A COURSE IN GEODETIC SURVEY

Training Material

JAPAN INTERNATIONAL COOPERATION AGENCY

GEOGRAPHICAL SURVEY INSTITUTE

MINISTRY OF CONSTRUCTION

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PART I.

ESTABLISHMENT OF
THE PRECISE GEODETIC SURVEY NETWORK OF
THE JAPANESE ISLANDS

PREFACE

Japan's network of geodetic control points, established half a century ago during the late Meiji and early Taisho periods, is now losing its function as a network of control points due to numerous natural and artificial obstacles. Consequently, it is a pressing need to revise the Meiji geodetic datum and consolidate a highly precise datum of the Showa period.

The survey techniques which have been developed rapidly in recent years, especially the distance measuring techniques utilizing light, have revolutionalized the field of geodetic survey, drastically improving the accuracy in determining positions and also survey efficiency.

The new "Precise Geodetic Survey Network" will thus play not only the conventional role of providing control points for the preparation of various maps but also that of providing accurate coordinates for cadastration, precise survey of land, large scale or regional remoulding of national land and conservation works. Furthermore, it will detect crustal movements in detail by repeating the survey of the network of control points of high density, thereby prediction of destructive large scale earthquakes may be put into practice and basic data for permanent disaster prevention measures may also be provided. These data may also be used as basic material for solving many academic problems, forming a cultural property of universal value to be left for future generations.

CHAPTER 1. CHARACTERISTICS OF THE JAPANESE ISLANDS

Recently one often hears such a phrase as "Irreplaceable earth". The earth is indeed the irreplaceable planet for mankind. Space explorations carried out by the United States and the Soviet Union have almost confirmed that the Mars, which had for a long time stimulated our imagination that there might be living creatures there, was after all a dead planet without even vegetation, not to speak of animals. As a result, it has become clear that the space which will be navigational for mankind in the near future is entirely a desolate world of death and that the earth will never cease to be our irreplaceable planet even in the distant future.

Similarly, the Japanese Islands provide the Japanese people with irreplaceable land for cultural and economic activities. With its mild climate and varied and beautiful physical features, the land of Japan has influenced its inhabitants who developed a delicate and refined culture. Further, after World War II, with their diligence and ingenuity, these people have built up their economic strength on their land which finds few parallels in the world today.

However, in order to enrich national life, effective development and remoulding of the national land will have to be continued in future. They will then cover not only the land area but also submerged continental shelf areas and require more precise land remoulding works on a larger scale. In parallel with these, the system of environmental conservation recessitated by the gigantic cycle of production and consumption will have to be provided on a large scale. For that purpose, it will be necessary first to ascertain accurately the actual condition of the national land.

The Japanese Islands are part of the Pacific archipelago situated on the tangent line of two large structures, the northwestern part of continental Asia and the West Pacific Ocean, continuing to carry out orogenic activities and forming one of the noted earthquake and volcanic areas in the world. The skeleton of the land consists of recent folded ranges running in parallel to the islands, and the coast line is abundant in indentations forming numerous promotories and inlets due to the uplift and sedimentation of land.

