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys

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