MANUAL OF PHOTOGRAMMETRY

With Particular Reference to The [illegible]
River Basin Development in Indonesia

REPUBLIC OF INDONESIA

May 1974

Overseas Technical Cooperation Agency

(JAPAN)

MANUAL OF PHOTOGRAMMETRY

With Particular Reference to the Barito
River Basin Development in Kalimantan

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BACKGROUND

1. Dispatch of the survey team for the Barito River Basin Development.

In compliance with the request of the Government of Indonesia, the Government of Japan proposed the five-stage programmes to set up a master plan for the development of the Barito River Basin and entrusted the Overseas Technical Cooperation Agency (OTCA) with execution of the survey for: the formulation of survey programmes for collecting basic data which would be required to set up the master plan; and the primary recognition of the feasibility of resources development.

OTCA sent to Indonesia a survey team comprising 15 experts of various fields during the period from September to November 1970.

According to its survey report, "Map and aerial photos required for various development programmes and surveys in the Barito River Basin are extremely deficient. Aerial photography, ground control and mapping should therefore be implemented promptly and systematically to produce the required maps and photos. It is desirable that the aerial photography be immediately used by aerial photo interpretation for the survey of forests, soil and geology in the Basin."

2. Topographic Mapping of the Barito River Basin

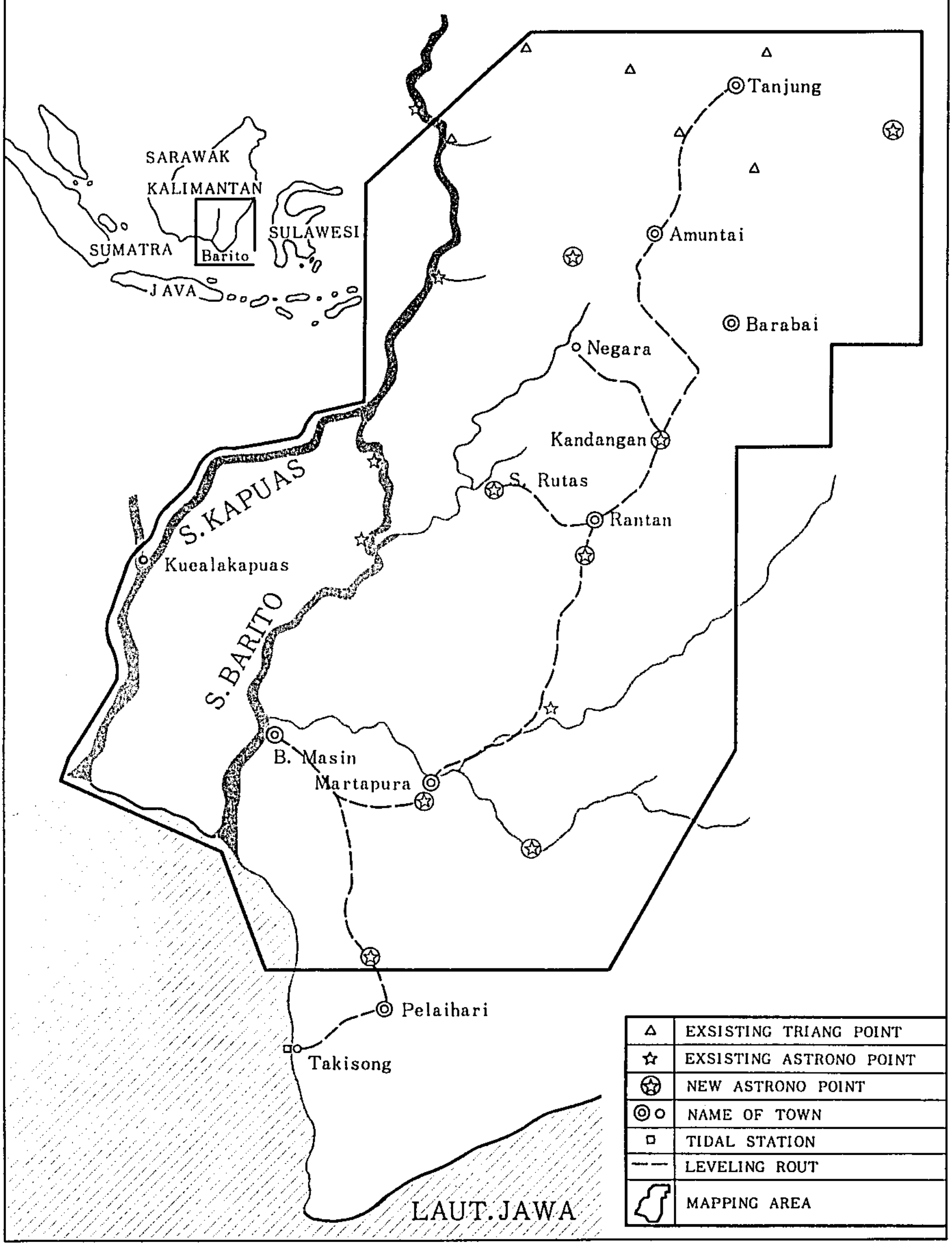
Upon the request of the Government of Indonesia in August 1971 calling for the preparation of topographic maps necessary for the planning of the Barito River Basin Development, the Government of Japan considered it most essential for the development of such a vast area as the Barito River Basin and decided to cooperate Indonesia through Japan's technical cooperation programme.

As the mapping of the Basin necessitates 3-year continuous work, it was planned ,to carry out the work for 3 years from Fiscal Year 1971 to prepare topographic maps (coverage: 16,800 km^2 scale: 1/50,000) covering the eastern part of the Power Basin extending along a number of tributaries such as the Negara and Martapura which is fairely well developed and has a relatively large population.

With some extension of the coverage thereafter, the topographic maps (coverage: app. 20,000 km^2, scale: 1/50,000) are scheduled to be completed and delivered to the Government of Indonesia in July 1974.

This Manual contains report on the process of the above mentioned works and at the same time, it gives references on which planning of the mapping work based.

BARITO RIVER BASIN
MAPPING PROJECT LOCATION MAP



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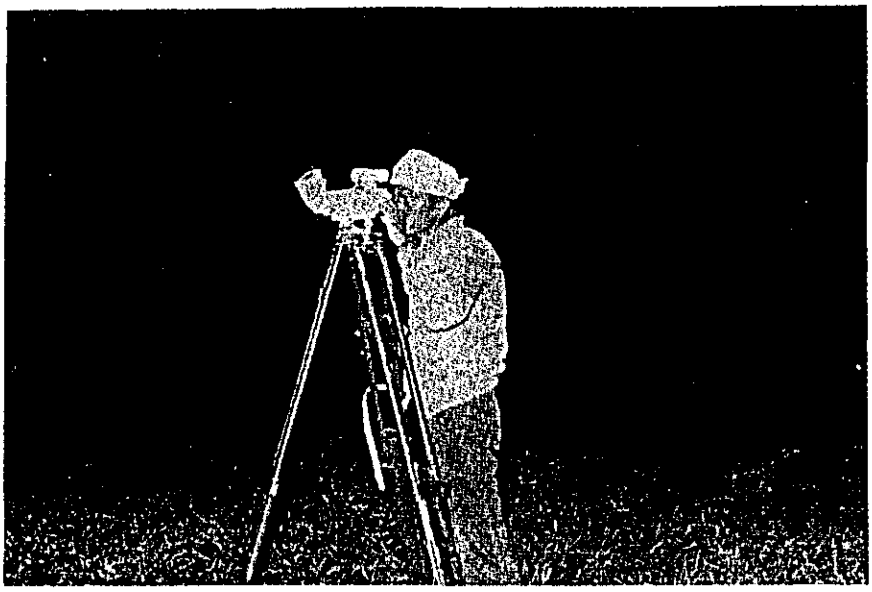
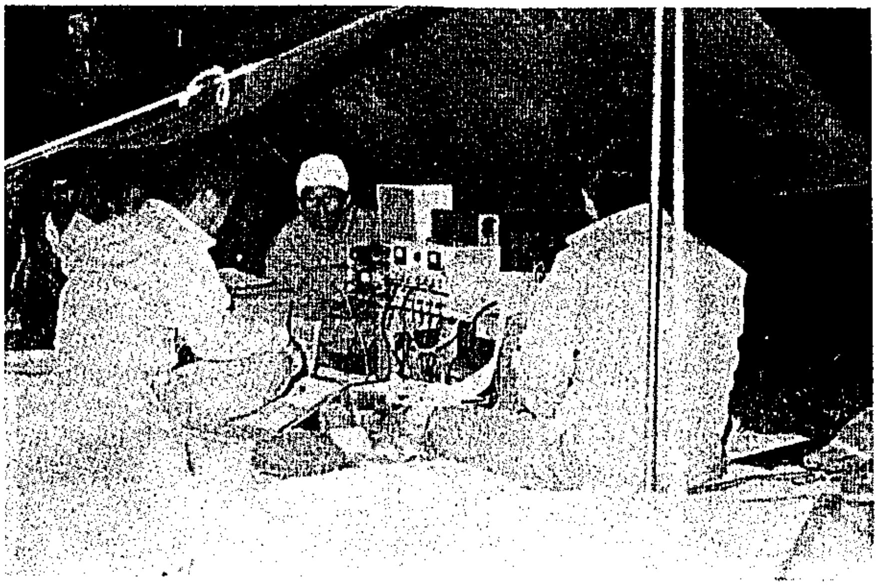
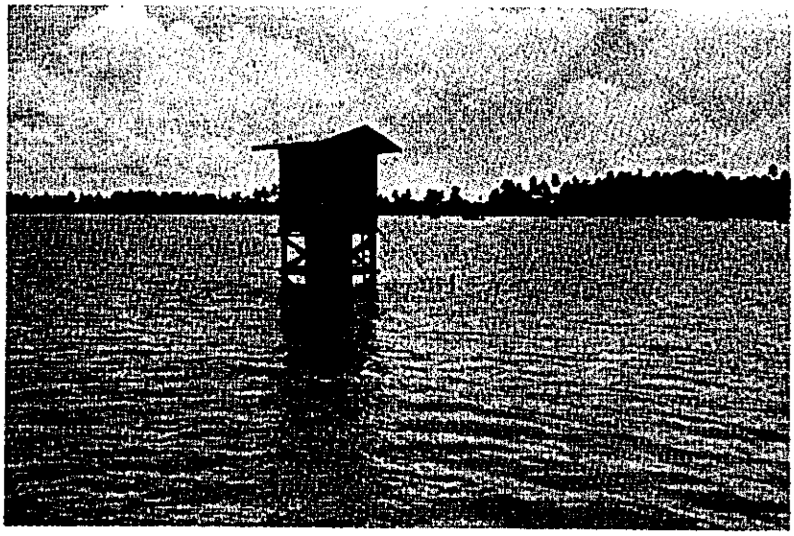
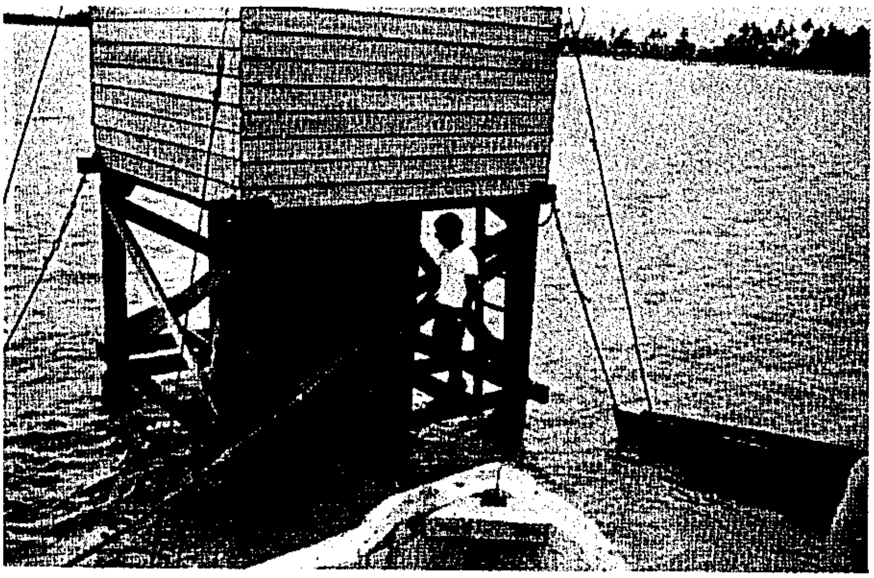




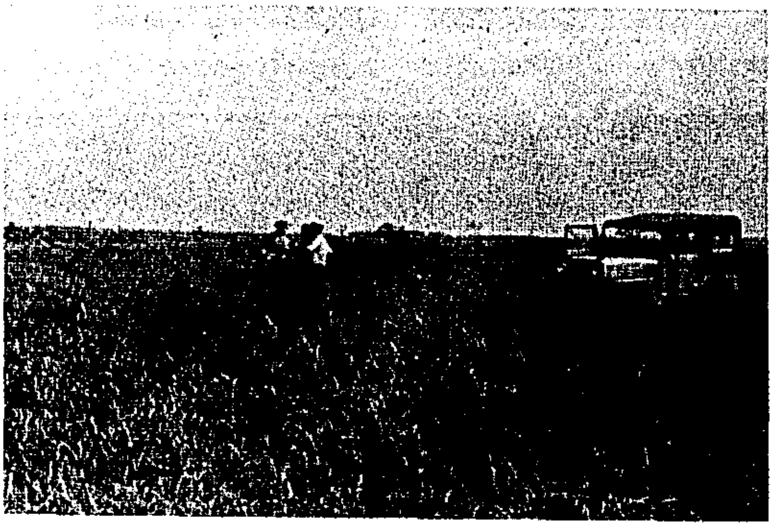
ASTORONOMICAL SURVEY (PELAIHARI)



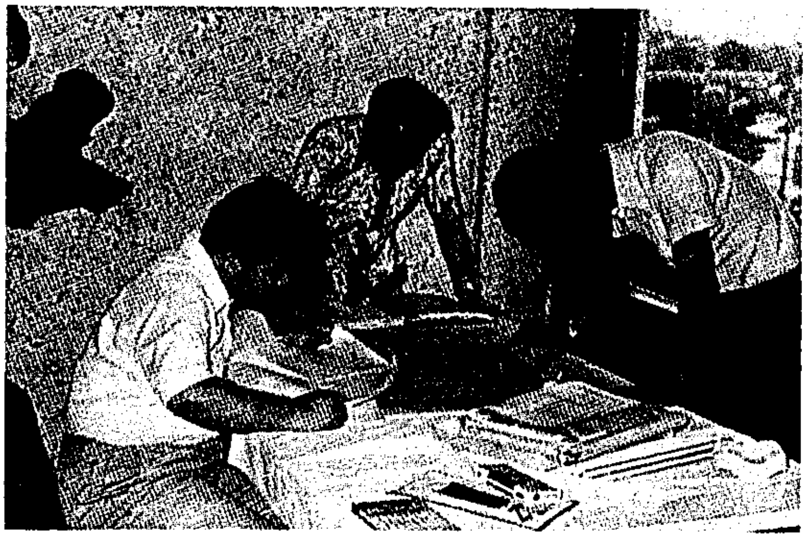


TIDE STATION (TAKISONG)

LEVELING SURVEY (KANADANGAN)



FIELD RECONNAISSANCE



PROOF READING (BANJARMASIN)

CONTENTS

I. GENERAL PLANNING

Forward

When regional development is undertaken, first of all, it is necessary to understand regional conditions at once before making a plan. Maps and aerial photographs will be needed for this purpose, and more precise maps and photographs will also be necessary in the case of making practical designs. The implementation of this work will required the decision of a horizontal position and height which it is based upon. Such control points will be needed for mapping too. The unification of those systems along with the expansion of the region which needs maps and control points will be advantageous and economical.

Besides, this is effective for mapping too, and in this report, we have compiled the details of the present survey and have made an approach by way of an idea to put into a unified system the control point and basic topographic mapping, particularly photogrammetry which aerial photographs are used for.

1. OUTLINE

1.1 System of Survey and Mapping

The systems of survey and mapping in developing countries are fundamentally not different from those of advanced countries, but the needs or feasibility are not the same according to each of the countries. These will change with prerequisites, political, cultural, natural environemtnal, technical, or of people in each country. Therefore, it will be difficult to apply the technical system of one country to another one, or to change its existing system in a large scale. In some cases, even if some method is useful, it will take much time until the application has been realized, or will be more expensive from the stand-point of economy. Developing countries give preference to "can-see" work such as civil survey or plan-making, and in most cases they put aside fundamental recording work in the topographic survey or cadastral survey. Because of their fundamentality, it should not be ignored that these be given preference at the beginning. The degree of preference in the work, necessities of maps, or economic or technical abilities to excute the survey project are not on the same levels according to the countries involved. Practically speaking, it will be different according to the situations that which method of surveying should be adopted, or in what degree perfection should be required. And attention should be paid not let the large part of the work be useless through the hasty surveying and mapping.

1.2 System of Map Scales

At the beginning of planning, the choice of a system of map scales should be made of, first of all, for the purpose. Various scale system will not be needed from the stand-point of economy. It will be desirable that plotting will be made with one of the scales, by noticing a series of scales because the enlargement of scales for cities may be considered necessary. Once the scales are decided, they can be the standards for it. And the density of points or scopes can be decided accordingly. Every twice scale seems to be a little too small, and a scale of every four or five times will be available for this purpose; 1/500 – 1/2,000 – (1/2,500) – 1/10,000 – 1/50,000 – (2/250,000) or taking a system of every 10 times: 1/500 – 1/5,000 – (1/500,000), or 1/1,000 – 1/10,000 – 1/100,000 – (1/1,000,000).

It is more advantageous to make a plan for starting from the plotting of small scales at first, because this will not take much time. If small scale maps are available, necessary sections for large-scale photography and plotting can be selected from them. It is advisable to pay attention to how to select scales so that the density of items shown on the map will be approximately equal.

The scales of standard topographical maps are 1/50,000 in most cases. 1 mm on these maps are equivalent to 50 m. Taking into consideration the thickness of lines or expansion and contraction of paper, the precision of locations representable on the maps the limits of representing power of ground features may be taken as about 0.5 mm, and this can represent 25 m on the spot from the scale of 1/50,000. Therefore, it is possible to represent the unit of a village, although an individual house building can not be represented. But there is some doubt from the standpoint of production and utilization, because it exceeds the limits to draw smaller parts than the representing power. In other words, a larger scale should be adopted, in case much smaller parts are necessary, or mapping is needed for designing or city planning.

1.3 Contour Intervals

Contour intervals being the case too, although small contour intervals are drawn at random, their mutual distance become very small on the maps. Therefore, the utilization values for the whole map will be less, because they will not have the utilization value and the other small parts will not be seen clearly.

For the general standards, in the case of the maximum slope (about 45°), the horizontal distance between contours on the map should be approximately 0.5 to 1.0 mm. The standards in most cases are the following combinations;

Map Scales	1/5,000	1/10,000	1/25,000	1/50,000
Contour Intervals	5 m	10 m	10 to 20 m	20 to 50 m
Horizontal Distances on Maps (in case of 45°)	1.0 mm	1.0 mm	0.4 to 0.8 mm	0.4 to 1.0 mm

1.4 Utilization of Photomaps and Orthophotographs

In case it should be understood that the plotting by way of the plotting machine will be very expensive or will take too much time, and it is advisable to use rectified photographs or orthophotographs. The orthophotographs which have corrected the shifting of positions caused by undulation by orthogonal-projecting will be used in hilly, areas and so on. Either of them is faster in comparison with the topographical mapping, but there still is a disadvantage that the photo interpretation is left to the user. In other words, a special machine to be called orthophotograph will be necessary for the work of orthophotos..

2. STANDARD COORDINATES SYSTEM

2.1 Standard Coordinates System

When topographical mapping survey is executed, first of all, geodetic coordinates systems should be decided on in a nation-wide way. The international reference ellipsoid should be made a choice of as the geodetic coordinates system, but from the standpoint of contacting relations with neighboring countries, there may be some cases that this ellipsoid can not be used. Therefore, consideration should be taken into account which method will be better in each case. As a rule, the Greenwich longitude is used for coordinates to obtain the geographic longitude and latitude. However, these only are inconvenient in the practical survey or mapping, and it is a practical way to calculate local coordinates with the 6° interval Universal Transverse Mercator system.

2.2 Standards of Heights

The height is measured from the geoid on the hypothesis that the geoid is at a certain point. Practically speaking, the mean sea level in a certain area are will be taken as the geoid. Tide observations will be conducted a suitable beach in the vicinity for this purpose. It is advisable to continue this work more than one year, if possible.

If it should be difficult to set up points or leveling routes which will be used for height control, the height will be worked out by way of indirect leveling or barometic leveling,

In the case of leveling surveying, its relations with datum points should be made clear so that there will not be discrepancy in each part later on.

3. CONTROL POINT SURVEY

3.1 Introduction

The national-wide control point network will be the foundation not only for mapping but also for the cadastral survey, coastal hydrographic survey, cities and towns, roads and irrigation planning.

Therefore, it will not always correspond perfectly to the adaptability to topographic mapping or in a long run to photogrammetry, and in some cases supplementary control point surveys may be necessary for the purpose of topographic mapping.

The control point survey is roughly divided into the survey of height and that of horizontal position; for the former, the tidal observation or leveling survey will be conducted, and for the latter, the astronomical survey, triangulation or traversing will be adopted. The astronomical survey had better be utilized instead as much as possible, because the old style triangulation will take much time and money. And it is also necessary to correct the twists of control point networks by adjusting the differences between both of them, after conducting the astronimical survey at some triangulation points. Attention should be paid to check at least whether Laplace adjustments will be necessary or not. The outline of the standard control point surveys will be given in the following item.

3.2 Items

3.2.1 Astronomical Survey

This is the most fundamental survey to get the longitude at a point on the globe, and if the observation of celestial body and time signal receiving are available, the positions of each point will be obtained individually. Therefore, in case the views to the far distance are not open to the eyes, and the observations can not be conducted across seas, this method will be adopted. And when the meridian transit or Astrolabe is used for more accurate observations, the accuracy of 1 to 2 seconds (30 to 60 m) will determine the positions, but in the case of simpler instruments, the accuracy will be 3 to 5 seconds (90 to 150 m). In this case, what to be obtained directly is not XY, length of the unit, because of the effects of the earth curvature but the longitude and latitude of the angle on the curved surface of the earth. And it may be affected in some cases by the irregular gravity caused by the irregular distribution of inner mass of the earth, and there may be constant errors of approximately 10 to 300 m.