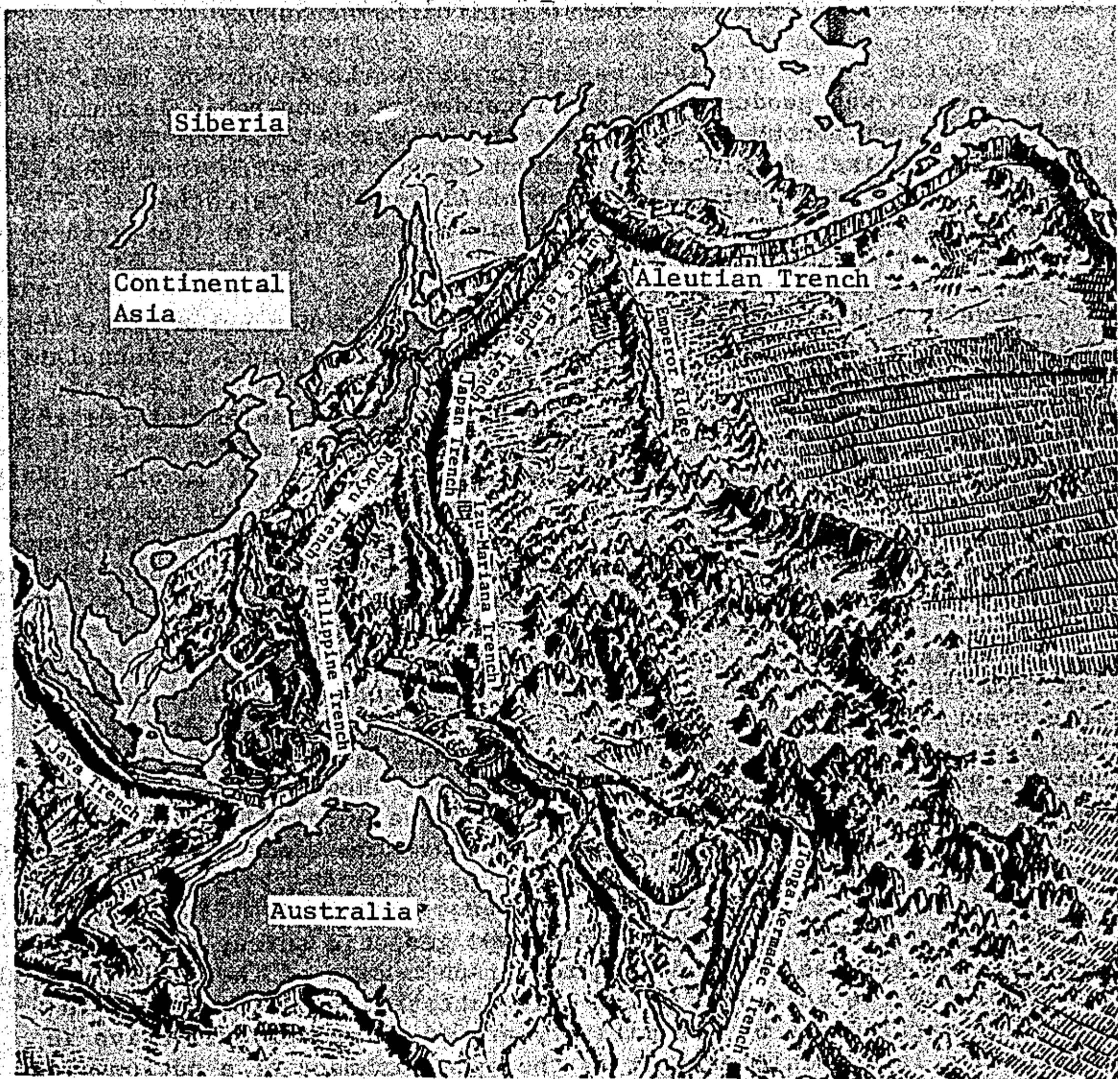
On the ocean side of the Japanese Islands run the Japan Trench reaching the depth of 10,000m, Izu-Mariana Trench Nankai boat-shaped basin and the Ryukyu Trench; then run in parallel with them the gravity anomaly zone, earthquake zone and the volcanic zone. The Japanese Islands may thus be described as "living" land. In fact, there are numerous active faults running reticulately, which may cause tremors at any time.

Most of the Japanese people live on alluvial plains formed during the latest geological period. Since these plains are on such soft ground, they are inclined to experience sedimentation even under normal circumstances. In the case of a large scale earthquake, it is obvious that the soft foundation may cause serious damages such as the destruction of housing and various structures.