3.2.2 Artificial Satellite Observation

It is possible to get any point on the globe by conducting the observation by way of the geodetic satellite like the astronomical survey does. In this case, the accuracy is so high that it will be obtained with the accuracy of less than 1 second (less than 30 m). However, it is necessary to conduct simultaneous observations at three control points, the locations of which have been known. In this case, it will be all right although the direct observation can not be conducted between the mutual control point distances, but the locations of these points on the globe should be all right, if the locations (longitude and latitude) of these points on the globe are fixed. This method will not be affected by the vertical deviation.

3.2.3 Base Line Measurement

As it is necessary to know at least the length of a side as fundamentality in the triangulation survey, we must take the measure of the length of 5 to 10 km at a very high accuracy. However, this work takes much time and more expenses because of difficulties, and it is not much practise at present. It has been replaced with the direct measurement of a side length by the electro-magnetic wave distance measuring instrument.

3.2.4 Electro-Magnetic Wave Distance Measurement

It has become possible to determine a long side of scores of kilometers at some centimeters accuracy by taking the advantage of electric waves and light waves. This can replace the base line measurement and triangulation surveys.

a. Electric Wave Survey
The accuracy of this is a little inferior to others, but its reaching distance is long, and regardless of weather, or days or nights, observations are possible by way of it. It is important that legal controls of the utilization are different according to countries, and that its utilization is limited or prohibited there-in.

b. Light Wave Survey
Because of the use of light waves, its straightforwardness is strong and the accuracy is good, but the reaching distance is not so long as that of the electric wave distance measuring insturment. The reaching distance is short particularly in the day time, but it becomes possible to use it to cover up to 500 km even in the day time if the present LASER is adopted. But it is controlled by weather, when it is rainy of foggy. The usage of it will not be controlled legally.

3.2.5 Triangulation and Trilateration

This is the most fundamental method that determines the positions of each point by making the nets and chains of triangles and observing the inner angles by the transit. When the surveying scope is wide and the far distance with mountains and hills in it is open to the eyes, the accuracy and effectiveness is good. Recently, the trilateration has also been used which determines the positions by taking measure of the lengths of each side of the triangle instead of measuring the angles. The accuracy of this method is superior to the triangulation. In the case of the triangulation, it is necessary to get the approximate value of the height. A marker will be laid down at the triangulation point for permanent preservation. The standards of the triangulation are as follows:

ORDER	1	2	3	4
Average Side Length	60 km	8 to 15 km	4 km	2 km
Angle Measuring	1.5 to 2 seconds	4 seconds	8 seconds	20 seconds
Times of Measuring of One Angle	18 paris	6 pairs	3 pairs	2 pairs
Closure Error of Triangle	1 second	5 seconds	10 seconds	20 seconds

Of these, the 1st and 2nd triangulation points are the fundamental network of a national geodetic system, and have usually been established when the topographic survey is conducted. But if topographic or traffic conditions prevent the excution of the triangulation survey, some other methods such as the astronomical survey or something should be taken.
The 3rd triangulation point is also a part of national networks and a control point of the topographic survey, but this can be replaced by the aerial triangulation survey in photogrammetry. The 4th triangulation point is a control point for the investigation or project for the district utilization such as a cadastral survey or city planning, etc. However, it is not always good for the entire area, and it is not

necessarily needed for the topographic survey.

3.2.6 Traversing

This is a surveying method to determine the positions by taking the measures of the distances and angles between many points located along the route.
As it is a prerequisite to find a transportation method to reach each point one by one, it is effective in the case of leveled lands with disturbed views, which is not suitable for the triangulation survey but has a comparatively developed transportation means.
The distances between each point could not be taken the measure of so long in the past years. However, its effectiveness has developed because of the success in extending the distance from some kilometers to scores of kilometers with the utilization of the electro-magnetic wave surveying instrument.

3.2.7 Tide Observation

The standards of topographical mapping and others for the height of land are all based on the sea level, which changes often due to the tides, etc. The average which are obtained in the observations conducted in a period of time will be a level for the elevation. At least more than one year will be desirable, but the required accuracy may reduce it to about one month. The observation for this purpose is a tide observation. The connection of the average sea level in the observation with the leveling datum points will make the elevation of a bench mark. The datum face as the standard level will be an mean value of observation values in a long time generally, in other words, a mean level between the high tide and low tide.
The lowest tide face will be taken in most cases for hydrography, harbor or river works, etc. The difference between the mean sea level and this sometimes become 1 m to 2 m, and it is necessary to make clear which level as the datum was taken for the height measure. At the same time, the relations of this standard level with the local datum such as the lowest tide level and other should also be made clear.
Furthermore, this standard level is the sea level on the earth and not a leveled face. Therefore, this should be taken into consideration for a vast area.
In conducting the tide observation, rock-exposed and stable seashore will be required in order to take the measure of the difference between the control points fixed on the rocks and the sea level.

3.2.8 Direct Leveling

This is a surveying method to determine the differences of the heights of each point from the datum as the standard. It requires the most accurate height, and it take very much time because the measure should be taken of on the road one by one.
Recently, these conditions have been developed improved to some extent because automatic levels have been developed.
In conducting an accurate survey, it is advisable to use Inval wires for the staff because it is very high in accuracy and has the least flexibility in temperature changes. Much attention should be paid to as many a point as possible for the purpose of maintaining accuracy so that the distances between surveying points will be within 50 m. An accurate leveling makes, by forming a net, fundamental surveying nets of the country. In general, it has been completed before topographic surveying. When only topographic surveying is aimed at in the case of the

incompletion of the network, an accurate survey like direct leveling will not necessarily wanted. But taking into consideration its related projects, it is desirable to conduct an accurate leveling and lay down markers at level points for the permanent preservation. This should be done at least for the important areas. Accuracy in leveling will be determined by the instruments and staffs to be used in it, and it will be shown as below:

First $2\ \text{mm}\sqrt{S} \sim 3\ \text{mm}\sqrt{S}$

Second $5\ \text{mm}\sqrt{S} \sim 7.5\ \text{mm}\sqrt{S}$

Third $10\ \text{mm}\sqrt{S}$

Fourth $20\ \text{mm}\sqrt{S} \sim 50\ \text{mm}\sqrt{S}$

The error of leveling is in proportion to the square root of the measured distance S, that is, $\sqrt{S}$. In case of taking the leveling for a long distance, it will be impossible to get the accuracy at the same degree unless it is taken the measure of at a higher accuracy. It is widely practised to distribute the errors by making a closed loop of standard routes.

Of the above, the 1st and 2nd leveling are fundamental. The 3rd is taken for fundamental surveys of large scale local projects such as tunnels, dams, etc. The 4th is used in accordance with the accuracy which the purpose needs, within restricted objects like investigation work and the like.

3.2.9 Leveling of River Crossing

In taking the measure of the elevations on the both shores crossing rivers, valleys or seas, the foresights and backsights will be uneven distances and long distances by reason of topographic relations, and observation value will include observation errors, instrumental errors and those of refraction and also the ball-like shape of the earth. In order to cancel these errors and to get the elevation difference, a special observation method will be necessary. Like the case of the general leveling, is divided into a direct leveling and indirect leveling. Either of them will require many observations from both shores; for instance, the number of times of observations in accurate leveling are as follows:

Distances:	300 m	500 m	700 m	1,000 m	1,500 m	2,000 m	3,000 m	5,000 m
Number of observations:	1	1	2	4	8	13	25	57

3.2.10 Triangulation Leveling

This is a method to get elevation differences by taking the measure of elevation angles between two points by the transit. It is good if the distance betweeen two points quite long but the sight is all right. The efficiency of this method is better than the direct leveling but its accuracy is not so good. On the other hand its accuracy is good enough for control point of a topographical survey.

The triangulation leveling has the following errors;

a. Constant Errors by Refraction
   Its constant errors are large when the elevation difference between two points is small, and in the case of observations in which the sight lines are close to the ground.
b. Constant Errors by Ball-like Shape of the Earth
   These are the errors which were caused because the vertical directions between two points are not parallel.
   It is desirable to get the mean values by observing from the both shores in order to cancel the constant errors.
   It is limited that the distances between two points in triangulation leveling should be approximately 5 km. The accuracy available in this case is in a standard case about 0.2 m. The larger the distance is, the larger the error and it will be in proportion to about 1.5 powers of it.

Distances:	2 km	5 km	10 km	20 km
Errors	5 cm	20 cm	60 cm	160 cm

3.2.11 Barometric Leveling

This is a method to determine heights by the barometer, and it can be taken to control the heights in topographic surveys if they are conducted carefully in stabilized weather conditions.

In barometric leveling, the work should be checked and corrected back at the starting point, taking into consideration the changes in weather conditions. Its accuracy, when checked and corrected with much attention within the limits of about 2 hours, will be around 2 to 3 m, but if the weather conditions are not good, it will be 10 to 15 m in some cases.

4. WORK PROCESS OF TOPOGRAPHIC MAPPING

Work process of topographic mapping is divided as follows:

4.1 Planning and preparation work

1) Decision of the scale of the map, Contour intervals and the format of the map.
2) Decision of the time of the works, especially the time of aerial photography
3) Investigation of existing control points, maps and aerial photographs and the possibility to utilize them for the project
4) Legal steps (Procedures ?)

4.2 Aerial photograph, Air photosignal

1) Planning of aerial photography
   Decision of photographing height and position of runs, and selection of air bases, airplanes, aerial cameras and films etc.
2) Establishment of air photo signals
3) Enforcement of aerial photography

4) Photographic process, development and reproduction etc.
5) Inspection of photographs, preparation of the orientation map

4.3 Control Point Survey

1) Collection of data of existing control points
2) Planning of distribution of control points (for planning and for height)
3) Inquiry and decision of surveying method
4) Observation and computation
5) Establishment of monuments if it is necessary

4.4 Aerial Triangulation and Block Adjustment

1) Pricking of control point on a diapositive
2) Selection of pass points and tie points and pricking of them on a diapositive
3) a) Orientation and observation (analogue stereo-plotter-mechanical method)
   b) Observation and computation (Stereo-comparator-analytical method)
4) Computation of block adjustment

4.5 Photo-Interpretation and Field Reconnaissance

Before restitution, necessary photo-interpretation work and field reconnaissance shall be executed using the aerial photographs in order to make plotting work easier.

4.6 Plotting with Instruments

1) Writing down the result of photo-interpretation on the photograph
2) Orientation
3) Plotting

4.7 Editing

Details which are plotted on the draftmap with the instrument are rewritten suing proper symbols in conformity with the rule of expression and geographical name, boundaries height are written.

4.8 Field Check

The area where was not properly plotted during restitution work is supplementally surveyed in the field using the draft map.

4.9 Drafting, Reproduction

1) Drafting	a.	inking	(1)
	b.	scribing	(2)
2) Reproduction	a.	Photographic Reproduction	(1)
	b.	Blue Print	(2)
	c.	Printing	(3)

However work process mentioned above shows general order of work division in topographic mapping, control point survey and establishment of air-photo-signal can be done simultaneously. And work divisions indicated by roman numerical figure such as (1), (2) are Alternative each other.

The follow diagram of the work process of topographic mapping is as follows:

5. PLANNING OF AERIAL PHOTOGRAPHY

5.1 Photographing height

Flying height for aerial photography (during exposure) is determined by the scale of the aerial photograph. The scale of the aerial photograph is determined considering the map scale, air-planes, cameras, plotting instruments to be used.

Larger photo-scale means more photographs and more ground control points resulting uneconomy. If the photo-scale is to small, accuracy may not be satisfactory and interpretation becomes difficult.

It is known from the experience that there exist the following formula between the map scale 1/mk and the proper photo-scale 1/mb if the map scale is smaller than 1 : 2500

$$mb = a\sqrt{mk}$$

a is the constant of which value is about 200 ~ 250.

map scale and photo scale are shown in the table in case a = 250.

map scale	1/5000	1/10000	1/25000	1/50000
photoscale	1/18000	1/25000	1/40000	1/56000
enlargement ratio	3.6	2.5	1.6	1.1

It is general in practice that 4 ~ 6 times is adopted for the enlargement ratio (ratio-between map scale and photo-scale). From the interpreted point of view, it is preferable to avoid to use photographs of which scale is smaller than 1,: 80000

5.2 C-factor

In photogrammetric mapping, photographing height is also limited by the contour intervals required. It is known from the experience that there exist the following relation between the photographing height (above the reference plane) and the smallest contour interval Δ which can be plotted using the photograph.

$$H = c \cdot \Delta$$

Coefficient c is called c = factox. It is a variable of type of camera and plotting instrument used, topography and vegetation of the terrain and photographing condition etc. Among these influencing factors, type of instruments has at most predominant effect.

C-factor of instruments can be assamed as follows:

type of instrument			c-factor
Universal plotter	(1st order)		1200 ~ 1800
Precision plotter	(2nd order)	A class	1000 ~ 1500
Kelsh type plotter	(2nd order)	B class	800 ~ 1200
Multiplex type plotter	(3rd order)		500 ~ 800

For example, in order to plot two meter interval contour lines with a 2nd order instrument of which c-factor is 1200, photographing height H should be

$$H = 2\text{ m} \times 1200 = 2400$$

or lower.

Even if the photographing height is the same, photo-scale varies depending on the type of the camera used. For example, with a wide angel camera of which focal length is 8.8 cm, Photo-scales should be

0.15/2400 = 1/16000 6 or the wide angle camera.

0.088/2400 = 1/27000 6 or the super wide angle camera.

In case the surface of the terrain is covered by corn or grass and we can not measure the height of the terrain very accurately. Therefore it is better not to plot contour lines of which intervals are smaller than one meter by photogrammetic methods. If it is necessary, we have to execute terrestrial survey.

Vice versa, if photo scale and contour intervals are given the c-factor of the instrument is determined. For example.

Scale of the photograph is 1/40000 which is taken with a angle camera photographing height H is

$$H = 15 \text{ cm} \times 40000 = 6000 \text{ m}$$

To plot contour lines whose intervals are 5 m, c-factor of the instrument should be:

$$6000 \text{ m} \div 5 \text{ m} = 1200$$

5.3 Aerial camera.

After the selection of proper photographing height, selection of aerial cameras and airplane can be done. In mountainous area, however, deed angle behind the mountain increases and super wide angle camera loses its advantages.

In case that available camera and air-plane is restricted, photographing height should be determined considering these conditions.

Technical date (or performances ?) of some cameras are shown in the following table.

5.4 Air base.

The air base for aerial photographing should be as close to the photographing area as possible and should be the spot where it is convenient to catch the informations of metrological conditions of the photographing area. However freedom of choosing the air base usually not so big.

5.5 Choice of aerial film

The aerial film should be dimensionally stable and the emulsion should have a high resolution. Panchromatic emulsion is used for normal mapping purpose often. Infrared, colour or flase color films have advantages for the study of vegetation, environmental study and many other special purposes.

5.6 Flight design

The flight map shows the beginning, end and location of each flight line. For the flight map it is desirable to use the existing map the scale of which is approximately half of the photo-scale.

If proper existing map is not available, a small scale map (even 1/500000 scale map) or mosaic photo should be used. The area should be covered by parallel flight lines, since it usually result in the least number of flight lines and the least number of photographs to cover the area and we can expect better result of aerial triangulation except the configuration of the area is not suitable to cover by parallel flight lines.

The direction of flight is determined considering the configuration of the area and topographic conditions of the area such as mountaines and rivers and it is preferable

that the direction of flight is parallel to the direction of predominant wind of the area to keep drift of photographs as least as possible.
For the block adjustment of aerial triangulation, it is important .to have control strips crossing the group of runs perpendicularly.
Distribution of ground control points must be considered in flight design to achieve high accuracy in aerial triangulation.
Usually overlap and sidelap of photographs are 60% and 30% respectively. These percentages can be changed in order to cover a map sheet by the least number of stereo-pairs.
To avoid unacceptable gaps of photographs, 80~90% overlap and 40~50% sidelap is often adopted for mountainous area or under other special conditions
Aerial photographic mission is very much affected by the weather. Therefore the best season should be selected for aerial photography when we can expect many fine days. Flood season should be avoided for normal mapping purposes.

5.7 Air-photo signal (target)

The air-photo-signal is the target placed on the ground control station (such as a triangulation point, a traverse station, a bench mark) prior to aerial photography to identify the point on the photograph.
The target must be symmetrical and must be centered on the station. Shapes of targets can be square, cross or Y. It is emperically known that the minimum length of a side of the square shape target : 1 is given as follows:

$$1 \geqq mb/40{,}000 \text{ m} \quad (mb : \text{Scale factor})$$

In practice, Twice Large targets are used for safety.

(Table Target size)

Experience shows that white is the best colour for the target material.
Plywood board or polyfoam are excellent target material.
Air-photo-signal must be observed stereoscopically on the aerial photographs without obstruction. Therefore it is necessary to keep wide open ceiling above the target. If there is no open ceiling above the ground control point, the target must be established on the place from where air stations can be seen without obstruction and the ground coordinates of the target must be determined by eccentric survey.

5.8 Pricking

Control point identification can be accomplished by pricking existing natural or cultural features on the photograph instead of placing the air-photo-signal on the ground in the following case.

1) Near the ground control point, there is a clearly identifiable object on the photograph and the ground coordinates of the object can be determined by eccentric survey.
2) Placement of target is impossible prior to photography because of work process or other reasons.
3) Mapping is executed using existing photographs.

Pricking is a method to show the position of the ground control point on the photograph by a pin-hole. Pricking must be executed under exact stereoscopic observation. The accuracy of identification of the control point by pricking is less than that by air-photo-signal.

6. GROUND CONTROL POINT SURVEY

Photogrammetric mapping needs certain number of ground control points to determine the scale, azimuth and attachment to the datum of the photogrammetric plot. In the country where the basic control survey is not yet completed and even in the country where the basic survey is completed also, often existing ground control point are not sufficient enough to fulfill this requirement. In this chapter, ground control point survey for photogrammetic mapping is treated.

6.1 Distribution (or configuration) of ground control points

The configuration of ground control points for topographic mapping should be determined considering the size of the mapping area and the scale of the map. More than three ground control points are required for absolute orientation of a model. It is not practical from the economical and time consuming points of view to establish all three ground control points by field survey. It is why in many cases aerial triangulation is carried out to establish control points for absolute orientation of a model.

For the adjustments of aerial triangulation, there must be certain number of ground control points in a strip, for example 3 points is the beginning of the strip, one at the center and two at the end, totally 7 points in one strip.

In order to reduce the amount of field of ground control survey (or the number of newly established ground control points), existing control points must be utilized as much as possible and it is desirable that ground control point locates in a side – lapping zone of two adjacent strip since it can be used for adjustment of both strips. It is desirable to establish air – photo signals at these ground control points for easy identification of the points.

6.2 The accuracy of ground control points for photogrammetric plotting

The accuracy of ground control points for photogrammetric plotting should be properly determined considering (or matched to) the scale, the purpose and the accuracy of the map.

Since the unnecessary high accuracy results more money (high cost) more time (long period), accuracy should be determined just to satisfy (fulfill) the requirement, if there is no other purpose.

The tolerance limits of ground control points for photogrammetric mapping is as followed:

1. Plainimetric accuracy

   It is emperically given that the standard deviation m.p. of the planimetric coordinates of ground points for photogrammetric mapping is

$$mp = \frac{0.5}{\sqrt{Mk}} = 0.5\sqrt{mk}\ \text{(cm)}$$

   where $Mk = \frac{1}{mk}$ is the scale of the map.

2) Height

   The accuracy requirement of height control point is mainly related with contour interval Δ H

   The standard deviation (mh) of height is emperically given by the following formula:

$$mh = 0.1 \Delta H \sim 0.3 \Delta H$$

Some example is given in the table.

7 AERIAL – TRIANGULATION AND BLOCK ADJUSTMENT

7.1 Aerial triangulation

For plotting, it is necessary to have three control points whose terrerin coordinates are known in one model. If we establish these control points by field survey, cost and time are not acceptable. Time and cost can be reduced by aerial triangulation since aerial triangulation can be done at a office. Successive photographs which a 6 per cent forward overlap are used for aerial triangulation.

Relative position of two adjacent photographs are determined by relative orientation and repeated operations of bridging models reconstruct strip of models. The coordinates of this strip can be transfered to the ground coordinates and the ground coordinates of pass points can be obtained. This transformation can be done if there are more than three ground control points whose strip coordinates are known as well as their ground coordinates. Since aerial triangulation is principally the method of resection in space, certain errors of systematic and random characters are arising and errors in strip coordinates increases rapidly with the number of models. Therefore 20 is the practical limit of the number of models in one strip. There are two methods of aerial triangulation. One is the mechanical method using a first order instrument and the other is the analytical method. In the analytical aerial triangulation, photo-coordinates of a point are measured using a stereocomparator and measured photo-coordinates are transfered to ground coordinates by computation using an electronic-computor.

Generally the analytical method is more efficient than the mechanical method. However, experience shows that the accuracies obtained by both methods are almost the same. In the case that the strip consists of 10 models and the photo-scale is 1/10,000, the expected accuracy is such: standard deviation of 0.5 m and maximum error of 1.0 m.

Better accuracy can be expected if the strip is shorter and the photo-scale is larger. However there is a limit of accuracy what can be expected even if very large scale photographs are used. Pass points established by aerial triangulation should be pricked on the photographs or clearly sketched for easy identification in plotting. Accuracy of aerial triangulation mainly depends on the following factors, the scale of photograph, number of models, number of ground control points used for adjustment and their distribution, and the method of identification of ground control points i.g. by air-photo-signal or by pricking in the field.

7.2 Block Adjustment

Block adjustment is very efficient for aerial triangulation of large area. The strips are formed into block by tie points and simultaneous transformation of the block coordinates into the ground coordinates is carried out. If the area is too large to adjust simultaneously, the area can be divided into several sub-blocks of suitable size, i.g. sub-blocks of 200–500 models. There are two method of block-adjustment such as the analogue method and the analytical method. Since the analytical method is usually more efficient and more accurate than the analogue method, it must be tried to adopt a analytical method. Block adjustment is very efficient especially for the large area with a small number available ground control points.

8. PHOTO-INTERPRETATION

Most terrain feature can surveyed in the office by photo-interpretation without field work and cost and time of field work can be saved very much.
Possibility of photo-interpretation of each terrain feature is listed in the following table. Survey of a terrain feature, photo-interpretation of which is difficult or impossible must be executed by field work or based on the other reliable data. And also sampling check in the field is necessary even for the terrain feature, photo-interpretation of which is possible in order to make a correspondence between the image on the photograph and its object on the terrain.

9. FIELD RECONNAISSANCE

Field reconnaissance is work done before plotting to collect necessary and useful information for plotting and compilation in the field. In advance to field work, photographs should be studied carefully in order to find obscured object on the photographs, object covered by trees and object of which size should be measured such as width of a road and a canal or height of a building etc. Positions of such object should be marked on the photographs. Also information about geographic names and boundaries should be collected before field reconnaissance and necessary information must be noted on the photograph which is used for field work. It is not necessary to identify directly in the field all object that are unidentified in the previous photo-interpretation work. Sampling method can be adopted.
Generally speaking, 80% of items of symbols can be identified on the photographs. Of course, some of them can be interpreted easily and some of them are difficult to identify. As an acerage, 80% of object of each items can be identified correctly. Therefore 64% of all object what should be plotted on the map, can be identified by photo-interpretation and remaining 36% of object should be surveyed in the field. Before entering the field, survey routes and order of work must be planned using photographs. The results of field reconnaissance must be properly arranged and written in coloured ink on the photograph in order to make the operator understand correctly who is not familiar with the area.

10. STEREOPLOTTING

10.1 Plotting with the stereoplotters

Topographic map details are plotted using a stereoplotter. There are many types of plotters from very precise one to very simple one.
The proper type of plotter should be selected considering the scale of photographs can be used and the number of photographs and ground control points can be reduced, it is more economical to use the precise instrument than to use the less precise instrument.
It is not necessary that the plotting scale is equal to the scale of the final map. It is

possible to plot with larger scale and to reduce it photographically after complication. Accuracy of the map may improve with this method. However additional photographic process is necessary.
Stable and reliable material should be used for the plotting sheet such as the polyester film or the aluminum foiled paper. Recently the direct scribing method becomes familiar more and more.
By direct scribing method, details and contours are scribed on the coated film directly with the instrument.

10.2 Selection of the plotting instrument

Stereo-plotting instrument are divided into three classes: 1st class, 2nd class and 3rd class according to their accuracy.
1st class instrument is the most precise instrument in which the plotting is carried out with original size diapositives and the universal instrument with which aerial triangulation and cross section survey can be executed. The c-factor of the 1st class instrument is 1200–1800 and enlargement ratio between photo scale and plotting scale is 6–10. It is mainly used for large scale plotting. Also in the 2nd class instrument, original size diapositives are used. However, aerial triangulation can not be method and the 2nd class instrument expect by independent model method and the 2nd class instruments are used for precise plotting. Further the 2nd class instruments are classified into 2 classes namely A class and B class according to their accuracy. The c-factor of the 2nd A class instruments is 1,000–1,500 and the plotting can be done with up to 8 times the scale of the original photograph. The 2nd A class instruments are mainly used for medium and large scale plotting. The c-factor of the 2nd B class is 800–1200 and the acceptable maximum plotting scale is 6 time the scale of the original photographs. The 2nd – B class instruments are mainly used for the medium scale plotting. The plotting is executed with twice the scale of the original photographs in the 3rd class instrument. The 3rd class instrument are not accurate as 1st class and 2nd class instruments and are used for the small scale plotting.
Main instruments are shown in the following table.

10.3 Symbols. (legend)

The accuracy of the map and the details that is represented on the scale of the map. In order to unify the map representation, the legend should be established previously. In the legend, both the kinds of objects that should be represented on the map and the way how to show the objects, should be specified. The minimum object should be selected to fulfill the purpose of the map. Too many details on the map results a too complicated map. It is not only uneconomical wasting time and money in production, but also not convenient in use.

1) Marginal description
   The name of the map, the scale of the map, contour intervals, the longitude and the latitude of the corner of the map, projection system, the date of production, the organization which produced the map. The aerial photographs used, legend.
2) details
   a. road, rail way
   b. building, factory
   c. vegetation

   d. *object*
   e. river, lake, bay
   f. ground control point
3) geographic name
   a. *administrative name*
   b. name of a natural object (mountain, river, sea, lake, bay, etc)
   c. name of a building
   d. name of road
   e. name of a village
4) topography
   a. contour lines
   b. isobathic contour lines
   c. volcano
   d. distorted surface area
5) land use
   a. cultivated area
   b. forest
   c. town and village
   d. factory
   e. mine
   f. harbour

11. EDITING

The draft map plotted with a stereoplotter should be edited referring to the results of field reconnaissance. Considering the purpose of the map, objects should be selected and represented with the symbols of the legend. Usually pencils and color pencils are used in the editing work. Only long experienced technicians can select and represent objects properly, especially in the case of small and medium scale map compilation. The final draft map is completed by the compilation work. The compilation work is not yet automated and done by skilled technicians. At times color separation is executed in the compilation work. In this case, lines and patterns which are represented by the same color on the map, are compiled on the same polyester sheet and a set of several polyester sheets is the final draft map.

12. FIELD CHECK

In order to inspect the map, objects on the map are selected by sampling method and their shapes on the map are compared with the shapes of the object on the terrain in the field.

If a part of the area is hidden by clouds and mapping of that area can not be executed or details have been changed very much after taking photographs, in order to complete the map, completion survey should be carried out by plane table method.

The first step of the field check, is to conduct an investigation into the representation of details, symbols and letters by comparing the objects in the map with the corresponding objects on the terrain. And then inspection of the map of the sampled area is carried out and if it is necessary the completion survey is excuted. The results of the completion survey should be drawn in ink.

13. DRAFTING AND REPRODUCTION

13.1 Drafting

The drafting is the work to draw finally the marginal description, lines and symbols of the final draft map after completion survey in accordance with the legend.
The drafting is the final process of map production, but it is not yet automated and still it is executed by skillful draftment. The quality of the map greatly depends on the skill of the draftman especially in the case of small scale maps. Hitherto the inking method is used and recently the scribing method becomes predominant.
The scribing is the method to cut the opaque material coated on the transparent plastic base using a needle and to make a negative.

13.2 Reproduction

Several methods are developed to reproduce maps out of drafts. Photo-copy is rather expensive, but it is easy and very dear copy can be produced. Therefore photo-copy method is suitable for the reproduction of a few number of sheets of the map.
If several tens of sheets are required, blue prints using scribed draft is most economical. Recently electronic copying instruments, such like zerox, becomes utilized.
If more than 500 sheets of a map are necessary, printing is the most effective method. Lines of a printed map are clear and fine. Usually the negative-positive process is used in printing plate making. Therefore in order to make a printing plate, at first the negatives is made by photographing the positives – the original draft – and these negatives are copied on zinc plates or p.s. plates (pre-sensitized plate). These printing plates are used for printing with a off-set printing machine.
Zinc plates can be used several times if they are kept carefully and each time sensitive materials should be coated. The pre-sensitized plate can be used only once. However, since sensitive materials are coated previously, time can be saved and recently it is employed widely.
The processes of plate making and printing are quite different from the processes of photogrammetric work and then printing should be carried out in a special factory by experts separated from the photogrammetric section.

14. SCHEDULING

14.1 Basis for Scheduling

The number of days required for completion of the mapping work can be obtained from the amount and standard efficiency of individual works. If the amount of work is large and calls for extension of the work period, then the operation parties should be reinforced and additional instrumental and equipment be provided in order that the whole work will be completed within the predetermined period of time.
The work schedule is prepared on the basis of individual plans for such personnel and equipment reinforcement.

A suitable interval should be provided between respective processes so that no loss time resulting from the suspension of work flow nor overflow of work will be caused.

In planning the work schedule, due account must be taken of the nature of individual works listed below.

1) Works that should be given priority over others.
2) Works that cannot be started until completion of others or that can be implemented in parallel with other works.
3) Works which can be readily carried on from one to the next stage in each of the districts into which the survey area is divided, and works which reject such arrangement.
4) Works like photographing and field reconnaissance which call for consideration of the seasonal effect.

Mapping work is generally performed in the following sequence.

Photographing ---→ Field Reconnaissance → Plotting → Compilation → Field Check → Scribing → Reproduction

Control Point Survey ---→ Aerial Triangulation → Plotting

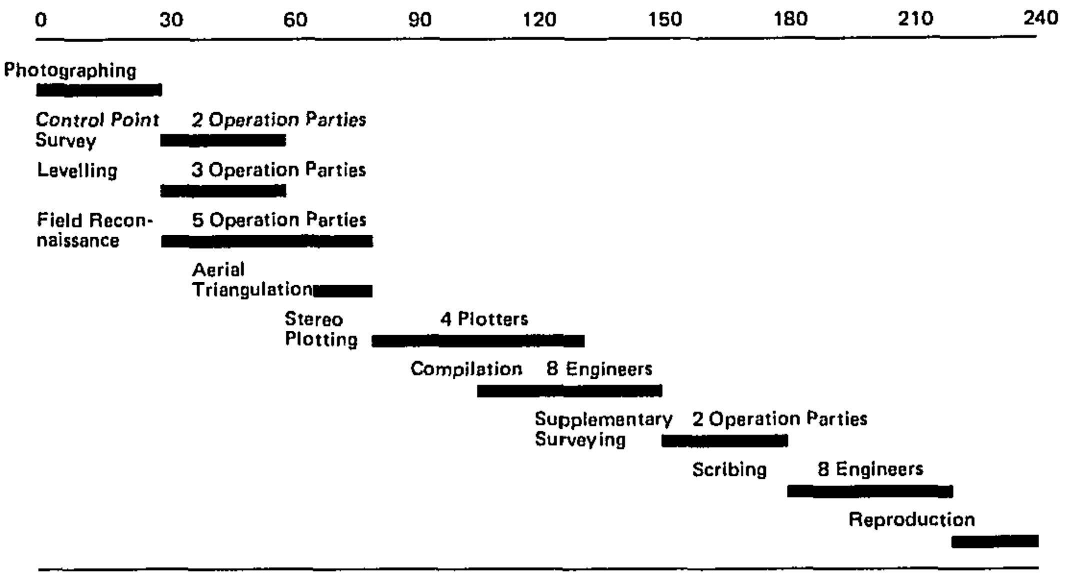
In the case of the above flow chart, photographing work can be performed in parallel with control survey and field reconnaissance with aerial triangulation.

Division of the survey area into a number of districts makes it rather difficult to proceed from aerial triangulation to the next stage, i.e., plotting, because of the nature of actual works involved in the two stages. However, progress from plotting to compilation is not heavily impeded by such division of the survey area.

A work schedule prepared with consideration given to these factors is shown below.

Work Schedule for Preparation of
1/10,000 Topographic Map Covering Model Area (500 km^2)

Total number of days (cumulative)



Since the photographing work is occasionally retarded to an extreme extent by weather condition, its schedule should be planned carefully without account taken of the possible delay.

14.2 Standard Work Efficiency

The standard efficiency of work is something that should be evaluated empirically from the past accumulation of experiences. Ordinarily, it is evaluated for each kind of work, topography and scale.

For works of routine and fixed type which can be classified with relative ease into a number of stages, the standard efficiency can be readily established. In the case of works of other types like triangulation which is largely affected by topography and vegetation, the efficiency cannot be established so easily.

The standard efficiency of major works is given below in approximate values.

A. Office Work

1) Aerial Triangulation

0.7 day/model by a party consisting of 3 engineers using pricking device, comparator and electronic computer.

2) Stereo Plotting and Editing Work

1/5,000 map	0.5 ~ 1.0 km^2/day
1/10,000 map	1.2 ~ 2.0 km^2/day
1/50,000 map	5.0 ~ 10.0 km^2/day
Operation party	2 engineers using precision plotter.

3) Scribing

1/5,000 map	1.5 ~ 2.5 km^2/day
1/10,000 map	3.0 ~ 5.0 km^2/day
1/50,000 map	20.0 ~ 30.0 km^2/day
Operation party	1 engineer

B. Field Work

1) Triangulation (side length = approx. 5 km)

5 ~ 7 days/point

Operation party 2 engineers and 3 or more labourers, using theodolite.

2) Traversing (side length = 2 ~ 10 km; electromagnetic distance meter used)

2 ~ 5 km/day

Operation party 3 engineers and 3 or more labourers, using theodolite and electromagnetic distance measuring instrument.

3) Leveling (3rd order)

3.5 ~ 1.5 km/day

Operation party 2 engineers with 2 or more labourers, using level.

The values shown above are the approximate efficiency calculated with consideration given to the planning, incidental works and a suitable number of resting days for each kind of work.

15. COST ESTIMATION

15.1 Basis of Calculation

The cost of mapping work consists of the direct cost which is composed of the direct personnel cost required for workers engaged in the actual mapping work, depreciation expense of instrument and equipment, and incidental expenses, and the indirect cost which is comprised of the depreciation expense of buildings, cost for training engineers, management cost of the contracting company, and welfare expenses for the company's employees.

The cost is briefly classified below for easier understanding.

A. Direct Cost
   1) Direct personnel cost
   2) Incidental site expenses
   3) Depreciation expense of instrument and equipment

B. Indirect Cost
   1) Indirect cost

While the direct cost can be calculated directly from the required numbers of workers, instrument and equipment, and days, the indirect cost is obtained by multiplying the sum total of other costs by a relevant factor.

The standard numbers of required workers, days, and instrument and equipment should be assorted and classified according to the scale and topography. In calculating the cost of field reconnaissance, due attention should be directed to the effect of topography, vegetation, and traffic condition. Attention should also be directed to the weather condition which affects the photographing work to an extreme extent.

15.2 Effect of Operation Condition of Cost

Since the efficiency values are largely affected by topography, landform and amount of work, the following factors should be taken into consideration when calculating them.

A. Area and Shape

   The unit cost for mapping a same place at a same scale varies if the total area to be covered changes. The cost does not necessarily increase ten-fold even if the total mapped area is increased ten times.

   The cost also varies by the shape of the area such as square and rectangle.

   If mapping is required not for one but a number of areas, the costs for respective areas are summed up.

B. Scale

   In the total cost of photographing or mapping work, the travelling expenses to and from the survey area are fixed and not affected by the acreage of mapped area or scale, but the numbers of photos and models increase in proportion to the mapped area and to the square of photo scale.

   The cost of photographing work is usually proportionate to the photo scale. The reason is that total length of flight lines is approximately proportional to the photo scale because the length of each flight line is irrelevant to the scale and only the interval between flight lines is inversely proportional to the scale.

   Thus, the mapping cost can be divided into the constant portion which has nothing to do with the scale and the portions which is proportionate to the scale or its square.

   In a general way, it can be said that about 60% of the mapping cost is proportionate to the scale, about 35% to its square, and the remaining 5% is constant and not affected by the scale.

C. Contour Interval

If the contour interval is small, orientation and measurement call for a higher accuracy and the correspondingly detailed compilation is usually required. Accordingly, if the contour interval is decreased to half, the cost increases by about 10 to 20%.

D. Scale and Kind of Photograph

In any of the above-mentioned cases, the cost can be cut down if the scale is reduced because this will result in a smaller number of photos and less density of control.

Other factors to be considered in cost estimation include the selection of flight course and timing of photographing (public facilities, vegetation and other natural features are subejct to considerable secular change after photographing).

II. OPERATION

OUTLINE: This chapter will deal with the concrete, field plan, organization, equipment and tools for use, standard days required, work methods, accuracy, etc. An explanation will be given, for instance, with the case of 1/50,000 mapping for approximately 16,800 km^2 of the Barito river valley in the south Kalimantan Province, Indonesia.

Its data is as follows:

Scales			1/50,000
Contour interval			25 m
Accuracy of horizontal locations on map			± 0.5 mm
Plotting area			16,800 km^2
Number of sheets			28 pieces
Size of sheets:	length	about 55.3 cm	(latititude 15')
	width	about 55.5 cm	(longitude 15')

I. OUTLINE OF LANDS TO SURVEY

The area of the present topographic mapping operations covered at first about 100 km from east to west between the Maratus mountains and the Barito river; southern parts of the Kalimantan province of about 200 km^2 from north to south up to the northern hilly lands from the cost; and the central part of Kalimantan. But it has amounted to 20,000 km^2 in all, being addied to with 3,000 km^2 of the 17,000 km^2 area, east of the Kapoeas river including Banjarmasin. The topography and conditions of the whole area will be given in the following sections:

1.1 Topography

In general, the topography may be classified into mountainous districts, hilly lands, tablelands and lowlands. The topography of these are equivalent, geologically, to Mesozoic formation (Jurassic and chalk), Tertiary formation, diluvial formation and alluvial formation. Generally speaking, each formation before the diluvial formation has folded structures with an axis approximately from north-south-east to south-south-west, and the uplifts after erosion are exposed in the mountainous districts.

a. Mountainous Districts

The Meratus mountains extend toward the south-west, and are about 1,000 m above the sea level in the north but below 500 m where going down toward the south. The weathered soil of the mountainous districts is thick laterite but barerocks of cliffs of lime stone are exposed to view.

b. Hilly Lands

The hilly lands are distributed widely between the mountains and lowlands, and to the north of the lands to survey. Most of them are approximately 50 to 100 m above the sea levél, and the line of the mountain tops show undulation. The valleys cutting the line are very small and many, and weathered soil is red lateritic in most cases.

Hilly lands are distributed among mountain valleys too. But these are developed erosion of mountains, and the creases of their topography are rough, unlike the former are not.

c. Tablelands

Tablelands develop widely in the area from Pelaihari to Banjar-Baru. And also there lie well-developed tablelands in the area from Amuntai to Tanjung; in the central of the Kalimantan Province on the west of Tanjung. Either of them is a diluvial tableland, and is below 30 m below the sea level. Most of them may be classified into an upper or lower one. The upper one has been lateritic, and there are many places which consist of gravely sand and clay. There are terrain features tableland-shaped terrain features in the basin among mountains.

d. Lowlands

The lowlands are divided into an alluvial fan, a flood plain and delta. The alluvial fan develops widely in the vicinity of Rantou, Kandangan and Barabai. It is made of sand mixed with gravel, and descends slowly from east to west. The flood plain extends on the north of Mengatip in the upper valley of the Barito river, and in the area of the Negara river, it goes down near Negara goes down along the river to the vicinity of Negara.

1.2 Identification

It seems that natural levees which will not contain water even in the rainy season, but it can not be made certain because this area is covered by forests. The down stream area of the plain is submerged at the delta districts except the sand bars near the coast. This is a so-called swamp. The lands behind the sand bars, old river courses, the parts between the flood plain along the Negara river and the alluvial bar area of Barabai and Kandangan are wet through the year because of bad drainage.

a. Forests

The forests in this area are generally classified into riparian forests, swampy forests, lowland forests, hill forests, mountain forests, residential forests, etc.

Sea water affects as far as Marbahan, a point in the of 70 km from the mouth of the Barito river, and in this area riparian forests reaching 20 m in height are distributed. In the up stream area beyond this forests of 5 to 20 m in height are open to the eyes.

Swampy forests of 30 m in height and secondary forests of below 15 m in height consisting primarily of galam which is used as firewood or cabin-building wood by the inhabitants come into sight up to the points of about 30 to 40 km ahead toward the land in the east from the Barito river.

And swampy coppice forests and swampy bushlands are also seen in this area. The latter occupys a larger part of the area from Negara to Amuntai.

The settlements of towns and villages are developing along the trunk road from Banjarmasin to Tanjoeng behind these swampy forests; and residential forests, coconut palm plantations, rubber plantations are scattered over these districts, and lowland forests, where coppice grows, come into sight.

Alang-Alang grasslands on the fire-burned fields in further eastward tablelands and hilly coppice forests are distributed widely, and hill forests, regarded as genuine tropical rain forest, and mountain forests of 40 to 50 m in height grow in the Meratus mountain areas.

b. Grasslands

Grasslands are widely distributed in either of mountainous areas, hilly lands, tablelands and lowlands. Tall ones are called Alang-Alang. Most of the grasslands were the lands which had been involved in a field fire and had changed grassy later. They are widely distributed near the settlements but a little distributed in the north. The grasslands in the lowlands are wet and exuberate in the dry season, but it seems that they will be submerged and be dried up in the rainy season.

c. Paddy Fields

Irrigation channels can hardly come into sight in the paddy fields. Channels are made use of as drainages. The supply of irrigation water is dependent upon rainfalls and flooded rivers. Many paddy fields are in the delta and alluvial fan areas of the lowlands, but small scale ones are also distributed in the valleys of the mountainous areas. Irrigation is conducted in a large scale by way of utilizing tidal motion in the fields under the development project of the Government in the vicinity of the Barito river. This method is called a tidal irrigation. The whole area of flooded lands will be changing into paddy fields in the mountainous areas because water remains there, and they can not be distinguished from grasslands, only judging from their appearance.

d. Croplands

Croplands are many in tablelands and hilly lands; and can also be seen at the foots of mountains and in lowlands, but most of them are fire-burned fields. However, they will be given up after being used once in most cases. Therefore, most of them can not be distinguished from grasslands, only from the appearance.

e. Orchards

Most of large scale orchards are coconut palm fields. There are growth lands in the vicinity of Banjarmasin, along the net-shaped channels in the alluvial fans, along the road between Amuntai and Tanjung, in the coast, along the rivers such as the Barito main river and the Negara river.

f. Plantations

There are two kinds of plantations such as those that had been existed before the Independence and those that were developed after the Independence, but rubber plantations are overwhelmingly more than the others. However, rubber plantations are being converted into those of plants such as coconut palm, coffee and perfume, because the collection of rubber is a little at present. And small scale rubber plantations are seen in lowlands or hilly lands, but it is difficult to distinguish them from forests on the maps because they are mixed with coppice.

g. Settlements and Graveyards

In general, settlements are arranged along the roads and rivers, and a type of settlements found numerous is the form of village collection.

Graveyards are classified into the Islam type and the military one, and their area is small in general. Christian grave yards are in the compound of individual.

h. Other Remarks

There lie oilfield zones in Tanjen as their center in the north, and there are nets of roads to convey oils along the Negara which connects the zones. There are air-fields in Panjor-Baru and Tanjung which have golf courses and other recreation facilities with them.

The Riam-Kanan dam has a hydraulic power station, from which power *transmission lines extend toward major cities and towns.*

1.3 Facilities

a. Facilities for Roads

The national highway from Banjarmasin as its starting point ot Tanjun through Martapura, Rantau, Kandangan and Amuntai is a major road which is 6 to 8 m in width. Beside the above, there are main roads such as from Banjar-Baru to Pelaihari; from Amuntai and Tanjung toward the central part of Kalimantan; from Rantau and Kandagan to the Negara Province. The national highway will not be submerged even in the rainy season, but other ones will be otherwise. Some of the bridges over the rivers are made of concrete but most of them are wooden make.

b. Facilities for Rivers

There run narrow channels of combination use for draingage too in the town area of Banjarmasin and along the roads connecting rivers and the national highway.