Since the dawn of Japanese history, many lives and properties have thus been lost in great earthquakes and volcanic eruptions. These orogenic activities will inevitably continue on a geological time scale and the Japanese people will never be free from the continuous threat of destruction by great earthquakes. There is no other country such as Japan among the advanced nations which is situated in an orogenic activities zone of the peak period.

Tragic results of destructive earthquakes are clear without referring to the extent of the damage caused by the Great Earthquake of 1923. Especially in view of the recent tendency for industrial cities to concentrate in the Pacific belt area which happens to be the danger zone where a large scale earthquake is anticipated, promotion of earthquake prediction is not only an important task for seismologists but also an urgent social need of the Japanese people who have no other alternative but to construct a highly industrial state on these islands.

Accordingly, it is necessary to ascertain the actual condition of the Japanese Islands as a prerequisite for the construction of an advanced welfare state on the islands--irreplaceable national land. For that purpose, the scope of the survey should include, more than a mere static survey of national land, follow-up study of dynamic behavior of changing land and elucidation of its mechanism. This is the problem peculiar to Japan which is fundamentally different from western countries where orogenic activities have already ceased.



Submarine topographic map of West Pacific

CHAPTER 2. SURVEY OF NATIONAL LAND

Section 1. Development of Geodetic Control Point Survey

Geodetic survey provides basis for all works involving land. That is the reason why geodesy was long regarded as a monarchic learning ranking with astronomy and calendar, and it was not without good reason that geodesy first developed in ancient Egypt, the birthplace of civilization. In fact, during the reign of Alexander, Eratosthenes (276-194BC), the forefather of geodesy, estimated the size of the earth with an error of only 16%.

Modern geodetic works began in the 17th-18th centuries, like other sciences, in Western countries. In France, for instance, triangulation was systematically carried out during the 17th century to ascertain the shape and the size of the earth; this led to the establishment of the modern mapping method based on triangulation. In Prussia, modern cadastration based on the triangulation network of first to third order points was put into practice. As these examples indicate, geodesy has been developed and conducted by the State for compiling the basic map necessary for running the country and also as the basis of land measures such as rational collection of land tax.

In Japan, too, with the emergence of the modern government resulting from the Meiji Restoration of 1868, geodetic work was at once put into practice. In 1871, the Survey Department was set up in the Ministry of Construction as an organ for mapping and systematic land survey, and triangulation was thus commenced. However, for military and other reasons, mapping operation has to be hurried, and geodetic works were incorporated into the Army to form the Land Survey Department, the General Staff Office.

The Land Survey Department commenced geodetic survey from around 1882 for the purpose of preparing topographic maps of the scale 1/50,000 and completed the survey in 1916. The fact that geodetic survey in Japan was carried out by the military proved to be very effective in that such a large scale project as the establishment of a nation-wide geodetic network was completed in a short time, and the works carried out by the Land Survey Department formed a significant cultural asset left by the Japanese Army.

On the other hand, the military monopoly of geodetic works which provided the basis for every type of survey caused an important problem for Japan's survey system. That is, land survey became separated from geodetic works and the inaccurate Land Register of the Meiji period had to be left as it was. Further, survey for public works was carried out independently of geodetic survey by the state and its reliability caused often problems.

After World War II, the Land Survey Act was promulgated to facilitate speedy restoration of the devastated national land. It was then decided to place the standard of survey on coordinates of state triangulation points to maintain and standardize accuracy in cadastral survey. Further, public survey such as those for construction projects was to be conducted according to the state system of geodetic coordinates to maintain accuracy. The distorted form of survey of the prewar period was thus at last corrected.

The geodetic triangulation network of the Meiji period consisted of first, second and third triangulation points each forming its network: first triangulation points (average distance 45km; total 330 points), first order triangulation supplementary points (average distance 25km; total 619 points); second order triangulation points (average distance 8km; total 5,048 points); and third triangulation points (average distance 4km; total 32,675 points).