There are anchorage places on the rivers, which are made of raft.

As regards dams, the dam under construction by assistance of the Government of Japan only will store the water of 13 thousand million tons.

c. Religious Facilities

Islam temples are in a great number, and there is one temple at least in each settlement. Larger ones have two or three in general.

Christian churches are found in a small number in Banjaru-Baru, etc.

d. Administrative Facilities

Beside the branch office of the central government in Banjarmasin, a sub-branch office of Department of P.U.T.L. is placed in each town where a provincial government is located.

A facility of the Pertamina is located in every oilfied zone, and public service facilities such as post offices, hospitals, etc. are established in a large settlement. The whole settlement has at least one school, and in towns, there are senior high schools.

2. OUTLINE OF WORK

A work plan was set up as follows, taking into considration the general conditions stated above and another circumstances regarded as necessary:

1. It made a general principle to undertake some parts of the work economically within one fiscal year; the whole volume of work was divided into three lots during the period from 1971 to 1973; the 1971 fiscal year covers photo-taking, the 1972 fiscal year control points survey, and the final year plotting and field check. Each of the lots finishes as exactly as planned in the fiscal year involved.
2. The field work was in every process made as little as possible because the traffic conditions in the areas under survey have not fully developed yet, except for the main roads. Therefore, it made us take as much as use as possible of photo-interpretation, the field check for plotting being put at minimum. And the triangulation was replaced by astronomic survey. The traffic conditions were one of the main reasons why the aerial signal had been replaced by the aerial photo pricking.
3. The photo-taking was conducted in the dry season, avoiding the rainy season, and it was to finish in the dry season of the 1971 fiscal year. The pricking work was conducted, while sufficient time had not been given in establishing the aerial signal beforehand. The real photo-taking had not been completed until December, 1972 because of the weather conditions or smokes of burned fields
4. Control points on the ground are located in small numbers in the Maratas mountains on the east of the areas, and the rest are the astronomic survey points which are scattered on the coast by the Barito river and in the vicinity of the delta areas. The necessary number of control points at minimum was established additionally by astronomic surveying, furthermore, necessary control points for plotting were established through the use of phtogrammetry by aerial triangulation and block adjustment. This is a minimum necessity measure, and minimum conditions were fulfilled in order to make topographic maps of 1/50,000. Therefore, this method was not adopted for more than that. This is the reason why the astronomic survey was utilized instead of the triangulation or polyangulation as follows:
   (a) There were limits of budgets and time.
   (b) It was too large for the triangulation because the distances between the points were wide in huge areas.
   (c) The intervisibility was difficult and was not good for the triangulation, because the topography is horizontal and covered by jungles, except some parts of the east and north.
   (d) Difficulties of the selection of suitable points arose due to bad traffic and transportation even at good lands in topography which are suitable for the triangulation and polyangulation.
5. In connection with the elevation too, the density of the standard points were not sufficient like the triangular points. Particularly, the mean sea level is not established yet, which is to be the standard for the elevation. Therefore, a temporary tidal station was set up on the coast of Kakisong and the sea level was determined by which standard level surveying was conducted with accuracy enough to apply it to river projects and investigation later.
6. Aerial triangulation and block adjustment were conducted in Tokyo. The aerial

triangulation was made by the analytic method, and as regard the block adjustment, the whole area was divided into three blocks for the work. The number of models is about 800 in all.

7. The plotting work was made separately in Tokyo and Djakarta. The plotting work in Tokyo was on the area of about 17,000 km, the photos of which was taken with a super-wide camera for the first time in 1971. The Djakarta plotting work was based on the wide-angle photos taken in 1968 which had been planned and supplemented later. This was for the area of about 3,000 km in the delta area of the Barito river.
8. The plan of plotting was based on the plan of topographic maps of Indonesia.
9. For the names of places and boundaries, the documents which the Government of Indonesia had prepared were referred to.
10. The Scribing method was adopted for drafting, and the maps were five-colored.

3. AERIAL PHOTO TAKING

Prior to the control points survey and plotting works, the taking of aerial photos was conducted during a period from May to December. The photo-maps, which were made for the above photos by uncontrolled mozaics, were taken the best use of as a master plan in the case of control points survey, field investigation and plotting works. The outline of the works are as follows: (Aerial signals were not established, and in the case of field investigation,the prickings were conducted.)

1. Photo-Taking
   a. area for taking — 16,800 km^2
   b. distance of taking — 3,300 km in all
   c. strips and number of taking — 6 strips, 140 pieces
   Crossing strips were selected so that they would include as many a control point on the ground as possible, and also they would directly contact the normal strips. For this purpose, one of the strips became toward north-east south-west of the national highway where there are many control points, but the other five strips faced approximately toward east-west.
   strips for plotting (north-west) — 16 strips, 750 pieces
   total — 22 strips, 890 pieces
   d. standards of photo-taking

scale:	1/50,000		
altitude:	4,500 m ± 450 (± 10%)		
overlap and side lap:			
overlap:	field	60%	within ± 5%
	mountain	80%	within ± 5%
sidelap:		30%	within ± 5%
shifting of strips		within 5% of taking	
gradient:		within 15%	

: at average 5% on five photos taken in succession

time of taking : within the time zone when the sun is 30° above the horizon.

e. photo-taking instruments

cameras: cameras, the calibration of which were adjusted with the past two years by super-wide angle. And RC9 Super Aviogon, f = 88 mm, frame fullmat 23 x 23 cm, was used in the Barito valley.

filter: yellow filter (450 to 500) was used for the stressing of the contrast.

airplane: Cessna 402 type

film: Panchro-film for stabilized aerial Photo: Kodak plus-t Aerographic film 2402 (Estar vase)

f. Kinds of finished articles:

positive film	1
contact printed photo	2
index map	1

g. number of days for photo-taking: 12 days (actual days taken)

It took a period of 200 days in order to keep 12 actual working days because of weathers or other conditions.

h. expenses:

for each 16,800 km (in this case)	US$88,200
Fees for contact printing	3,000
unit cost per k km^2	about US$5.5 ≑ 2,000

4. PRICKING

a. Volume of Work

Newly established control points (astronomic-surveyed points):	8
Existing control points	13
Newly established standard points (within plotting scope)	50

b. Pricking Method

Comparing the field the aerial photos, the locations of points for pricking should be made sure of. Holes will be made on the photo with a needle by stereoscopic viewing of details. The diameter of a hole should be within 0.2 mm.

c. Pricking of Excentric Point

When it is difficult to catch the locations of control points because they lie at such featureless places as in the woods, etc. the location which is clear should be pricked beforehand, and the relations between the point and control point should be measured. In the case of topographic survey, its surveying accuracy will be about 1 m.

d. Instruments and Tools compass with legs. (for excentric points survey)

1. Aerial symbols (for reference)

The work should have been finished before photo-taking, when the pricking method is not used, but the aerial signal establishment is taken for use.

a. Methods and Materials.
   An equipment will be made with pieces of boards, which were cut according to thr standard sizes, or other materials, so that control points should be fallen upon the aerial photo. The kinds and shapes are a cross type, three-direction type, etc. The rectangular signal boards are placed in a radial way around the point as their center. The surfaces of the boards should be white-colored. The equipment should be kept with stakes at 30 to 50 cm above the ground to set it up horizontally.
b. Sizes of Signals
   The size of one side of the signals at centimeter unit will be around 1/300 X 1/800 of the denominator of the photoscale. For instance, in the case of phototaking in 1/50,000 photoscale, it should be 50,000/300 X 50,000/800, that is, approximately 170 cm X 65 cm. If the radially-arranged boards make the whole size of the signal around 4.0 m to 4.5 m, it will be understood that attention has been paid that the size of the image be 80 micron to 90 micron.
c. Members of Work Party

engineer	1
assistant	1
labors (denpend on situations)	some
vehicle	1

d. Kinds of Achievements
   Description of aerial signal points, description of pricking point and pricked photos
e. Volume of Work per Day
   About two points for aerial signal establishment and pricking
f. Expenses per Point ¥35,000

5. CONTROL POINT SURVEY

There are about 10 triangulation points established already along the national highway running in the eastern part of the survey area and about five astronomic points along the Barito. It was therefore determined to prick these existing control points on the aerial photograph, and to newly establish a minimum of eight required points by astronomic survey with account taken to topography, traffic condition and line of sight.

Further, in consideration of the future development of the Barito river basin, precise levelling was conducted over a distance of about 300 km to Tandjoeng, with pricking performed on the aerial photograph. In order to establish the elevation of the reference plane for the precise levelling, provisional tide measurement along

Takisoeng Talok coast was performed for a period of approximately one year.
At the eight astronomic points and the 61 bench marks established along the national highway at intervals of about 6 km, concrete monuments were erected.
The control point survey was conducted from August to December 1972 as outlined below, except that the tide measurement was continued until October 1973 though intermittently.

5.1 Astronomic survey

A) Outline

Amount of Work :	8 astronomic points.
Accuracy Required for Plotting:	± 100 m (±0.2 mm on the photograph).
Purpose :	Establishment of control points for block adjustment (horizontal accuracy correction) and triangulation.
Observation Method :	Simple astrolabe method using an Ni 2 automatic level with attachments.
Number of Observations :	3 ~ 5 sets.
Number of Observation Stars:	14 ~ 16/set.
Target Accuracy :	± 2 ~ 3 sec (60 ~ 90 m).
Observation Error :	± 0 ~ ± 1.8".
Junction with Existing Point:	Existing astronomic and triangulation points.
Observation Accuracy :	Observation accuracy of one star, m_o $m_o = \pm 10 \sim 20$ sec. Average accuracy of 16 stars/set, m_s $m_s = m_o / \sqrt{16} = \pm 2.5$ sec. Average accuracy of 3 sets, m_m $m_m == m_s / \sqrt{3} = \pm 1.4 \sim 2.9$ sec. Personal error, m_p $m_p = \pm 2$ sec. Error due to atmospheric dispersion, m_a $m_a = \pm 1$ sec. Overall error, m_t $m_t = \sqrt{m_m{}^2 + m_p{}^2 + m_a{}^2} = \pm 2.6 \sim 3.6$ sec.
Formation of Operation Party:	10-member party comprising 1 engineer, 2 assistants and 7 labourers.
Survey Equipment:	Ni 2 automatic level, short-wave receiver, crystal clock, pen-recorder, 12 V battery, generator, and tripod.
Survey Period Required per Control	15 days.
Division of Operation Team:	Divided into two parties each covering 4 control points.
Stone Marker:	Concrete post measuring 20 x 20 x 90 cm.
Survey Cost per Control Point:	1,800,000 yen

B) Selection of Control Points and Observation Points

Distribution of the existing and new control points was so planned that they will be uniformly distributed in the mapped area with consideration given to the plotting work. Effort was also made to include as many control points as possible within the control strip of aerial photography.

In selecting the astronomic observation points, endeavours were made to satisfy the following conditions.

i) Each point is located in an open space providing unobstructed field of view.
ii) There are found a minimum of artificial obstructions including smoke and lamplight.
iii) Ground condition is stable.
iv) Survey equipment can be transported with ease.

C) Preparations

Time required for one set of astronomic observation (which covered 16 fixed stars in the present survey) is usually 1 to 1.5 hours. During this time, it is necessary to observe the following conditions in order to attain the required precision of observation.

1) The tripod should be driven into the ground.
2) The survey equipment should be exposed and accustomed to the atmosphere for at least one hour before starting observation.
3) The crystal clock indication and the time signal should be recorded continuously for two minutes by the pen recorder before and after the observation and either of the two records should be adopted.
4) The personal error of the observer should preferably be measured and recorded.
5) During observation, the observer should not touch any unnecessary part of survey equipment, nor should anyone other than himself be allowed to come near the tripod.
6) If the bubble in the astrolabe is not off the bubble in the attachment, adjustment should be made.
7) The time and date of observation, number of the observed star, and name of the observer should be entered in the recording paper without fail.
8) Observation of one set (16 stars) should be completed within three hours.
9) Five or more sets of starts should be selected for each point for observation in two or more nights. However, the number of sets may be reduced to three if unavoidable due to unfavourable weather condition or other reasons.

Before starting observation, the following preparation is required.

The astrolabe protractor should be set approximately N direction by means of a compass. Two or three stars should then be selected optionally from the list, and the time shown in the schedule is to be confirmed by checking against the time at which the selected stars pass the centre of field of vision of the astrolabe. This work is required for correction of the astrolabe azimuth.

In addition, the stars to be observed on respective observation days should be selected from the schedule for preparation of the daily observing star list showing the time at which they reach the fixed altitude as well as their azimuth.

D) Observation

a) The Ni-2 sub-first order automatic level employed as astrolabe has two reticles, and it is designed to observe the time at which the star comes to the centre of the two reticles. Although five each of upper and lower pairs of reticles, totalling ten pairs, are provided, the passing time of each star was observed using three each of upper and lower pairs (six pairs in total) in the present survey.

b) On the basis of the observation schedule prepared in advance, the assistant observer tells the observer the azimuth in which the subject star rises. The observer sets the telescope in the proper direction by means of the astrolabe protractor, and depresses the recorder key when the star comes to the right position which is the centre of the two reticles. The assistant assigned to time recording checks if the key mark is indicated on the record. The mean value of recorded time is taken as the observation time.

E) Calculation and Adjustment

The average time obtained by observation is corrected by the difference between the time signal and crystal clock indication and also by making use of personal error, if necessary. Further, the atomic time UT_1 given by time signals is corrected to the universal time UT_2 to obtain the sidereal time. The right ascention and declination of the observed stars are obtained from the apparent places of stars with the latitude and longitude in the existing map taken as provisional latitude and longitude. Manual field calculation is worked out on the basis of these data in order to check the error of closure of sin H.

Calculations for establishing ground control points are worked out as follows.

The observation point is expressed by O, zanith by Z, celestial polar by P, celetial body by X, intersection of the hour circle passing P and X and the celestial equator by C, and intersection of the great circle passing Z and X and the horizontal by G. When the right ascension and declination of a telestial body X are respectively taken at α and δ, and hour angle at t, the relationship of the sidereal time θ with the hour angle and right ascention can be expressed by the following equation.

$t = \theta - \alpha$ (Angle formed by PZ and PX at P) .... (1)

$\delta = < XOC$

The zenith distance Z is the complementary angle of the altitude of celestial body, $<ZOX$. Hence, the following equation can be set up if the longitude and latitude at the observation point are respectively taken at λ and φ.

$\varphi = < ZOB$

Further, the sidereal time θ can be expressed as follows.

$\theta = UT + R + \lambda$ .... (2)

Hence, the hour angle t can be expressed as follows by substituting equation (1) for equation (2).

$t = (UT + R + \lambda) - \alpha$ .... (3)

where, UT : Greenwich mean time.

R : Value required for correcting UT to sidereal time.

The following three relationships exist in a spherical triangle PZX.

$PZ = 90^\circ - \varphi$

$PX = 90^\circ - \delta$

$ZX = Z = 90^\circ - H$ (altitude of celetial body)

The following equation can therefore be obtained by applying the cosine formula.

$\cos Z = \sin \varphi \cdot \sin \delta + \cos \varphi \cdot \cos \delta \cdot \cos t$ .... (4)

Equations (3) and (4) indicate that if the zenith distance Z or height H is obtained with satisfactory accuracy, the longitude, latitude and height can be obtained by observing the transit time of three or more stars at a fixed altitude. Observation equations can be established in the following way.

If the assumed longitude, latitude and height are respectively taken at λ_0, φ_0 and H_0 and their corrected values at $\Delta\lambda$, $\Delta\varphi$ and ΔH, the determined longitude (λ), latitude (φ) and height (H) are expressed as follows.

$$\left.\begin{aligned} \lambda &= \lambda_0 + \Delta\lambda \\ \varphi &= \varphi_0 + \Delta\varphi \\ H &= H_0 + \Delta H \end{aligned}\right\} \quad \cdots\cdots\cdots \quad (5)$$

By applying equation (2),

$$t = (UT + R + \lambda_0 + \Delta\lambda) - \alpha = h + \Delta\lambda \quad \cdots\cdots\cdots \quad (6)$$

$$\text{where, } h = (UT + R + \lambda_0) - \alpha$$

If $\Delta\lambda$ and $\Delta\varphi$ are omitted by substituting equations (5) and (6) for equation (4), the following equation can be set up.

$$\begin{aligned} \sin(H_0 + H) &= \sin(\varphi_0 + \Delta\varphi)\cdot\sin\delta + \cos(\varphi_0 + \Delta\varphi)\cos\delta\cdot\cos(h + \Delta\lambda) \\ &= \sin\varphi_0\cdot\sin\delta + \cos\varphi_0\cdot\cos\delta - \cos h + (\cos\varphi_0\cdot\sin\delta - \sin\varphi_0\cdot \\ &\quad \cos\delta\cdot\cos h)\,\Delta\varphi - \cos\varphi_0\cdot\cos\delta\cdot\sin h\cdot\Delta\lambda \quad \cdots\cdots\cdots \quad (7) \end{aligned}$$

On the other hand, since the azimuth (A_N) and height (H) of the spherical triangle are determined by the assumed longitude (β_0), hour angle (h) and apparent star declination (δ), they can be obtained by applying the formula of spherical triangle.

$$\left.\begin{aligned} \sin H &= \sin\varphi_0\cdot\sin\delta + \cos\varphi_0\cdot\cos\delta\cdot\cos h \\ \cos H\cdot\cos A_N &= \cos\varphi_0\cdot\sin\delta - \sin\varphi_0\cdot\cos\delta\cdot\cos h \\ \cos H\cdot\sin A_N &= -\cos\delta\cdot\sin h \end{aligned}\right\} \quad \cdots\cdots \quad (8)$$

The following observation equation can then be obtained by arranging equation (8) by substitution with equation (7).

$$\sin A_N\cdot\cos\varphi_0\cdot\Delta\lambda + \cos A_N\cdot\Delta\varphi - \Delta H = H_0 - H \quad \cdots\cdots\cdots \quad (9)$$

If the values of $\sin A_N$, $\cos A_N$ and H are obtained by equation (6), $\Delta\lambda$, $\Delta\varphi$ and ΔH can be calculated by the method of least squares by applying equation (7).

F) Result Table

The following result table shows the additional astronomic points established by the present survey.

Point	B	L	X	Y	Location
PUTAS	1–2°15'36".86	115°39'19",87	+ 750 k 130 m	+ 350 k 510 m	Muaraälong
PUTAS	2–2°27'50".60	115° 6'11".62	+ 727 k 520 m	+ 289 k 110 m	Danaupanggan
PUTAS	3–2°47'19".32	115°15'16".14	+ 691 k 650 m	+ 305 k 990 m	Kandangan
PUTAS	4–2°52'11".07	115° 1'42".38	+ 682 k 640 m	+ 280 k 870 m	Sungairutas
PUTAS	5–3° 3' 9".33	115° 6'41".84	+ 662 k 440 m	+ 290 k 153 m	Tatakan
PUTAS	6–3°26'24".44	114°49'11".71	+ 619 k 520 m	+ 257 k 810 m	Banjar baru
PUTAS	7–3°30'59".43	115° 0' 8".99	+ 611 k 110 m	+ 278 k 120 m	Riam Kanan
PUTAS	8–3°44'34".23	114°45' 5".91	+ 586 k 010 m	+ 250 k 310 m	Pelai hari

5.2 Tide Measurement

A) Outline

Amount of Work:	Establishment of one tide station.
Purpose :	Provision of data useful for determining the height of vertical control points (Onland height measurement is usually based on the mean value of the highest and lowest sea water levels observed over a long period time, whereas charts and port improvement works are based on the lowest L.W.L. The gap between the two should therefore be taken into due account).
Required Accuracy :	Within about 1/20 of the maximum tidal amplitude.
Method of Measurement :	(1) In the float system, an observation well is constructed with a float type self-recording tide gauge installed on it for continuous tide measurement. (2) In the pressure system, a pressure sensing weight is securely fixed on the suitable sea bottom rock and connected by a tube to the on-land automatic recorder for continuous tide measurement. (3) In either system, levelling is performed between the datum level for tide measurement and the bench mark set up near the station for determination of their elevation.
Period of Observation :	1 month at least and preferably 1 year or longer.
Target Accuracy :	10 cm (Minimum required accuracy = Maximum tidal range 3.0 m X 1/20 = 15 cm).
Accuracy of Measured Value :	± 8.4 cm.
Tide Gauge :	Float type tide gauge (Model FDC-30, product of Sokkisha Co., Ltd.) and pressure type tide gauge used temporarily (Model LPT manufactured by Kyowa Shoko Co., Ltd.).
Calculation :	Measured values are put to harmonic analysis to obtain the mean sea level as well as the period and phase of each component tide.
Monument :	A concrete post (20 x 20 x 90 cm) driven into the ground at the original bench mark with a metallic marker embedded on top.
Final Result :	Record of tide measurement and result table of computation.
Construction Cost :	1,200,000 yen.

B) Construction of Tide Station

In selecting the construction site of a tide station, care must be exerted to satisfy the following conditions.

1) It is located on a large exposed bed-rock.
2) Water depth is large enough for tide observation (i.e., the site is not on the coast with a shelving submarine topography).
3) Influence of wind and waves is small.
4) Suspended sediment is small in quantity.
5) The site is little subject to seiche and other secondary influences.
6) Artificial disturbances are limited.
7) Junction with the on-land bench mark is possible.

It was not possible to find a site that meets all these conditions during the survey period. It was therefore determined to construct the station on Takison coast which appeared to be favourably conditioned for junction with the levelling route. The team learned that this coast had been selected as the site of a temporary tide station before the outbreak of World War II.

After selection of the construction site, a pressure type gauge was installed in October 1972 and tide observation was conducted for about a month until November of the same year.

Observation by the pressure type gauge was later made impossible due to suspended sediment. Since the sea bottom has a mild slope and it was not possible to find another suitable site, an observation tower was constructed at a point off the coast with a float type gauge installed for continued tide observation.

The new tide station was commissioned in March 1973 and kept in operation until October of the same year though intermittently. Minimum required tide data were therefore obtained by the observation conducted at this station.

Past operation disclosed that special care must be taken against suspended sediment and damage of equipment due to adherence of oysters in the construction of a tide station on this coast.

C) Junction with On-land Datum Point

In order that the datum level determined by tide measurement will serve as the basis for on-land height measurement, its vertical relationship with a bench mark established in the neighbourhood should be made clear.

In the case of the pressure type gauge, ordinary levelling is conducted at the predetermined time when the sea is calm to obtain the relative height between the bench mark and the sea level at that time along the coast, and the height difference thus obtained is set at a definite value by comparing it with the corresponding height appearing on the tide record. In order to improve the accuracy of correspondence between the relative height on the tide record, tide level is observed several times and the mean value is adopted as the datum level for tide measurement.

In the case of the float type gauge, a small vessel containing sea water is fixed in the specified position inside the observation well of the tower in order to create an artificial calm water surface, and the float is suspended inside the vessel so that the height of the artifical sea surface will be recorded by the tide gauge. Further, levelling is conducted to obtain the relative height between the artificial sea surface and the bench mark established near the station in order to clarify the height difference between the datum level appearing on the tide record and the bench mark.

D) Observation

Observation is conducted by automatic recording on the recording paper which is wound by the function of a cell clock. This method calls for periodical equipment inspection and monthly replacement of the recording paper. Datum level measurement described in the foregoing item is required at the beginning and end of observation as well as at time of replacing the recording paper. The results obtained and the exact time of measurement should be entered on the recording paper.

In the event of any abnormality of the gauge performance or tide record, adjustment should be made at time of inspection, with careful study also made on the reliability of the data recorded before adjustment.

E) *Computation*

If tide data are recorded continuously for more than 1 year with acceptable accuracy, the mean value is taken as the mean sea level. In case no such perfect record is obtained as in the present work, main component tides should be put to a harmonic analysis and correction should be made to remove their influence. Tide level measured at each time, M, is expressed by the following equation.

$$M = M_o + \sum_{i=1}^{n} [A_i \cos (W_i\ t) + B_i \sin (W_i\ t)]$$

$$= M_o + \sum_{i=1}^{n} F_i \operatorname{Sin} (\frac{Zti\ \ t}{Ti} + t_i)$$

where,

M_o : Mean sea level.

F_i : Amplitude of each component tide.

T_i : Period of each component tide.

t_i : *Phase of each component tide.*

Constants of these periodic functions can be obtained by Fourier analysis. In the present work, however, the constants were obtained not by Fourier analysis but by the method of least squares for the following reasons.

1) The number of measured values was small.
2) Observation was concentrated within a specific period of time.
3) Measured values were not uniformly distributed throughout the observation *period.*

For the purpose of analysis, M_2, N_2, S_2, K_2, O_1, Q_1, M_1, P_1 and K_1 all having a short period as well as S_a and S_{sa} having a long period were used.

For the purpose of computation, a total of 1,512 measured values were read out at intervals of 1 hour.

F) Results

Final mean value and the values of component tide were as follows:

M_o = 3.32 *m*

S_{sa} a_1 = –0.550 m b_1 = –.799 m

S_a a_2 = –0.563 m b_2 = + 0.267 m

The arithmetic mean of all the observed values was as follows:

Mean value = 2.55 m

Since the height of the bench mark is know to be 5.020 m above the zero level read out from the tide record as a result of the bench mark levelling, the elevation of the bench mark above the mean sea level turns out to be 1.700 m.

5.3 Levelling Work

A) Outline

Amount of Work :	: 352 km.
Levelling Accuracy Required for Plotting	: ±2.5 m.
Levelling Accuracy Required for Irrigation and River Improvement Projects	: ±5 cm (within the survey area). Levelling accuracy of ±2.5 m suffices for the purpose of plotting. However, in order that the levelling will provide the height reference for future irrigation and river improvement projects, it was so conducted as will maintain the accuracy of the second order levelling and bench marks were also established.
Number of Height Control Points	; 21 main points (No. 0 ~ No. 20) 40 subpoints (No. 101 ~ No. 140)
Distance between Height Control Points	: 6 km on the average.
Survey Equipment	: WILD N3 (Sensitivity – 10", Magnification – 42); Levelling rod, wooden, graduated in 1 cm.
Check Point	: Establishment of check points at intervals of 1 km for duplicate levelling (existing road kilometer posts were used).
Bench Mark	: The bench mark was established near the Takison tide station, and its elevation was determined with the mean sea level in Takisong bay set at 0 m according to the tide record in order to provide the height reference for levelling work.
Elevation of Bench Mark	: 1.700 m
Tolerance of Error	: 7.5 mm $\sqrt{S}$ S : Route length in km.
Levelling Accuracy	: 2.9 mm $\sqrt{S}$ (Arithmetic mean of errors between height control points)
Bench Mark Specification	: Main point – concrete post measuring 20 x 20 x 90 cm, with a 7 cm diameter metal marker fixed on top. Sub-point – concrete post measuring 15 x 15 x 90 cm, with a 1.5 cm round-headed rivet fixed on top.
Formation of Operation	: 5-member party comprising 1 engineer, 1 assistant and 3 labourers and having 1 motor-vehicle.
Number of Operation Parties	: Beside the three parties into which the team was divided, one additional party was organized by recruiting its members from the Astronomic Survey Team.
Final Result	: Original draft, field note, result table,

route, and description of points.

Amount of Work per Day : 1.6 km.

Cost per km : 60,000 yen

B) Specification of Observation Work

The work method and survey equipment were determined with consideration given to the following conditions.

The standard error of observation along a distance of about 280 km from Takison to Tandjoeng was required to be held within ±5 cm.

To satisfy this requirement, the error per km should be maintained within 50 mm $/\sqrt{280} = 3.0$ mm (1).

It was assumed that the bubble level sensitivity was 10"/2 mm and the maximum length of sight on one side was less than 70 m. If the error in the bubble observation is 1/5 of 1 division = 2" ($\doteqdot 10^{-5}$) in this case, the reading error of one of the staffs, e, turns out to be as calculated below.

$$e = \pm 2'' \quad 70 \text{ m} \div 10^{-5} \times 70 \text{ m} = 0.7 \text{ mm}$$

Since the relative height at one station is the difference between the readings of two levelling rods erected in front and back, the error at that station, e_1 is $\sqrt{2}$ times e as shown below.

$$e_1 = \sqrt{2e} \doteqdot \sqrt{2} \times 0.7 = 1.0 \text{ mm}$$

The distance between the said two rods is 140 m (= 70 m x 2), so that more than 7 observations are conducted for each 1 km. The accumulative error involved in the 7 observation, σ, is $\sqrt{7}$ times the value of e_1.

$$\sigma = \sqrt{7}\ e_1 = \sqrt{7} \times 1.0 =: 2.6 \text{ mm}$$

By conducting levelling by this method, therefore, it is possible to attain the accuracy (1) mentioned above.

Further, since the standard error for each 1 km, σ, is known to be 2.6 mm from the above calculation, the discrepancy caused by duplicate 1 km levelling, σ_1, is $\sqrt{2}$ times the value of σ.

$$\sigma_1 = 2.6 \text{ mm} \sqrt{2} = 3.7 \text{ mm}$$

The probability of developing an error larger than twice σ_2 is about 5 %. Hence, the re-levelling rate for ensuring the required accuracy level can be held at 5% by conducting re-levelling only when the error in duplicate levelling is larger than this value. In other words, the upper limit of error in duplicate levelling, σ_2, is 7.4 mm as calculated below.

$$\sigma_2 = 2\sigma_1 = 3.7 \text{ mm} \times 2 = 7.4 \text{ mm}$$

For reasons and conditions described above, the following specification was adopted for the present survey.

1) Levels with an accuracy of 10"/2 mm should be used.
2) The maximum length of sight should be less than 70 m and the standard *length of sight should be set at 50 m.*
3) Reading should be performed in the unit of 0.1 mm.
4) The discrepancy in duplicate levelling between control points should be held within 7.5 mm$\sqrt{S}$.
5) The rate of re-levelling work is estimated at about 5%.

C) Observation Work

The route was so selected as will run along national highways and principal local roads, and 61 new bench marks were set up at conspicuous places including the compounds of government and public offices for easy maintenance and utilization.

One duplicate levelling was conducted along each section of the route, with its allowable error held within the aforementioned value.

The following are the major points observed int he actual levelling work.

1) The length of sight was set at 50 m (max. 70 m).
2) The measuring rod was placed perpendicularly on the steel foot plate which was driven and settled sufficiently into the ground.
3) On fine but heavily cloudy days, observation was not conducted from 2:00 to 3:00 p.m. where possible.
4) Inspection of observed values were conducted for the distance covered by each pair of check points involving bench marks.
5) Computation was worked out for all observation stations at which the staff was erected. While the relative height over a given section fo the route is the sum total of all relative heights, it should be equivalent to the value obtained by deducting the total plus sight from the total minus sight of that section. Two computations were therefore made for mutual checking of the balance thus obtained and the sum total of relative heights.

   Specifically, (a) — (b) appearing in the column of T of the separate field note should equal (c) — (d) as exemplified by the following actual computation.

LEVELLING NOTO

FROM B/R 42 TO B/R 43

DATE 121 Aep 1 1972

WX. Fine
WIND. Calm

INST. NO.173498–N3
STAFF NO. 5, 6
OBSERVER

NO	DIST	B. S.	F. S	HEIGHT DIFF +	HEIGHT DIFF −	REMARKS
		m	m	m	m	
1	50	0.6423	1.3001		0.6578	B/R 42 7 h 50 m
2	52	1.3752	1.1418	0.2334		
3	42	1.2954	1.4701		0.1747	
4	54	1.4226	1.3925	0.0301		
5	42	1.4948	1.4372	0.0576		
6	48	1.5677	1.5731		0.0054	
7	50	1.5938	1.4615	0.1323		
8	45	1.3743	1.5268		0.1525	
9	46	1.6234	1.5302	0.0932		
10	40	1.4637	1.3985	0.0652		
11	13	1.3954	1.4400		0.0446	
12	17	1.4641	1.0990	0.3651		B/R 43 8 h 45 m
		(a)	(b)	(c)	(d)	
T.	499	16.7127	16.7708	0.9769	1.0350	
C.			−0.0581	−0.0581		
R.					−0.0581	

16.7127 − 16.7708 = 0.9769 − 1.0305 = −0.0581.

If the above requirement is not satisfied, it indicates that the computation involves some error.

D) Final Result

The following final result table shows the elevation of respective stations determined on the basis of the mean sea level which was established by tide measurement and harmonic analysis.

DECIDED HEIGHT ABOVE SEA LEVEL

THE BARITO RIVER BASIN — Nov. 1972

Sta. No.	Δ H	H	LOCATION
	m	m	
		1.7000	Takisong
	− 0.4932		
PUTL 0		1.2068	Takisong
	+ 4.5464		
101		5.7532	
	+ 0.3132		
102		6.0664	
	+ 9,6532		
1		15.7196	Pelaihari
	+ 0.7951		
103		16.5147	
	− 11,4586		
104		5,0561	
	+ 4,0365		
2		9,0926	Tambanguang
	+ 2,6829		
105		11,7755	
	− 5,8142		
106		5,9613	
	+ 12,4448		
3		18,4061	
	− 12,2647		
107		6,1414	
	− 4,4466		
108		1,6948	
	+ 4,5214		
4		6,2162	
	+ 16,8042		
109		23,0204	
	− 7,6337		
110		15,3867	
	− 9,3590		
5		6,0277	Martapura
	− 1,4453		
111		4,5824	
	+ 1,8090		
112		6,3914	
	+ 1,6313		
6		8,0227	
	+ 1,5199		
113		9,5426	
	+ 2,7978		
114		12,3404	
	+ 19,1159		
7		31,4563	
	− 2,9016		
115		28,5547	
	−.10,0937		
116		18,4610	
	− 7,9822		
8		10,4788	Binuang
	+ 7,0298		
117		17,5086	
	− 4,7619		
118		12,7467	
	− 6,1714		
9		6,5753	
	+ 1,6689		
119		8,2442	
	+ 3,2292		
120		11,4734	
	− 3,2808		
10		8,1926	Rantan
	− 1.9438		
121		6,2488	
	+ 2,7890		
122		9,0378	
	+ 8,8519		
11		17,8897	
	− 8,7411		
123		9,1486	
	+ 0.2588		
124		9,4074	
	+ 1,0938		
12		10,5012	Kandangan
	− 3,7043		
125		6,7969	
	+ 0.0394		
126		6,8363	
	+ 3,2694		
13		10,1057	
	− 4,8141		
127		5,2916	
	− 1,4299		
128		3,8617	
	− 1,1430		
14		2,7187	
	+ 0.8989		
129		3,6176	
	+ 1,2034		
130		4,8210	
	− 0.0166		
15		4,8044	Amuntai
	+ 0.8834		
131		5,6878	
	+ 0.9726		
132		6,6604	
	+ 0.5328		
16		7,1932	
	+ 1,0399		
133		8,2331	
	+ 0,6143		
134		8,8474	
	+ 0.1406		
17		8,9880	
	+ 3,3745		
135		12,3625	
	+ 0.6137		
136		12,9762	
	+ 2.4888		
18		15.4650	Tanjung
PUTL 4		6,2162	
	− 5.2490		
137		0.9672	
	− 0.0547		
138		0.9125	
	− 0.2140		
19		0.6985	Banjar-masin
12		10,5012	Kandangan
	− 6,9643		
139		3,5369	
	− 0.4630		
140		3.0739	
	− 1.0410		
20		2,0329	Negara
10		8,1926	Rantau
	− 7,1150		
PUTAS 4		1,0776	

5.4 Triangulation and Traversing (Reference Data)

Neither triangulation or traversing was necessary for mapping the Barito river basin. If triangulation is required for preparation of a 1/10,000 topographic map demanding highly accurate surveying, it would have to be conducted according to the following specification.

A) Triangulation

		40 points (in an area of 16,800 km^2).
Amount of Work	:	30 km.
Purpose	:	Establishment of pass points for preparation of a 1/10,000 map and for possible detail survey (e.g., traversing) in future.
Equipment	:	Theodolite graduated in sec (equivalent in performance to WILD TZ), heliotrope (sun beam reflector for long distance collimation), and electromagnetic distance meter.
Observation Method	:	At least two sides are measured with the electromagnetic distance meter to determine the base line, and angle measurement is performed by six pairs of observations applying the direction method.
Computation Method	:	Net adjustment computation by the method of least squares.
Target Accuracy	:	Horizontal position ±50 cm.
Formation of Operation Party	:	14-member party comprising 2 engineers, 2 assistants and 10 labourers and equipped with a motor-vehicle.
Stone Marker	:	Concrete post.
Final Result	:	Field note, observation record, and result table (X.Y.H. B.L.).
Period Required per Point	:	8 days – 2 days for selection of points 4 days for cleaning and signal erection 2 days for observation
Period Required per Side (Base Line)	:	2 days (after completion of angle measurement).
Total Number of Days Required	:	324 days.
Kinds of Results	:	Field note, original draft, computation book, description of points, net map, and result table (X.Y.H. B.L.)
Cost per Point	:	1,200,000 yen = 150,000 yen (unit cost per operation party) x 8 days

B) Traversing

Amount of Work	:	50 points (for a traverse route of 500 km).
Average Side Length		
Average Side Length	:	10 km.
Purpose	:	Block adjustment (plane and height).
Supplementary Purpose	:	Establishment of pass points for prepara-

	tion of a 1/10,000 map and of control points for future detail survey including traversing.
Equipment	: Theodolite graduated in sec (equivlanet in performance to WILD 2), heliotrope, signal lamp, and electromagnetic distance meter. Specification of Tellurometer: Range of measurment — 100 m ~ 50 km Accuracy — ± 2 cm $+\frac{\text{Distance}}{300{,}000}$ Power source — 12 V battery
Observation Method	: 1) Horizontal angle is measured by 4 pairs of observations by the application of the direction method. 2) Vertical angle is measured by 4 pairs of observations. 3) Distance is measured by two observations made in opposite directions. 4) Azimuth is measured by astronomic observation every 5 to 6 points.
Computation Method	: Net adjusting computation by the method of least squares.
Target Accuracy	: Horizontal position ±50 cm, height ±5 cm.
Formation of Operation Party	: 16-member party consisting of 2 engineers, 4 assistants and 10 labourers, and equipped with two motor-vehicles (with chauffers).
Period Required per Point	: 6 days — 2 days for selection of points 2 days for clearing and signal erection 2 days for observation
Stone Marker	: Concrete post.
Kinds of results	: Field note, computation book, result table (X, Y, H, B, L.), description of points, and net map.
Cost per Point	: 1,250,000 yen = 208,500 yen (unit cost per party) X 6 days

6. AERIAL TRIANGULATION AND BLOCK ADJUSTMENT

Aerial triangulation and block adjustment were conducted by the analytical method during the period from January to March 1973.

The entire area was divided into three sub-blocks by the skeleton courses, and block adjustment was made for each of them.

The two sub-blocks on the east side were covered by a super wide angle camera having a focal length of 8.8 cm, and the remaining one sub-block on the west side

also by a super wide angle camera with f = 12.5.
Outline of the work is as described below.

6.1 Outline of Work

Purpose	: Establishment of additional orientation points required for the mapping work.
Amount of Work	: 53 strips, 854 models.
Control Data	: 1) Existing astronomical surveying points and contact prints showing such points. 2) New astronomical surveying points established during the present work and contact prints showing such points. 3) Existing triangulation points and contact prints showing such points. 4) Data of levelling work and contact prints showing such data. 5) 1/500,000 topographical map (reference map).
Instrument	: 1) Precision stereoscopic pricking device Model KRP for pricking work. 2) Precision stereo-comparator Model STKI for coordinate measurement. 3) Electronic computer DEMOS for computation work.
Photo (diapositive)	: 1) Photos taken by super wide angle camera, s = 1/50,000, f = 88.78. 2) Photos taken by super wide angle camera, s = 1/40,000, f = 152.64.
Division of Area	: The entire survey area was divided into three sub-blocks. For each of the three sub-blocks, simultaneous block adjustment computation was conducted for local strips upon completion of the frame which was worked out on the strength of principal strips. For sub-block 1 which covered the northern part of the survey area, block adjustment computation for local strips was carried out after the frame computation based on three each of principal strips running east to west and south to north respectively. For sub-block 2 which occupied the southern part of the survey area, block adjustment computation was conducted upon completion of the collimating line computation which was worked out suing seven principal strips. As for sub-block 3 which extended in the southwestern part of the survey area, block

		adjustment computation was conducted directly by means of the tie points with sub-block 2 as well as the control points within sub-block 3.
Accuracy of Computation	:	1) Target accuracy — 1 ~ 2 mm on the average on the topographic map. 2) Residual of control points — 1.8 mm on the map.
Formation of Operation Party	:	7-man party consisting of 1 chief engineer, 2 engineers and 4 assistant engineers.
Work Period	:	3 months.
Results	:	1) Diapostives with pricked orientation points. 2) *Final result table of orientation points.* 3) Index map.
Cost	:	14,000 yen per model

6.2 Operation Procedure

In order that aerial photos may be stereoscopically observed, strips are so photographed that they will overlap each other about 60%. The 60% overlapped portion is called "model" which is the unit employed in the stereoscopic observation and measurement.

In the computation work which follows the stereoscopic observation, correction for the film shrinkage and determination of the photo center are performed at first, then orientation is carried out in the order of relative orientation, successive orientation and abosolute orientation.

The relative orientation is conducted to discover the mutual relationship between two adjoing photos. In this work, the following five rotational elements are so decided that Y coordinates of the left and right photos will coincide with each other.

K_1 (swing) and ϕ_1 (pitching) of the left camera; and K_2 (swing), ϕ_2 (pitching) and W_2 (rolling) of the right camera.

For connection of the model produced by the first and second photos with that created by the second and third photos, the tilt of the second photo which is common to both models is set at an equal value, and the distance between the second photo's projection center and the pass point common to both models (object distance) is also set at an equal value.

By the repetition of this process, which is called successive orientation, photos taken continuously are connected successively into a single strip.

Respective points in the strip thus prepared are made to coincide with the corresponding ground control points for conversion to geodetic coordinates. This process is called absolute orientation.

The absolute orientation is conducted either for a single strip or for the whole of a block. In the present work, the survey area was divided into three sub-blocks, and the frame computation of each sub-block was worked out by the absolute orientation of a single strip, and then respective local courses were blocked for simultaneous adjustment computation.

Data of pass points for mapping are stored in the computer. Photo coordinates are converted to geodetic coordinates for each strip according to the factors of film shrinkage correction, relative orientation, successive orientation and absolute orientation, and compile into the final result table together with the elements of model orientation.

The following formula are employed in the computation.

1) Correction of Film Shrinkage

$$x = a_x X + b_x XY + C_x Y + d_x$$
$$y = a_y X + b_y XY + C_y Y + d_y$$

where, x, y: Corrected photo coordinates.
X, Y: Measured photo coordinates.

2) Relative Orientation

$$\Delta Y = y_2 - y_1 = X_1 K_1 - \frac{x_1 y_1}{f} \varphi_1 + X_2 K_2 + \frac{X_2 Y_2}{f} \varphi_2 + (1 + \frac{Y_2^2}{f^2}) W_2$$

where, x_1, y_1: Coordinates of the left photo.
X_1, Y_1: Coordinates of the right photo.

$$X_1{}^K = X_1 \text{ Coo } K_1 + Y_1 \text{ Sin } K_1, \quad X_1{}^{\varphi} = \frac{X_1{}^K \text{Coo } \varphi_1 - \text{Sin } \varphi_1}{\text{Coo } \varphi_1 + X_1{}^K \text{Sin } \varphi_1}, \quad X_x{}^w = \frac{X_2{}^{\varphi}}{Y_2{}^{\varphi}\text{SLW}_2 + \text{Coo } W_2}$$

$$Y_1{}^K - Y_1 \text{ Coo } K_1 - X_1 \text{ Sin } K_1, \quad Y_1{}^{\varphi} = \frac{Y_1{}^K}{\text{Coo } \varphi_1 + X_1{}^K \text{ Sin } \varphi_1} \quad Y_2{}^w = \frac{-\text{SLW}_2 + Y_2{}^{\varphi} \text{ Coo } W_2}{Y_2{}^{\varphi} \text{ Sin } W_2 + \text{Coo } W_2}$$

$$X_2{}^K = X_2 \text{ Coo } K_2 + Y_2 \text{ Sin } K_2 \qquad X_2{}^{\varphi} = \frac{X_2{}^K \text{ Coo } \varphi_2 - \text{Sin } \varphi_2}{\text{Coo } \varphi_2 + X_2{}^K \text{ Sin } \varphi_2}$$

$$Y_2{}^K = Y_2 \text{ Coo } K_2 - X_2 \text{ Sin } K_2 \qquad Y_2{}^{\varphi} = \frac{Y_2{}^K}{\text{Coo } \varphi_2 + X_2{}^K \text{ Sin } \varphi_2}$$

3) Successive Orientation

$$\begin{matrix} X_1 \\ Y_1 \\ Z_1 \end{matrix} = M (\varphi_{2,1}) (K_{2,1}) (-K_{1,2}) (-\varphi_{1,2}) (-W_{1,2}) \begin{matrix} X_2 \\ Y_2 \\ Z_2 \end{matrix}$$

where, x_1, y_1, z_1: Coordinates of the first model.
X_1, Y_1, Z_1: Coordinates of the second model.
$\varphi_{2.1}$: φ of the left photo of the second model.
$K_{1.2}$: K of the right photo of the first model.
M : Scale.