At the same time, first order bench marks (present total 15,000) were established for every 2km on major state roads to carry out leveling with the highest accuracy obtainable in outdoor survey. Thus the First National Survey was almost completed in 1913.

After World War II, the Land Survey was commenced in 1951 for more indense use of land and more accurate cadastration, and to provide a standard for the survey, fourth order triangulation (one for each $2km^2$ for plains and for each $1km^2$ for urban areas) was commenced. The triangulation has completed its first, second and the special promotion phases and is now in the new 10-year program.

Further, from 1961, the National Large Scale Map Project was commenced for the purpose of preparing large scale maps of the scale 1/5,000 and fourth order triangulation was also commenced to provide a standard for aerial photogrammetry. Fourth order triangulation is thus being carried out for both the Land Survey and the National Large Scale Map Project and, as a result, over 32,000 fourth order triangulation points have so far been established. Geodetic coordinates of these points are to be determined from the first, second and third order triangulation points.

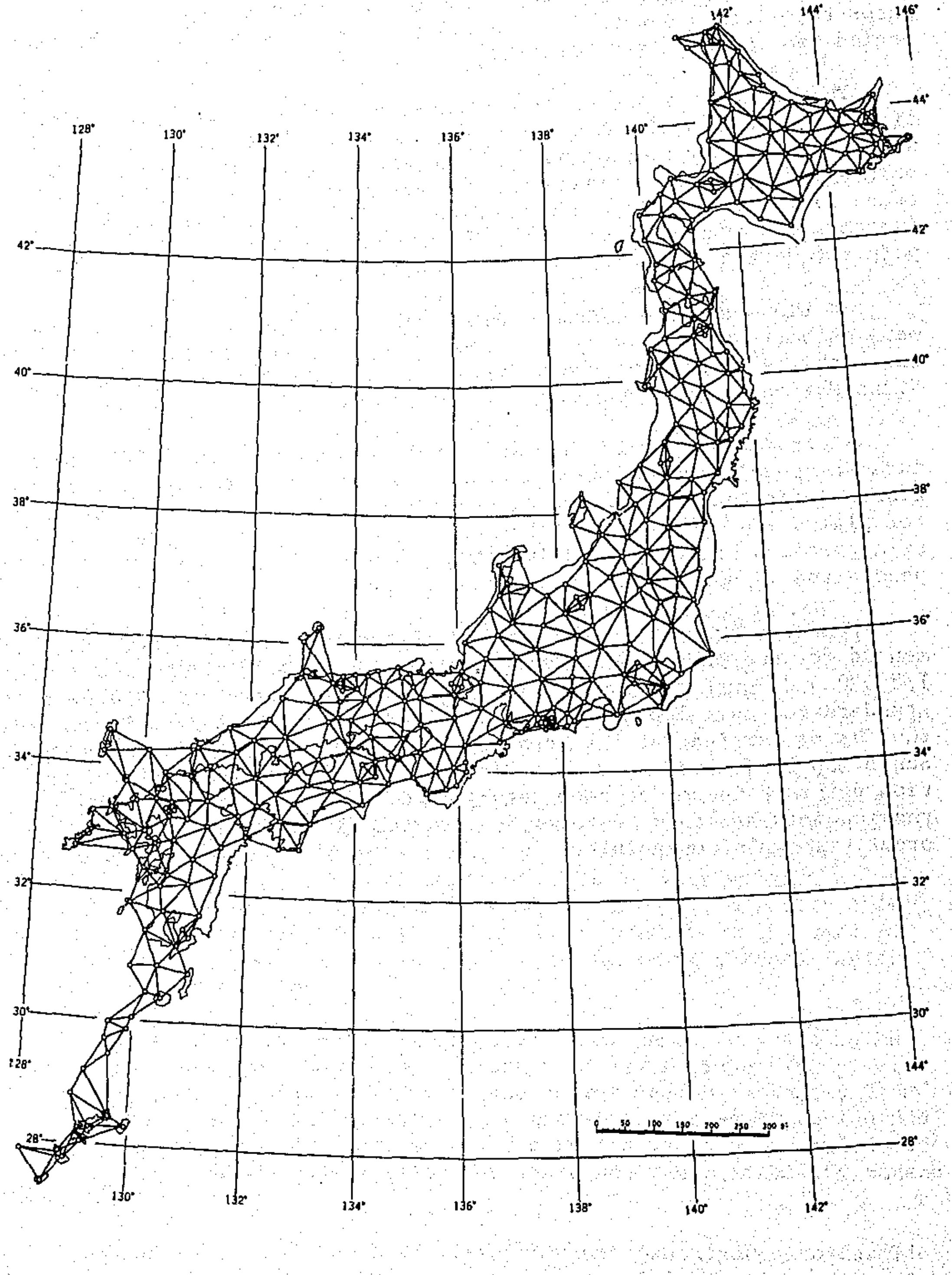


Fig. 1. Index map of first order triangulation network

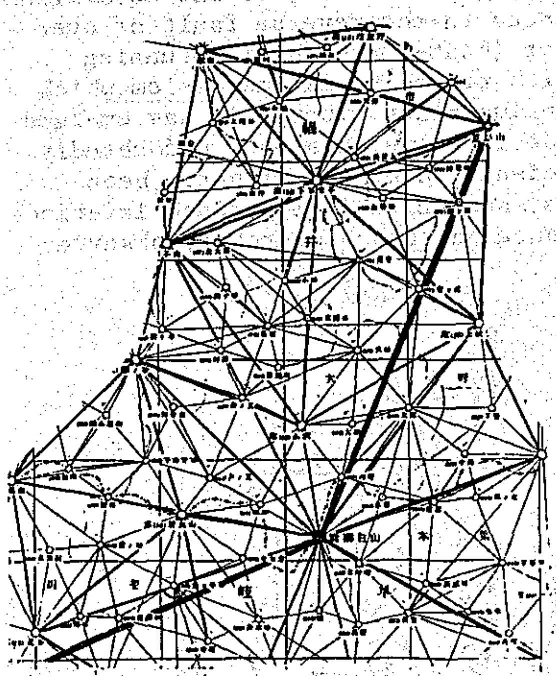


Fig. 2. Conventional second and third triangulation networks

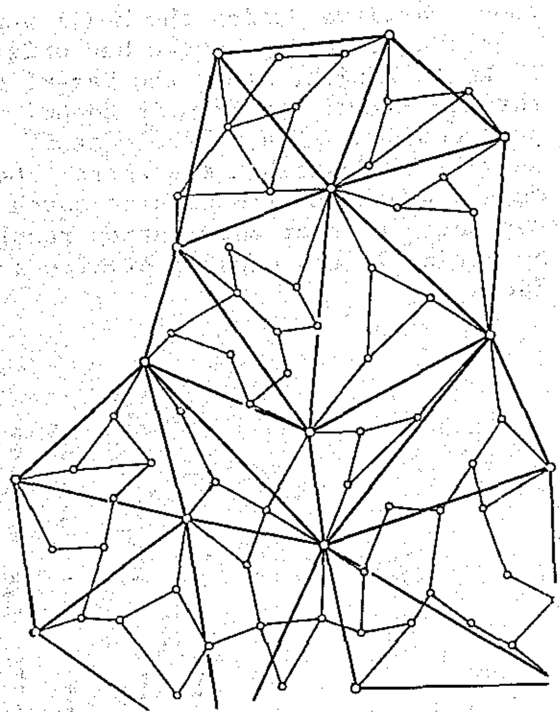