4) Absolute Orientation

$$\begin{matrix} X' \\ Y' \\ Z' \end{matrix} = R \begin{matrix} X \\ Y \\ Z \end{matrix}$$

$$R = \begin{matrix} \text{Coo } \phi \text{ Coo } K & \text{Coo } \Omega \text{ Sin } K + \text{Sin } \Omega \text{ Sin } \phi \text{ Coo } K & \text{Sin } \Omega \text{ Sin } K - \text{Coo } \Omega \text{ Sin } \phi \text{ Coo } K \\ -\text{Coo } \phi \text{ Sin } K & \text{Coo } \Omega \text{ Coo } K - \text{Sin } \Omega \text{ Sin } \phi \text{ Sin } K & \text{Sin } \Omega \text{ Coo } K + \text{Coo } \Omega \text{ Sin} \phi \text{ Sin } K \\ \text{Sin } \phi & -\text{Sin } \Omega \text{ Coo } \phi & \text{Coo } \Omega \text{ Coo } \phi \end{matrix}$$

$$X = ax - by + xo \qquad X = AX - BY + C(X^2 - Y^2) - 2 Dxy + Xo$$
$$Y = bx + ay + yo \qquad Y = BX + AY + D(X^3 - Y^2) + 2 Cxy + Yo$$

5) Elevation Correction Graph

A graph incorporating the earth curvature and the lens distortion was prepared for elevation correction of sub-blocks 1 and 2 which were photographed by a super wide angle camera.

7. COMPILATION

The compilation was commissioned to Japanese and Indonesian surveying companies. The two Indonesian contractors covered a total of about 2,500 km^2.
The compilation was carried out from April to August 1973. Outline of the work is as described below.

7.1 Outline of Work

Purpose	:	Preparation of the original sheet.
Amount of Work	:	20,000 km^2, 1,000 unit-days.
Source Materials	:	1) Diapositives with pricked pass points. 2) Final result table of aerial triangulation. 3) Data of field survey (Key for photo interpretation).
Instrument	:	Stereo plotter (Planimart Simplex IIC, Topocart B).
Operation Method	:	The original sheet was prepared by tracing the model which was produced on the stereo plotter according to the additional pass points established for each model by aerial triangulation.
Target Accuracy	:	Plotting error — horizontal ±0.5 m (on the map) — elevation ±1.5 m
Plotting Sheet	:	Polyester base, thickness — 0.075, shrinkage — 0.05%.
Formation of Operation Parties	:	10 parties each consisting of 1 engineer and 1 assistant engineer, with a chief engineer assigned to each five parties and a superintendent appointed to assume overall control and responsibility. 1,000 unit-days, 20 km^2/unit/day, total coverage approx. 20,000 km^2.
Results	:	1) Original sheet. 2) Date of pass points.
Cost per km^2		3,000 yen

7.2 Plotting

Neatlines, grid lines and ground control points are plotted on the plotting sheet. Plotting of ground control points and pass points is the simple work performed at the first stage of compilation. Hence, it is liable to be conducted without sufficient care and concentration. It is to be noted that errors in this stage are conductive to rejection and often lead to a fata failure.

7.3 Orientation

Orientation of each pair of photos is performed just as in aerial triangulation in the order of inner orientation, relative orientation and absolute orientation. In the absolute orientation, however, pass points established by aerial triangulation are used in place of the ground control points.

Since the analytical method is employed in the present work, the values of the elements of orientation are already available. By the input of these values to the plotter, a terrain model can be obtained with acceptably high accuracy.

7.4 Stereo Plotting

In the actual stereo plotting, the stereoscopic terrain model created in the plotter by orientation is traced accurately by means of the floating marks and tracing disk.

Ordinarily, plotting is performed in the order of lines (e.g., water systems, railways and roads), buildings and land classification boundaries along such lines, and then contour lines. All these features are expressed for easy identification. For instance, water is expressed in blue colour and rivers are indicated in true width whenever possible. In the case of straight lines, it is often the case that both ends are plotted at first and the line connecting them is drawn with a rule.

Contour lines are drawn by changing the colour pencil every five lines for easy indication of elevation.

Land classification boundaries of forests, fields, etc. are selectively delineated according to their size and need.

If the scale is smaller than 1/25,000, clusters are usually expressed by generalization.

Boundaries of vegetation, mountain roads, underground passageways, features shaded by mountains, etc. which are not clear on the photo or subject rto rapid secular change are plotted by referring to the data and photos obtained by field reconnaissance.

Further, since elevation of any selected point can be readily measured by photogrammetry, many independent spot elevations are indicated.

8. FIELD RECONNAISSANCE

Field reconnaissance was conducted in time with the control point survey, i.e., from September to December 1972.

Since the reconnaissance was intended primary for preparation of the key for photo interpretation, sufficient prior reading and examination of aerial photos was conducted before starting it. Outline of the work is as described.

8.1 Outline of Work

Amount of Work : Approx. 5,000 km^2 (conducted only in those parts of the survey area which allowed passage of the operation party)

(1) Compilation of the survey data into a key for photo interpretation for stereo plotting of details.

(2) Direct use of the survey data in the complilation for expression of various features not clear on aerial photos.

Operation Method : Reconnaissance was conducted in the following sequence.

(1) Prior reading and examination of on aerial photos and existing maps.

		(2) Selection of reconnaissance sites and route based on aerial photos and existing maps. (3) Collection of existing data from pertinent public organizations. (4) Reconnaissance by car and boat and on foot.
Content of Survey	:	(At Sample point) (1) General topographic condition of mountainous districts, hilly districts, plateaus, lowland districts, etc. (2) Existing condition of land use (forest, grassland, paddy field, upland field, orchard, plantation, cluster, etc.) and its comparison with aerial photos. (3) Confirmation of locations of public facilities (roads, rivers, religious structures and facilities, administrative and educational facilities, etc.) and comparison of their existing condition with aerial photos.
Instrument and Data	:	Measuring tape, stereoscope, photos (contact prints and mosaics) and other data.
Results	:	(1) Contact prints or mosaics on which survey data are indicated (enlarged photos are usually used if the scale is large). (2) Data serving as a key for photo interpretation.
Formation of Operation Party	:	2~3 member party consisting of 1~2 engineers and 1 guide and equipped with an automobile. 30 km^2 (area covered by direct reconnaissance survey)
Cost per km^2	:	2,300 yen 70,000 yen (cost per party) x 1/30 = 2,300 yen.

8.2 Photo Interpretation

Photo interpretation calls for the following work.

1) Collection of Data and Information
   Existing maps, data, materials, photos, etc. are collected to the maximum extent. At the same time, efforts are made to find out the situation of the survey area in advance according to the purpose of survey.
2) Photo Interpretation and Consolidation of Patterns
   Since the method of interpretation varies by purpose, the objects and extent of classification should be made clear at first.
   It is preferable that features presenting common tone, density and pattern be delineated on the map.
3) Planning of Field Reconnaissance
   Photo interpretation gives a fairly good idea about the features to be clarified by

reconnaissance as well as their locations. The field reconnaissance should be planned as will cover all such features in an efficient way.

8.3 Field Reconnaissance

Field reconnaissance is conducted to clarify the classification of vegetation and land use form, existing condition of facilities such as roads, railways, canals, bridges, etc., control points, geographic names, administrative boundaries, and so forth.

The operation party should follow the predetermined route, checking constantly its present position as well as each object against the aerial photo for successive correction of erroneous interpretation.

In the course of field reconnaissance, attention should be directed to the following.

1) Seasonal Change

   Comparison of various features with aerial photos becomes difficult if the field reconnaissance is not conducted in the same season in which aerial photography was undertaken. This is particularly so if the former is conducted in the wet season and the latter in the dry season, or vice versa.

   The same difficulty arises with vegetation in case there is a lapse of several years from the time of aerial photography to the field reconnaissance.

   In the field reconnaissance, therefore, it is necessary to form an adequate judgement bearing in mind when the photos were taken.

2) Artificial Change

   If aerial photos taken some years ago are to be used, attention should be directed to the new construction, alteration and abolition of public facilities such as roads, railways and buildings as well as to the changes in landuse form.

3) Concealments

   The route or shore line of small roads, ponds, swamps, rivers, etc. in forests often includes unclear portions on the photo. All such ambiguous portions should be covered by the field reconnaissance and indicated clearly on the photo.

8.4 Record of Field Reconnaissance

It is rarely the case that both field reconnaissance and stereo plotting are conducted by the same person. The data of field reconnaissance should therefore be recorded in a well consolidated form for anyone to readily understand it.

Ordinarily, water paint is used to indicate directly on the photo land classification signs in green, rivers and canals in blue, railways and other similar facilities in red, and annotations in red or other suitable colour.

For those features requiring detailed explanation, numbers alone are written in their position on the photo and a detailed explanation is added in a suitable space on the photo with such numbers as index numbers.

9. FIELD CHECK

Field check was conducted from August to October 1973 after completion of the stereo plotting work.

During this period, elevations were checked by levelling and a supplementary field survey was carried out for items not sufficient covered by the field reconnaissance. Outline of the work is as described below.

1. Outline of Work

Amount of Work	:	500 km^2 (area covered by sample survey).
Purpose	:	Inspection and correction of stereo plotting work.
Method of Field Check	:	(1) Checking of geographic names and administrative boundaries on the basis of data made available by government offices. (2) Checking of land classification and public facilities by the supplementary field reconnaissance. (3) Checking of spot elevations and contour lines by the inspecting survey.
Inspection Standard	:	(1) Checking of omissions and adequateness of position indication. (2) Tolerance of height — ±2.5 m
Source Materials	:	(1) Draft topographical map or its blueprint copy. (2) Aerial photographs. (3) Information on geographic names and boundary data. (4) Map symbols. (5) Photo coverage diagram and index map.
Instrument	:	(1) A set of level. (2) A set of writing utensils.
Results	:	(1) Topographic map completed after field check.
Formation of Operation Party	:	3-man party consisting of 2 engineers and 1 guide and equipped with an automobile.
Number of Parties	:	3.
Coverage per Party	:	1,600 km^2
Coverage per Person per Day	:	50 km^2 (Sample point alone)
Cost per km^2	:	1,500 yen 75,000 yen (cost per party)/50 km^2 = 1,500 yen

10. EDITING AND SCRIBING

Compilation was conducted by dividing the work between Indonesian and Japanese sides as in the case of stereo plotting.
Since the scribing and not the inking method was employed, drafting of all maps, including those plotted by the Indoneisan contractors, was conducted by Japanese side during the period from October 1973 to January 1974.
Outline of the work is as described below.

10.1 Outline

A. Compilation

Amount of Work	:	Approx. 20,000 km^2 (34 sheets)
Purpose	:	Completion of the original sheet into a draft topographical map with symbols and annotations.
Method of Work	:	Compilation in pencil according to the map symbols (due to the difference in scale by sheet).
Tolerance	:	Within ±0.5 mm on the map.
Source Materials	:	(1) Original sheet. (2) Contact prints. (3) Information on geographic names. (4) Road network map. (5) Map symbols.
Formation of Operation Party	:	6-man party consisting of 1 engineer and 5 assistant engineers.
Work Period	:	64 days.
Coverage per Person per day	:	11 km^2.
Cost per km^2	:	1,200 yen
Cost per Person per Day	:	Direct cost – 10,000 yen Indirect cost– 3,000 yen) 13,000 yen

B. Scribing

Amount of Work	:	Approx. 20,000 km^2 (34 sheets).
Purpose	:	Preparation of colour separation copies.
Method of Work	::	Scribing method for each colour plate

(1) 4 colour separation base plates.

Red plate – Roads, buildings, etc.

Blue plate – Rivers, paddy field, swamps, sea, etc.

Brown plate– Contour lines.

Black plate – Grid lines, independent buildings, vegetation, boundaries of vegetation, etc.

(2) 4 colour separation mask plates.

Green plate – Clusters (solid).

Blue plate (halftone) – Surface of sea, lakes and ponds, 2nd class rivers, etc.

Blue plate – paddy field symbols

Blue plate – Swamp symbols (solid expression).

* Halftone symbol plates to be prepared by the photo-mechancial process.

(3) Marginal information plate – 1 positive plate including annotations.

* Photo composition process to be applied to geographic names.

Tolerance	;	±0.5 mm on the map.

Specification of Drawing Paper	:	(1) Thickness Polyester base .... more than 0.075 mm Scribing base & peel coat base ..... more than 0.12 mm (2) Shrinkage ratio ..... less than 0.05%
Instrument and Materials	:	(1) Phototypesetting machine. (2) A set of scribing and drawing instrument. (3) Polyester base (1 marginal information plate for each sheet). Scribing base (4 for each sheet). Peel coat base (4 for each sheet).
Source Materials	:	(1) Map symbols. (2) Specification for annotation. (3) Data on marginal information.
Formation of Operation Parties	:	4 parties each consisting of 5 engineers and 2 controller.
Work Period	:	64 days.
Coverage per Person	:	10 km² 400 yen
Cost per km²	:	Direct cost 1,400 yen
Cost per Person	:	Direct cost – 10,000 yen Indirect cost – 4,000 yen) 14,000 yen

10.2 Operation Procedure

1. Colour Separation Scribing Method

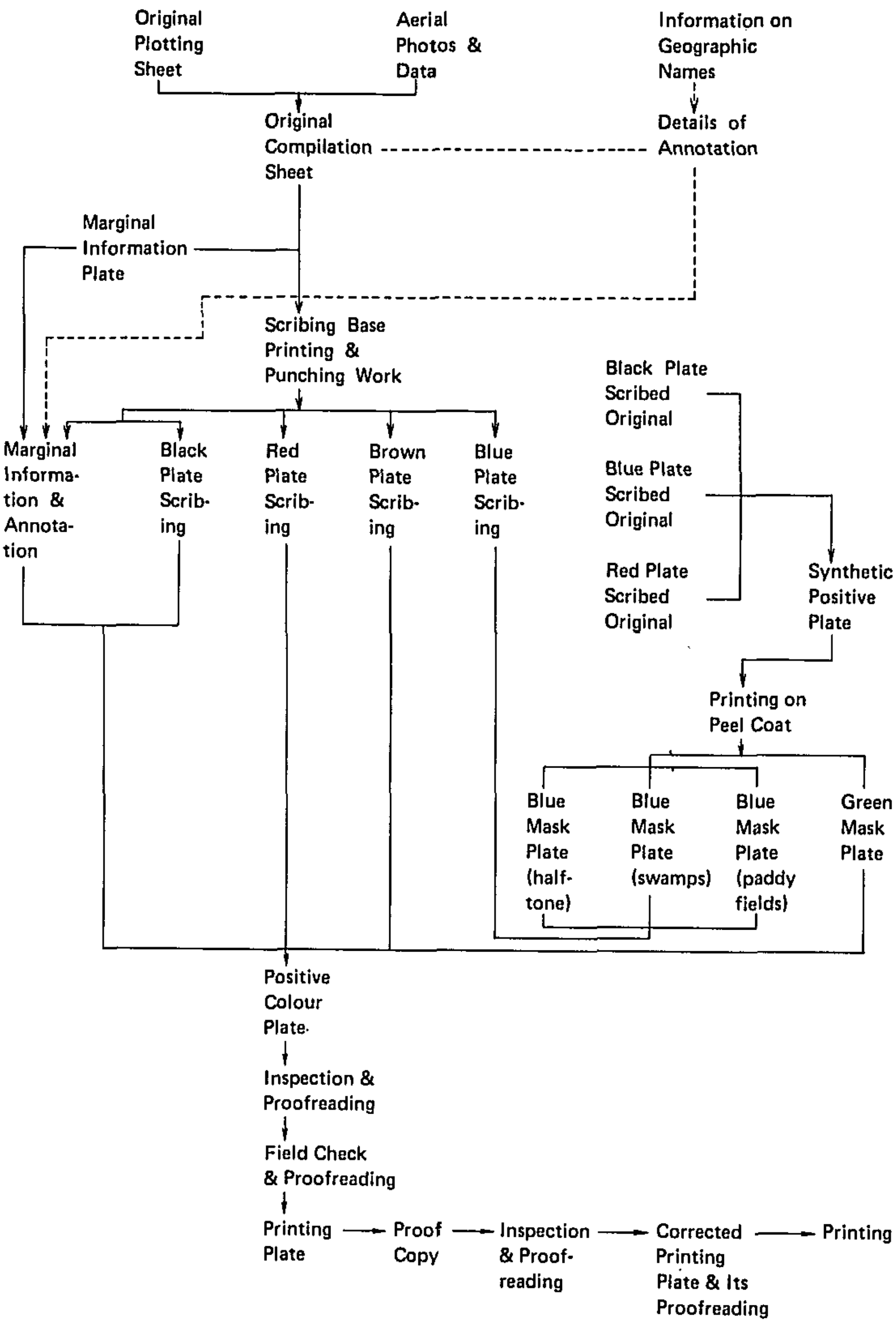
1) The editing sheet, marginal information plate and scribing base are subjected in advance to simultaneous punching work. Then, the editing sheet is printed on each scribing base by the Diazo method. Printing should be performed for all the scribing plates.
2) Positive marginal information plate is reproduced by the photomechanical process, and annotations and signs of a same colour are stuck on its base.
3) Scribing work is performed on each printed base of black, blue, red, and brown plates. In this case, the signs and lines should be so scribed that their size and thickness will conform to the map symbols.
4) Mask plates are prepared by producing a composite positive plate from the scribed base by the photo-mechanical process (triple printing) and by printing it on the peel coat. In this process, the film on the upper surface of the peel coat is cut by the delineations scribed on the positive plate and the desired portions of the film are peeled off.
   Besides the mask plates, land symbol plates (paddy fields and swamps) equivalent to them in size are prepared. In the case of swamps, for instance, the printing plate can be prepared by composite photo-mechanical process if the portions representing swamps are peeled off from the peel coat.
5) Positive colour plate is prepared by synthetic photo-mechanical printing of all scribing base, mask plates and marginal information plate. Final inspection is conducted on the copy produced from this plate.

2. Map Symbols

Symbols are indicated according to the system adopted in Indonesia.

In scribing the red plate, roads were classified into P.L.L. roads, asphalt paved roads and unpaved roads, and educational institutions were indicated only if they were

Flow Chart of Plotting, Compilation, Scribing and Printing Process



equivalent to or higher than high school. As for temples, churches, markets, hospitals, post offices, factories and cemetaries, only large ones were indicated.

In the case of the black plate, vegetation was not indicated in detail but classified into upland field, virgin forest, coppice forest, wasteland, grassland, mangrove forest, coconut palm plantation and rubber plantation, and these were indicated only if the size was larger than 5 mm^2 on the map. On the positive plate, marginal information, annotation and vegetation cover were indicated.

On the blue plate, rivers longer than 1.5 cm on the map were scribed. As for paddy fields and swamps, photo-mechanical synthetic method was employed in the plate making process using the mask plate and land symbol plate.

On the brown plate were indicated contour lines, with light contour interval set at 25 m and index contour interval at 250 m.

11. PROCESS CONTROL

Process Control is exercised over each stage of the map production work as explained below.

11.1 Purpose of Process Control

Since map production calls for smooth and successive completion of many processes, adequate and careful control of each process must be ensured to prevent rejection at any stage which impedes the progress of the whole work.

The following two are the main objectives of process control.

1) Checking of the progress of each process to cover its delay, if any, so as to minimize the resultant time lag effect on the subsequent processes.
2) Checking of the quality of work to assure that the final result will have an acceptable accuracy ahd conform to the specified standard. Stage-by-stage checking is not only necessary for this purpose but effective for preventing rejection.

Process control of photogrammetry is featured by the following points.

1) Since the greater part of the work excluding photographing and field reconnaissance is performed within office, the effect of weather is limited and the progress of work can be controlled with relative ease.
2) Control over the whole mapping process is of great importance because the work is divided into a substantially large number of workers.
3) The processes from photographing to drafting are very closely interrelated so that the work schedule of each process should be planned carefully to prevent any delay.
4) Since many expensive instruments and equipment are used, a heavy economic loss could be incurred by waiting time. In other words, photogrammetry is more like factory production than the conventional ground surveying method. Hence, adequate process and production control is required not only for economic reasons but to improve the accuracy of mapping work.

In the following sub-sections, cautions and points to heed in the process control of photogrammetry are described from the standpoint of inspectors and superintendents without touching on detailed professional instructions.

Since it entails extreme difficulty to correct a completed map, work in each process should be properly controlled from the initial stage.
The result of surveying is a chain of outcomes of respective processes, and its validity is determined by the weaked part of the chain. Each process should therefore be carefully checked and controlled before proceeding to the next.

11.2 Control of Photographing Process

Since the waiting time for suitable weather condition accounts for the greater part of the time required for photographing work, the following conditions should be satisfied to cut the time down to a minimum.

1) The plane's capacity is satisfactory.
2) The waiting place (photographing base) is favourably situated to find out the changing climatic condition of the area to be covered.
3) Instrument and equipment are kept in perfect working condition and workers are properly assigned so that photographing work can be performed whenever weather permits. In particular, constant care is given to the camera to maintain it completely free from any failure.

11.3 Process Control of Control Point Indication and Signal Installation

Control point survey is identical to the ordinary ground surveying, but indication of control points on aerial photos calls for the installation of signals and pricking work. Since the accuracy of photogrammetry is affected directly by how control points are indicated on the photo, the sketch map of control points needs to be prepared with prudent care.

11.4 Control of Aerial Triangulation Process

Aerial triangulation allows for relatively easy process control because it is most well automated by the use of computers in the whole photogrammetry work. The accuracy of work in this process can be checked by the residual in the control point survey as well as the discrepancies between tie points.
Errors in control points arise mostly from erroneous confirmation or indication of points on the photo. If any major error is detected, therefore, it is necessary to check if points are correctly indicated. Even in this case, however, it is prohibitive to pick up control points without good reasons or to make corrections not theoretically justified.
Inspection of the instrument and adjustment of the focal length are another important point to be observed for satisfactory process control. This is essential for plotting work, but carries a heavier weight in the aerial triangulation.

11.5 Control of Field Reconnaissance Process

Field reconnaissance is conducted to cover items of the greatest importance within a minimum of time, so that it calls for sufficient prior photo reading. Specific importance should be placed on geographic names and administrative boundaries which are not clear on the photo. In addition, enlarged photos should be prepared for interpretation in the course of the reconnaissance which should be planned with an ample time allowance.
Sending the photos to the survey points after departure of the operation party does not ensure smooth process control, nor does it yield satisfactory results.
Sequence and method of the field reconnaissance as well as important districts to be

covered need to be selected in advance on the basis of englarged aerial photos, so that all the representative points may be checked against the findings of prior photo interpretation.

11.6 Control of Plotting, Compilation and Scribing Process

Processes involved in photogrammetry are standardized and unifed to a substantial degree. Process control and inspection of the plotting work can therefore be performed by the sampling inspection described in Section 24–7 with stress placed on omissions and erroneous indications.

Rechecking the original sheets in the plotter is an effective way to inspect the plotting work. It is therefore advisable to incorporate such rechecking in the process control for execution by the groups equipped with plotters.

III. APPENDIX

1. OUTLINE OF GEODESY

1–1 Curvature of the Earth
1–2 Datum of geodetic works
1–3 Longitude and latitude
1–4 The coordinate system and projection
1–5 Mean sea level

2. OUTLINE OF PHOTOGRAMMETRY

2–1 Definition and classification of photogrammetry
2–2 Principle of photogrammetry
2–3 Characteristics of photogrammetry
2–4 Problems in photogrammetry
2–5 Limit in accuracy of photogrammetry

3. OUTLINE OF PHOTO-INTERPRETATION

3–1 Classification of photo-interpretation
3–2 Procedures of photo-interpretation
3–3 Elements of photo-interpretation

1. OUTLINE OF GEODESY

1–1 Curvature of the Earth

Unlike the simple surveying and mapping of a small area, the precise surveying of a larger area is influenced by the roundness of the Earth. This influence is even greater than generally supposed. Several examples follow:

1. Apparent horizon which is observed by a standing man on the sea shore is about 1.7 degrees lower than the theoretical horizon.
2. Directions of nadir lines at two different points on the earth's surface incline toward each other at about 0.55 degrees per one kilometer.
3. The length between two points at an altitude of about 700 meters is about 1/10,000 longer than that projected at sea level.
4. Imagine a triangle whose sides measure 50 km placed on the earth's surface; the sum of the three inner angles is larger by 0.7 seconds than 180 degrees.
5. It is not possible to project the surface of the earth on a strict plane without getting a distortion. Therefore, there is some distortion between the local plane coordinate system of X and Y and the longitude and latitude of the earth.
6. The geodetic line between two points on the surface of the earth (which is the shortest distance), does not coincide with the geometrical straight line connecting the two points.

1–2 Datum of Geodetic Works

1. Geoid.

The shape of the earth is defined by the undisturbed state of the sea's surface around the world. This is called "geoid." (Land height in this case is disregarded.) Although the geoid has a gravity equi-potential aspect, it has an irregular shape due to the irregular distribution of materials inside the earth. Therefore, it is not proper to adopt it as reference datum for surveying

2. Reference Ellipsoid.

To represent the surface of the earth by a geometrically regular surface, the simplest method is to use a sphere. But for survey datum, a flat rotational ellipsoid is generally adapted where the south and north act as the axis of rotation. This is called reference ellipsoid. Several examples of reference ellipsoids generally used are as follow:

International ellipsoid (Hayford's ellipsoid)

Major semi-radius	(a = 6378388.000m)
Minor semi-radius	(b = 6356911.946m)
Ratio of flatness	($f = a - b/a$ = 1/297.0000000

Bessel's Ellipsoid

Major semi-radius	(a = 6377397.155m)
Minor semi-radius	(b = 6356078.963m)
Ratio of flatness	(f = 1/299.1528168)

The above reference ellipsoids are to be decided in such a way that their positions and directions are in the best conformity with the shape of the earth.

1–3 Longitude and Latitude

Positions on the surface of the earth are represented by longitude and latitude. The latitude is measured from the equator along the direction of the meridian; the longitude is an angle between the Greenwich Standard Meridian and the meridian passing through the observation point. However, there are several kinds of longitude and latitude such as geodetic, geographical, etc. and as they do not correspond with each other, the best one must be adopted.

1. Geodetic longitude and latitude.

This is the longitude and latitude obtained by geodetically surveying angle and distance on the ground starting at a certain point (the origin). The longitude and latitude of the original point which is decided by astronomical observation will, therefore, coincide with the geodetic longitude and latitude only at the datum point. At other points, however, they will not always coincide with each other due to the vertical deflection even if there is no observation error. The large accumulated errors of geodetic coordinate, the longer the geodetic net may be so the twist of the geodetic net is corrected by Laplace condition equation. This points are called Laplace Station, where longitude and latitude, both astronomical and geodetic are surveyed.

1–4 The Coordinates system and projection

To represent the earth's surface on a flat map, it should be projected on a plane or a cylinder or conic surface by the proper method. It is not mathematically possible to project a spherical surface on a plane or cylinder without encountering distortion. Since the map represents some projected part of the earth's surface, the use should be made of such projection methods as to insure that the acreage, format, angle, azimuth and the distance are as least distorted as possible in the area concerned.
While each country uses different projection methods, the Universal Transverse Mercator Projection (U.T.M.) is widely used and may be considered international. By this method, the world is divided into sectors of six degree intervals along the longitude and these sectors are projected on a cylinder touching the central meridian of the sector. This is called the UTM Projection where the axis of this projection cylinder, unlike the usual Mercator Projection, is perpendicular to the rotation axis of the earth. By UTM projection, the distortion of the length increases toward the end of the sector. Therefore, in order to equally distribute the distortion, the length at the central meridian is shortened by 4/10,000 than the real length and at the end of approximately three degrees from the center it is made longer by about 6/10,000. If the UTM coordinates are applied to express a position on the earth, they may create discontinuity between adjacent sectors but not within one sector. But this discontinuity cannot be avoided when the spherical surface is projected on a flat plane. Expression by longitude and latitude shall be, therefore, used to avoid the discontinuity.

1–5 Mean Sea Level

As the datum plane the mean sea level shall be determined from the results of tidal observation. The tides, that is the change of the sea surface, are influenced not only by the attraction of the sun and the moon but also by seasonal winds, change of temperature, tidal current and waves. Therefore, mean sea level shall be derived from the average of observed value taken continuously for a long period. Data of tidal observation can also be provided if necessary to determine the surface of the lowest low tide or the highest high tide. There are various periods in tides according to their causes such as the sun, moon, and other heavenly bodies and the topographic character of the sea which is called component tide. The mean value of each component tide msut be determined on the basis of data observed longer than at least one period. That is, the minimum intervals of observation must be for more than one month though its mean value may, to some extent, still be subject to the influence of component tides with a period of longer than one month. So, it is desirable to use the data which was observed for over a one year period. Main component tides and their periods are as follows below. (Theoretically, the amplitude of a tide cannot exceed one meter, but it so happens that the real tide amounts to three meters or even more than five meters due to influence of the topography of the sea bottom and the effects of the rotation of the earth.)

NOTATION	TIDE COMPONENT	PERIOD
M_2	Principal lunar semi-diurnal tide	12.4206 hr.
S_2	Principal solar semi-diurnal tide	12.0000 hr.
N_2	Principal lunar elliptic component tide	12.6583 hr.
K_2	Solar and lunar semi-diurnal component tide	11.9672 hr.
O_1	Principal lunar diurnal tide	25.8193 hr.
P_1	Principal solar diurnal tide	24.0659 hr.
K_1	Solar and lunar diurnal component tide	23.9345 hr.
S_{sa}	Solar semi-annual tide	182.6211 days
S_a	Solar annual tide	365.2422 days

2. OUTLINE OF PHOTOGRAMMETRY

2–1 Definition and Classification of Photogrammetry

Photogrammetry is defined as a technique by which to measure quantitative size, shape and position of an object; or, to identify qualitatively what is involved in the photographic image. Here, we shall be mostly concerned with the former case.

The plotting process of photogrammetry consists of four steps: photo taking, observation, computation and adjustment, and compiling. With these processes, not only planimetry but also height measurements, contour mapping and drawing cross-sections are all possible. However, several ground control points are still indispensible for carrying out photogrammetry. Photogrammetry can be classified according to the instruments employed, purpose, etc. However, the utilization, purpose and accuracy of photogrammetry are changeable according to the method of taking the photographs. Therefore, photogrammetry is classified by the method of taking the photographs. Photographic methods are as follow:

1. Terrestrial Photogrammetry.

This method, which is older than aerial photogrammetry, is not a major part of photogrammetry today. However, it is still applied in super large scale precise surveying since it is the best way for precise surveying of limited areas, structures and smaller objects.

2. Aerial Photogrammetry.
   This method is widely employed and suited to surveying wide areas and large scales. This is because aerial photos can be taken of large areas in a short period of time. Recently, the taking of superlarge scale photos from a helicopter and the application of long focal lengths are being put to practical use.
3. Applied Photogrammetry.
   The use is also increasing of surveying by X-ray, micro-photograph and artificial satellite. These are utilized in such fields as civil engineering, architecture, medical sciences, etc.

2–2 The Principle of Photogrammetry

The principle of qualitative photogrammetry for mapping is based on a kind of method of intersection which is an application of triangle geometry. Plane table surveying and triangulation are also applications of trigonometry. But, in the cases of plane table surveying and triangulation, only the two dimensional coordinate system of X and Y is obtained by assuming a triangle on the plane; photogrammetry, on the other hand, addresses itself to the simultaneous determination of the coordinate system of X, Y and Z on the basis of the three dimensional triangle. Therefore, in photogrammetry, the photo station (camera station) in the air is not determined; so, we must determine this point. It is determined by the method of resection. The method of resection involves observing three points which are known from an unknown point. Then, the unknown point is determined.

However, as the position of the camera station in the air is not known, this should first of all be determined.

Comparing the relative position of two consecutive photographs, a model is made by deciding its relative inclination, rotation and displanement of camera. This is called relative orientation. Next, the position and scale of the model are determined by means of control points on the ground. This process is called absolute orientation. The operating of this orientation is based on stereoscopic observation where two photos are simultaneously observed with both eyes in the stereo plotting instrument. Stereoscopic observation is such that the same object photographed from two different and slightly remote points and observed separately by the left and right eyes, brings out the image with a depth impression as if the object existed in the viewers presence. This principle enables one to measure depth (the height in case of an aerial photo), and also improves the accuracy and efficiency of measurement. Height, being surveyed in this way, it is also possible to draw contour lines which connect all points at a certain height. In doing stereoscopic measurement, the object should be imaged on two consecutive overlapping photographs. So, the phtographs should overlap by over 50% in order to cover a certain area by stereoscopic observation. If overlap is less than 50%, part of the area survey will not be covered on two consecutive photos. Therefore, stereoscopic observation will not be possible. In extreme cases where two photos are completely overlapped, that is, 100%

overlapped, stereoscopic observation is also impossible because the photos are completely the same. Taking into account all of these factors, the overlap rate is generally set at about 60%. According to the above principle, two photos whose areas overlap are placed on a stereo plotting instrument and then they are adjusted so that the relative position and condition of tilt of the camera is reproduced just as it was when the photos were originally taken. From this we can determine any ground point positions by means of tracing back the incident of light rays through both lenses of the stereo plotting instrument.

2–3 The Characteristics of Photogrammetry

Photogrammetry has the following merits:

1. It is efficient and economical. It is possible to quickly photograph the survey area from the air. Since almost all the processing is carried out in the laboratory, the amount of field survey work can be minimized. Survey projects are, therefore, relatively free from such external factors as weather conditions, etc.
2. Uniform accuracy is obtained. A conventional procedure is plane table surveying begins with ground control points and ends up with map details — accumulting errors gradually. This does not always ensure that the accuracy of plotting will be unifrom overall because during plotting, errors accumulate gradually. In the case of photogrammetry, topography and planimetric details are drawn directly from orientation with several ground control points and therefore errors remain uniform everywhere.
3. Contour lines are plotted directly. Quantitative accuracy of contours by plane table survey is not very high because they are drawn by interpolation from several primary points which are only directly observed. On the contrary, the accuracy of photogrammetric contours are high since they are directly and continuously drawn as the locus of constant height on the terrain observed with a measuring mark in the plotting instrument.

2–4 Problems involved in Photogrammetry.

Regarding the accuracy of photogrammetry, it should not be applied to mapping on a scale of more than 1/500 and a range of contour lines of less than 1.0 meter (except in experimental and research work). Since there is difficulty in identifying small objects on the photograph, photo-interpretation is limited according to the scale of the photo. So, mapping only by the photo-interpretation procedure has limitations. For a detailed survey, it is necessary to carry out field surveying to complete the detail representation.

Special equipment such as airplane, air camera stereo-plotting instruments, etc. are also essential but they are very expensive compared with other conventional instruments and amount to thousands of dollars.

The larger the mapping area, the more economical photogrammetry tends to be. On the contrary, the smaller the area, the more costly it becomes. For an area of less than one square kilometer, a field survey is sometimes more practical. Photographing and mapping of large areas if done all at one time costs much less.

While topographic features of the mapping area do not affect the efficiency of the mapping work considerably, mapping costs are higher in developed areas which include more mapping details than the less developed areas do. Efficiency may decrease and expenditures become greater if the format of the area is irregular or narrow.

The efficiency of air photography is largely influenced by the seasonal elements such as weather conditions, altitude of sunshine, etc. Not only the seasons but also the location affects weather conditions. Further, in zones of high and middle latitudes, if photos are taken when the altitude of the sun is low, terrain details become obscure due to dense and long shadows of mountains or exposure becomes insufficient or the surface of the ground becomes covered with snow. It is, therefore, recommended that the best season be selected for photographing; this selection will be influenced by the location.

Although the amount of field surveying can be reduced through application of photo-interpretation (except for the precise expression of terrain details), it is still necessary to carry out field surveys to complete the results of photo-interpretation.

As in the case of terrestrial surveys, the quality of the map completed is hard to evaluate at one's desk (especially the accuracy). Therefore, a plotting check and a field check should be carried out during each part of the work and/or the inspection should be assigned to a qualified institution.

2–5. Limitations in the Accuracy of Photogrammetry.

The accuracy of photogrammetry based on the observation of photos is subject to the physical and chemical restrictions of the various materials and instruments used for photographing and mapping.

1. Photo Emulsion

   Sensitive emulsion of photos consists of fine particles of black and white grains which can be observed through a magnifying microscope. Objects smaller than the size of the grain cannot be observed.

2. Error of Lens.

   The lens of the camera has a very small distortion; however, there is a distortion of about 4–10 micron and the pattern of this distortion differs slightly lens by lens.

3. Error of camera.

   It so happens that the optical axis of the lens system and the axis of the camera (when lens is attached to camera) do not coincide completely and the focal distance differs slightly from the expected value.

4. Shrinkage of Film.

   The film base plastered with emulsion is usually flat and not very shrinkable; however, there remains a little shrinkage and unflatness in it. Uniform shrinkage does not affect accuracy so much but local and irregular shrinkage does affect accuracy. The irregular shrinkage dose not affect the dry plate much – but, in the case of film it may amount to 10 or 20 microns. This irregular shrinkage affects not only negative film but also positive film.

5. Other Errors Affecting Aerial Photos.

   a. Atmospheric refraction becomes about three times greater at the margins of the lens and the film of the super wide angle camera.

   b. The influence of the curvature of the Earth. Due to the spherical surface of the Earth, the central part of the model is swelled a little compared with the margins. In the case of super wide angle photographs with a scale of 1/50,000, the center is about 3 meters higher than the margins.

c. Image Movement. Due to the movement of the airplane and the oscillation of the camera, the quality of the photo decreases.

6. Error of Plotting Instruments.
The error of 1st order instruments is about 5 to 10 microns and for 2nd order instruments it is about 10 to 27 microns; and, about 3 microns for the stereo comparator.

7. Observation Error.
It is said that the resolving power of the human eye is one minute; and by stereoscopic observation it is about 0.5 minutes (one centigrade).
Observing the photos from a distance of 15 cm, they correspond to the errors of about 20 microns and 10 microns on the photo respectively; and, the errors can be reduced 1/4 by observing the photos through a lens with a magnification of four times.

8. Ground Control Point Surveying.
A part of the error in photogrammetry comes from errors in triangulation points, bench marks and observational errors of the air photo signals on the air photos. The size of the trigonometric monument is already about 10 cm x 10 cm in superficies without the precise identification of the center of the monument. Therefore, it is almost impossible to reduce the identification error less than 10 cm which is equivalent to 10 microns at a scale of 1/10,000. In summing up, it is appropriate to assume that the limit of error in photogrammetry should lie between 30 to 50 microns in photo-coordinates.

3. OUTLINE OF PHOTO-INTERPRETATION

3–1 Classification of Photo-interpretation.

Aerial photo-interpretation which helps to recognize natural states or artificial phenomenon on the ground is a technique to get necessary information by observing and analyzing photo images. The sample information obtained from field surveys is very significant to photo-interpretation; photo-interpretation draws from the aerial photo information relevent to the environment. It must also be emphasized that one must have a proper knowledge of aerial photos and be well versed in their treatment in order to obtain the right and efficient photo-interpretation. At the same time, it is necessary to get well acquainted with the background knowledge of the subject field in order to analyze and interpret the information contained on the photo concerned.

Photo-interpretation may be classified (by purpose) as follows:

a. photo geological interpretation
b. photo forestric interpretation
c. photo soil interpretation
d. military photo interpretation
e. other: e.g. archaeology, disaster, etc.

However, most of the basic procedures are common to all cases and some of them are explained as follows:

3–2 Procedures of Photo-interpretation.

1. Photo Reading.
   Photo reading is an act of recognizing precisely photo images and their positions through observation of photos. Photo reading is mostly applied to topographic surveying of which railroads, rivers, houses, and forests are the objects for interpretation for mapping purposes.
2. Photo Analysis.
   Photo analysis is the procedure of further reading of photos and analyzing the resultant information to get more detailed results.
3. Photo-interpretation.
   Photo-interpretation is the making of a judgement (interpretation) synthesized from knowledge as well as the results of photo analysis. This is done by the combination of deductive and inductive reasoning. For instance, reasoning proceeds as follows: a river terrace is recognized from its form on an aerial photo and further, that the river terrace is composed of rocks and sand.

3–3 Elements of Photo-interpretation.

Basic knowledge of the photo is a prerequisite for photo-interpretation. Such knowledge includes photo scale, photographing seasons and regions, types of emulsions and filters, etc. And, the subject is to be analyzed according to the following elements:

1) tone 2) texture 3) pattern 4) size 5) shape 6) shadow 7) color

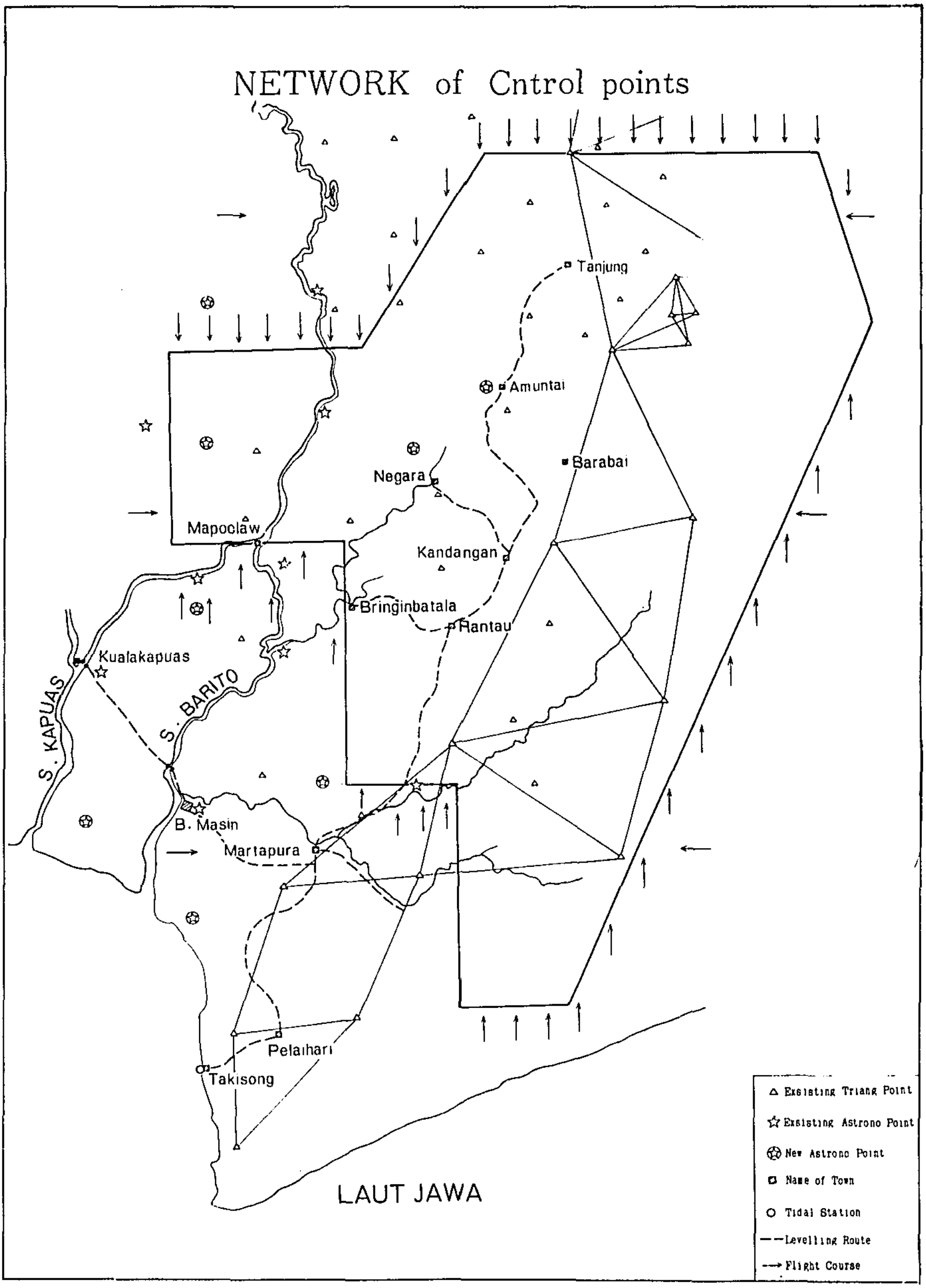
Comments on each element follow: and the final conclusion of photo-interpretation is obtained from the combination of resultant information from all the elements.