Fig. 3. New precise geodetic network

Section 2. Geodetic Control Point Survey to Contribute to Academic Survey

As has been seen, geodetic control points were initially established for the purpose of preparing medium scale maps of national land. Since the theory and instrumentation techniques are the culmination of improvement and ingenuity extending over the period of 200 years in Western countries, the precision of observation is comparable to that in other precise experiments in physics or astronomical observations. For instance, the precision of the angle measurement in first order triangulation survey is at a high level of ±0.6-0.8" (relative error of several hundred thousandth), and the relative height between two points with a distance of several tens km may be obtained in first order leveling with an error of ±1cm. Accordingly, since prewar days, precise geodetic survey has been used for various academic surveys. In the case of Japan, results of the survey for restoration after the Great Earthquake contributed greatly to the understanding of the mechanism of earthquakes.

Fig. 4 and Fig. 5 show horizontal and vertical diastrophism in the south Kanto area due to the Great Earthquake of 1923. These have been obtained by comparing the results of restoration survey of 850 first, second and third order triangulation control points and bench marks and those obtained during the Meiji period. As a result, it was established that the Great Earthquake had originated in the reverse fault of over 100km in length, along the Sagami traf (boat-shaped basin) running through Sagami Bay from SE to NW. With regard to the mechanism which produces such a large scale fault, an important hypothesis has emerged as will be described elsewhere. Geodetic survey has thus undoubtedly made a great contribution to earth science which had hitherto been dependent on poor observation results and inclined to produce inferences, by giving it contents of precise science based on accurately observed facts.

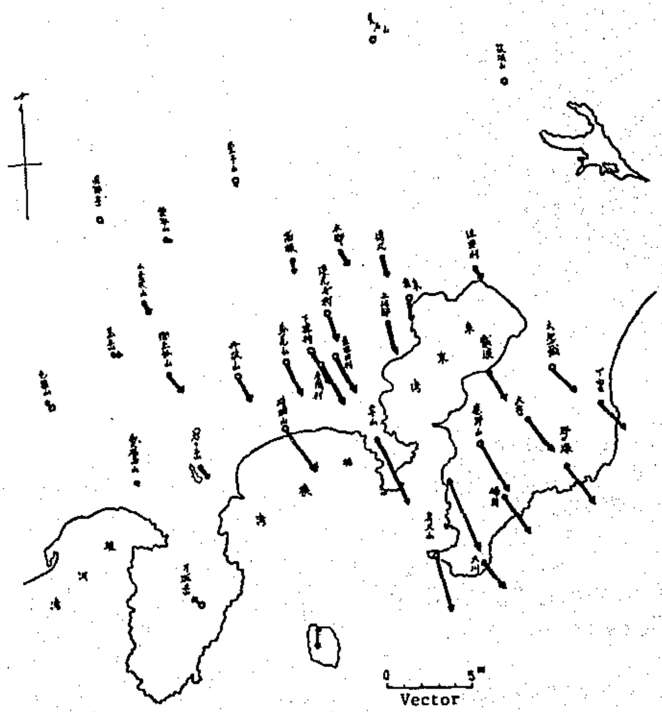


Fig. 4. Movements of first order triangulation control points due to the Great Earthquake of 1923 (assuming that Kokushidake and Teruishiyama were immobile).

After the Great Earthquake, geodetic restoration survey carried out after the North Tango Earthquake (1927), North Izu Earthquake (1930), Nankaido Earthquake (1946) and others elucidated geophysical characteristics of destructive earthquakes which occur in Japan. For instance, abnormal diastrophism was observed about 10 years prior to the Niigata Earthquake of 1964 (Fig. 21), and survey of Matsushiro sporadic earthquakes (1965-) produced significant results. These findings resulted in the inauguration of the Earthquake Prediction Committee, the first of its kind in the world, consisting of various related Government agencies, to carry out joint research for practical application of earthquake prediction.

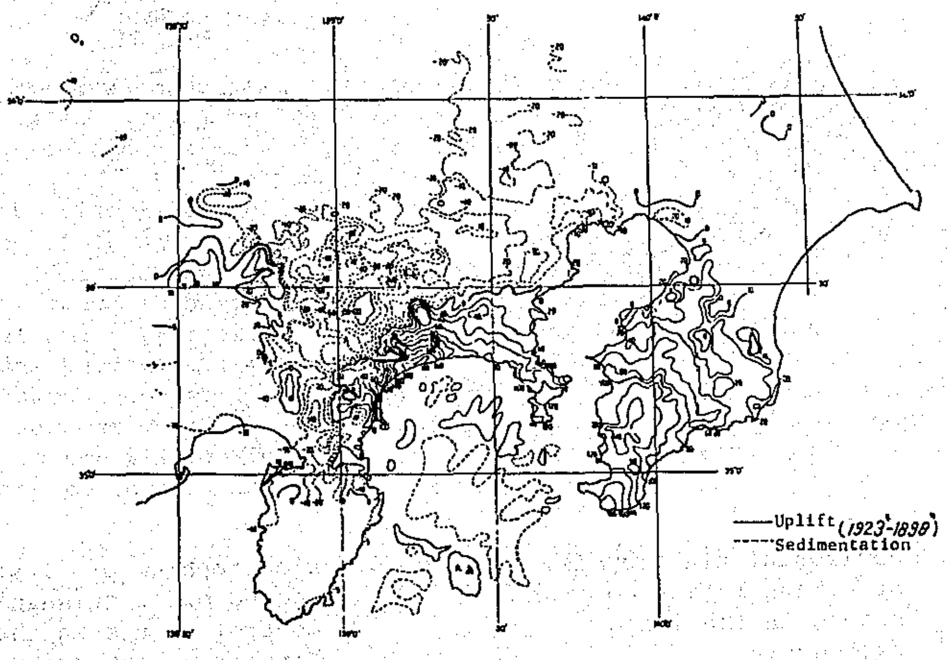


Fig. 5. Distribution of vertical diastrophism
in the Great Earthquake

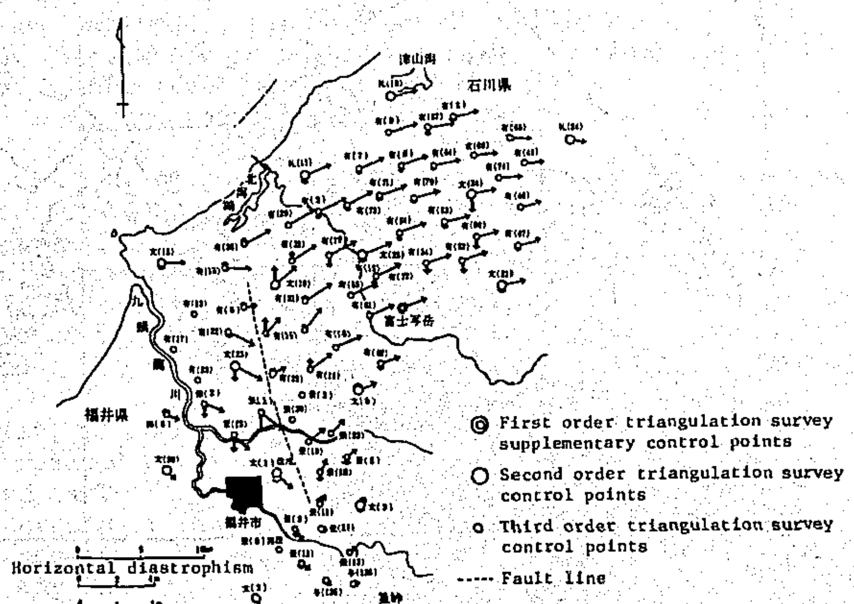


Fig. 6. Diastrophism of triangulation control
points in the Fukui Earthquake

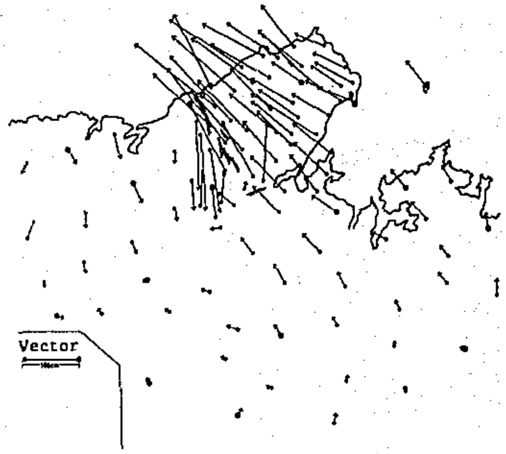