1) tone. The tone of a photo indicates the extent to which photo brightness and darkness is identified and is the most basic discerning element on monochrome photos which are influenced by many elements. Above all, the difference of tone occurs as a result of different reflection coefficients and is dominated by the position of the sun at the time of exposure. The type of filter used and the sensitivity of the film also affects the tone of air photos.
2) texture. The texture of photo images is the frequency of change of tone in the complexity of the photo image and is influenced by the scale of the photo. For instance, in small scale photos, each tree in a forest is not distinguishable but in the larger scale photo individual tree crowns can be distinguished and further, the vacant terrace between them can be recognized. In the former case, the individual tree itself makes the texture of the forest an element of a collective object; but in the latter case, each leaf and twig makes up the texture of the tree corwns as a collective object. In the former case, texture itself is characteristic of the kind of trees composing the forest unit. In the latter case, the kind of tree is judged individually based on its form, size, texture, etc.
3) pattern. Pattern is a type of distribution of object on the photo and is also dominated by the scale of the photos.
4) size. The size as well as the form of the photo image is a very important element in photo-interpretation. Horizontal and vertical distance between each point on the surface of the ground can be measured on the air photo with its gimutal direction. Further, inclination can be estimated on the basis of the above mentioned data or by other methods.

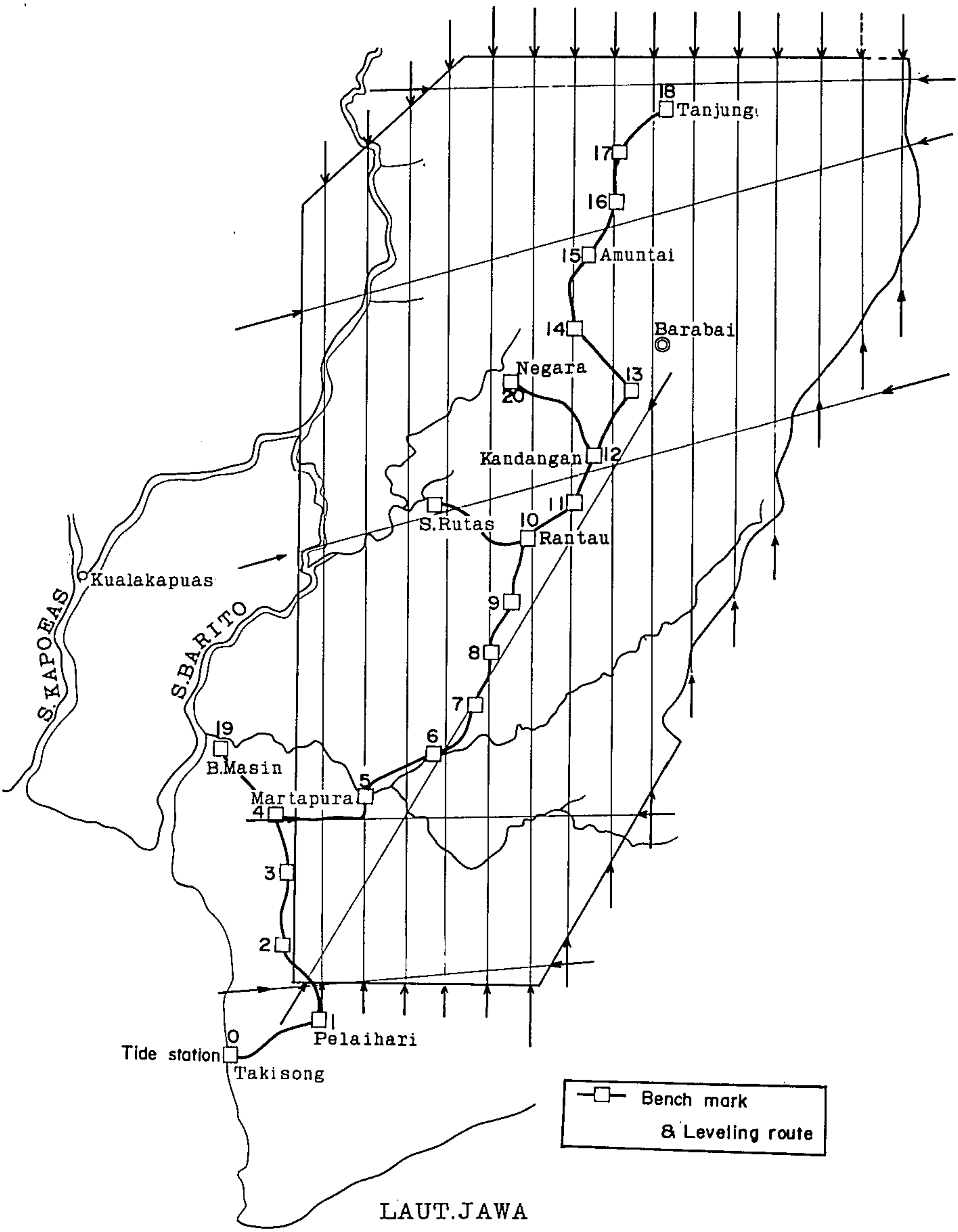
5) shape. As we observe the object from horizontal or oblique direction in daily life, the shape viewed from above (as in the case of aerial photos) may seem strange. But when aerial photos are employed the planimetric format of the object photographed from above becomes a powerful clue in photo-interpretation.

6) shadow. Shadow plays an important role in identifying size and form of an object in vertical aerial photographs. For instance, the form of trees can be recognized by the shape of their shadows and from which the kind of trees can also be judged. Usually, aerial photos are taken during mid-day when the sun is higher than a certain altitude so that the shadow effect is of minimum influence to the photo images. On the contrary, photographing in the morning or evening when the sun is at a low altitude results in heavy shadows on the surface of the ground; but detailed relief of trees or terrain can be recognized clearly by the shadows which enable us to distinguish the slight difference of tree growth or geological structures, etc.

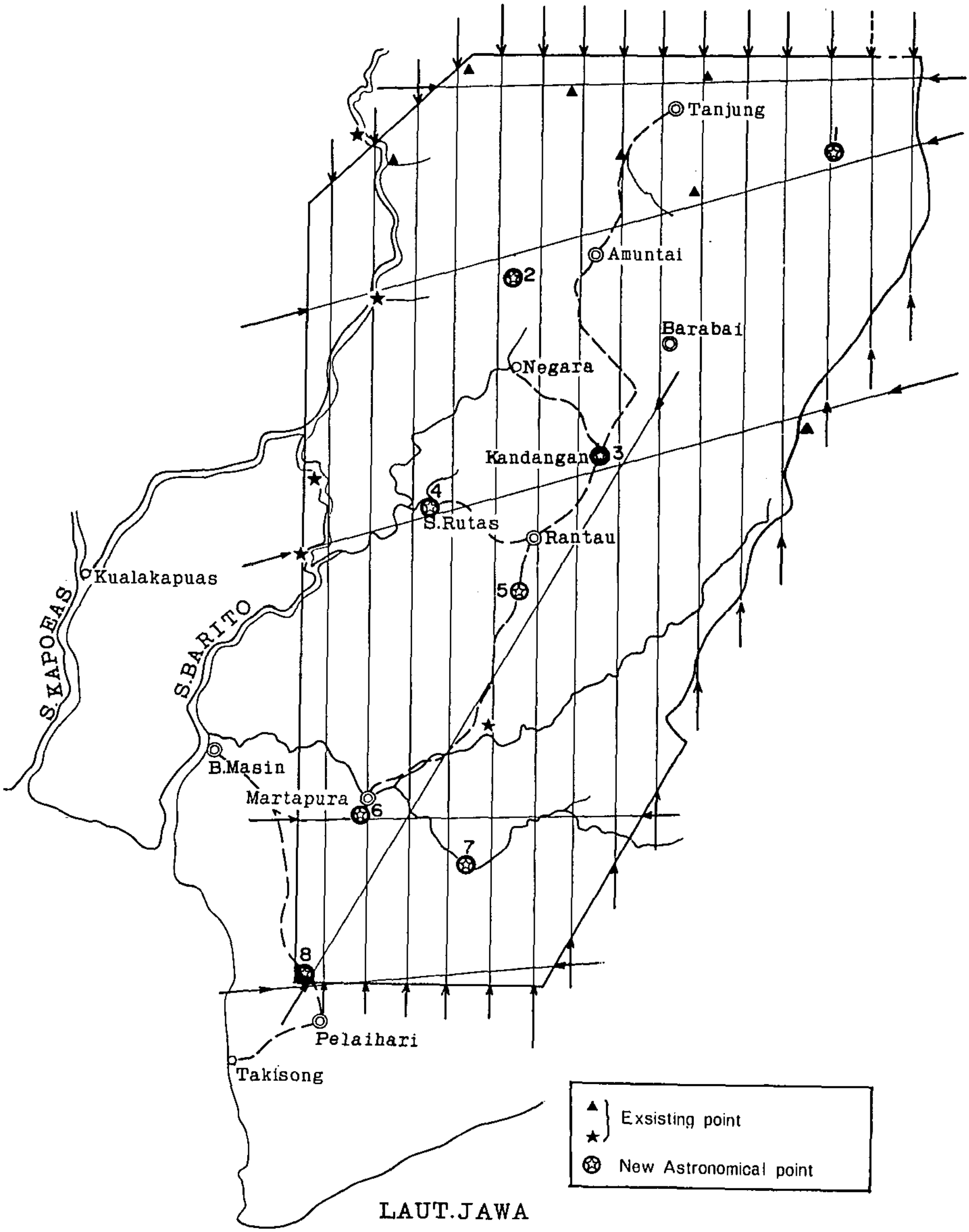
7) color. The above mentioned elements are also applicable to back and white photos as well as color photos; but color photos provide a new photo-interpretation tool for the user. Compared with black and white photos which only provide one dimensional informaiton, color photos provide the possibility of two dimensional information. For instance, it becomes possible to distinguish the green color of plants and the red color of latelite soil, etc. Recently, a new technique of false color was developed. Now it is possible to give a special color to a specific object. For instance, we can give a diseased plant a special color which gives us new possibilities of photo-interpretation.

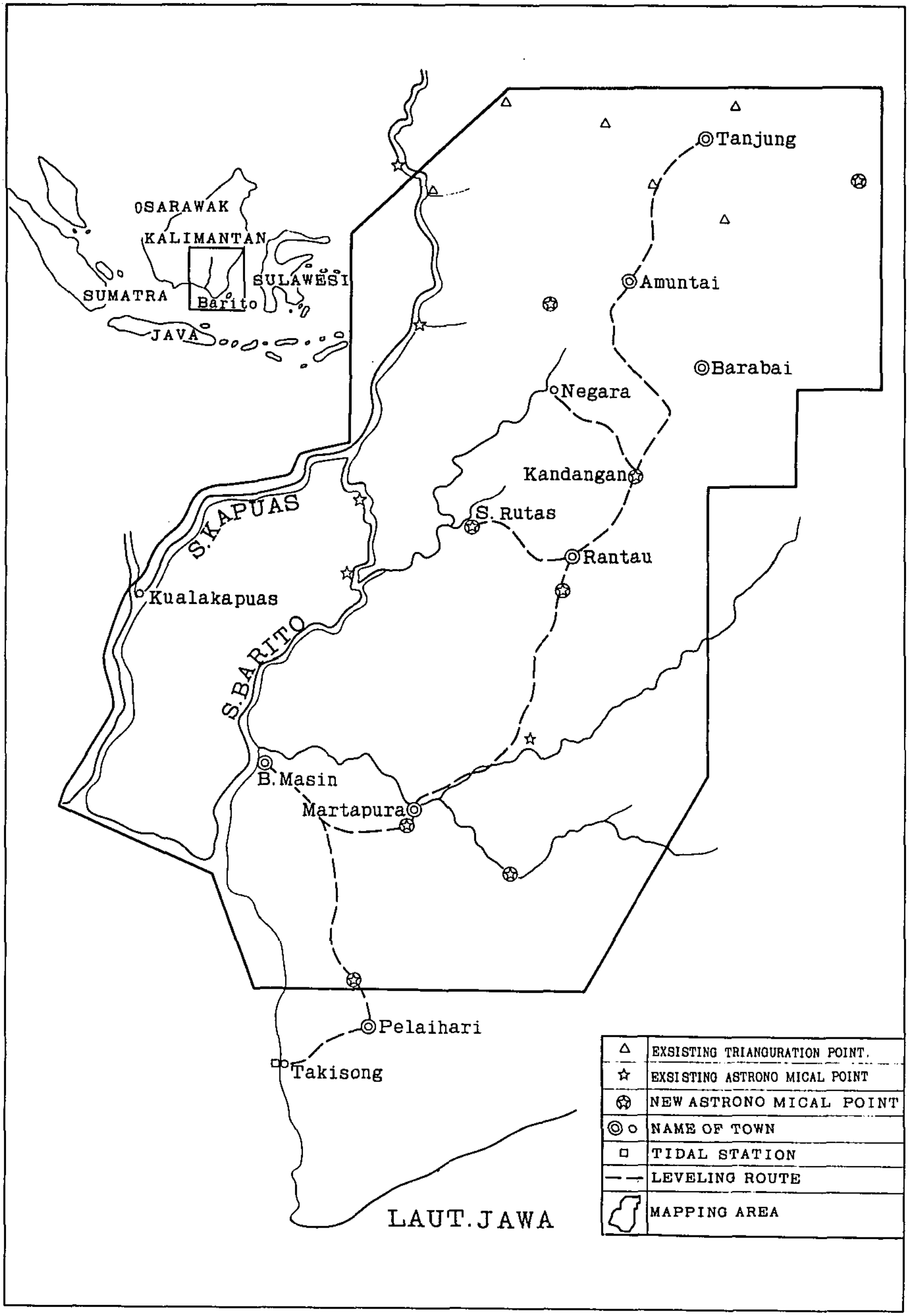

NETWORK of Cntrol points
Tanjung
Amuntai
Barabai
Negara
Mapoclaw
Kandangan
Bringinbatala
Rantau
Kualakapuas
S. BARITO
S. KAPUAS
B. Masin
Martapura
Pelaihari
Takisong
LAUT JAWA
Existing Triang Point
Existing Astrono Point
New Astrono Point
Name of Town
Tidal Station
Levelling Route
Flight Course

Bench mark & Leveling route



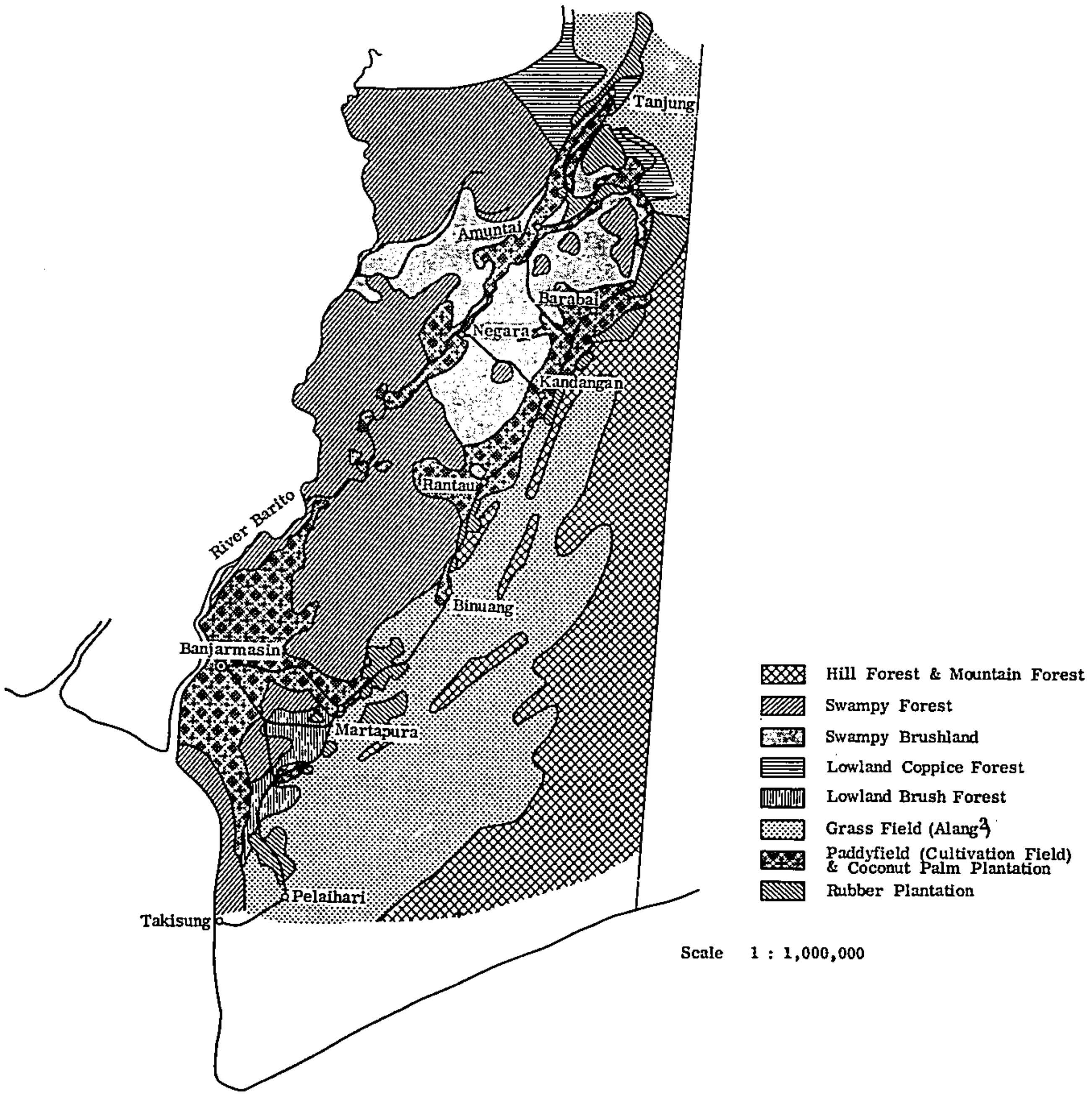
Arrangement of Astronomical position



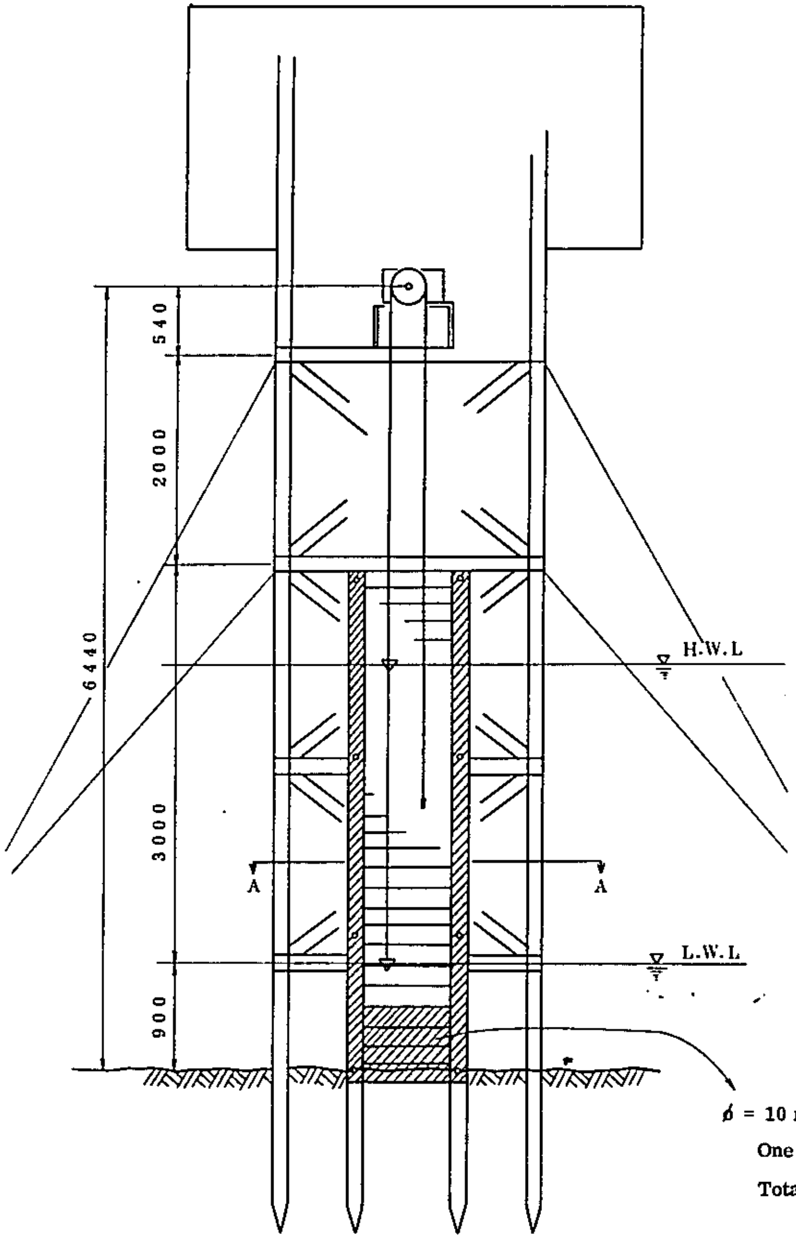


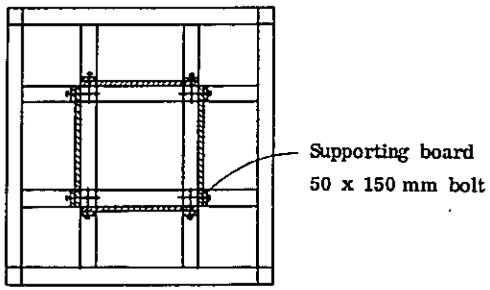
SARAWAK
KALIMANTAN
SULAWESI
SUMATRA
Barito
JAVA
Tanjung
Amuntai
Barabai
Negara
Kandangan
S. Rutas
Rantau
S.KAPUAS
Kualakapuas
S.BARITO
B.Masin
Martapura
Pelaihari
Takisong
LAUT.JAWA
EXSISTING TRIANGURATION POINT.
EXSISTING ASTRONO MICAL POINT
NEW ASTRONO MICAL POINT
NAME OF TOWN
TIDAL STATION
LEVELING ROUTE
MAPPING AREA

Vegetation Map in the Barito Valley



Structure of Station





ϕ = 10 mm (water hole)
One side : 20
Total : 80 Nos

Sketch Map of Takisong Tide Station