Fig. 7. Horizontal diastrophism of triangulation control points in the Tango Earthquake

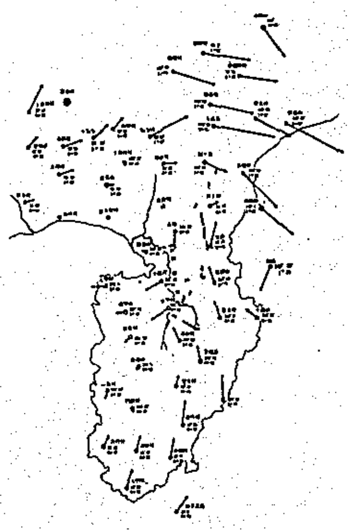


Fig. 8. Diastrophism of first and second order triangulation control points in the North Izu Earthquake

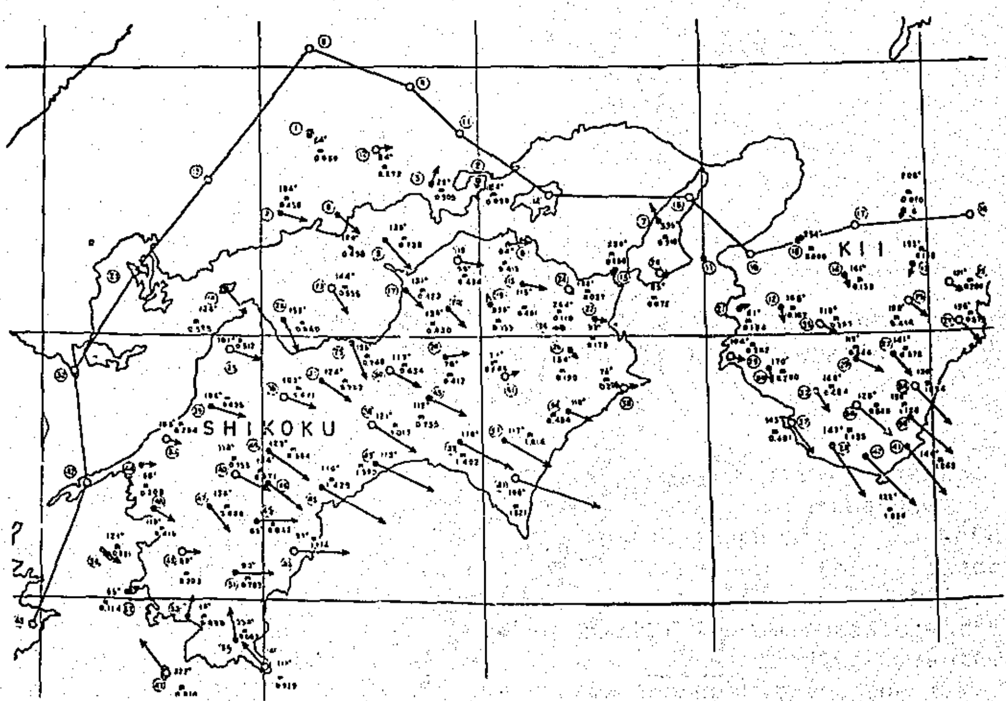


Fig. 9. Diastrophism of triangulation Control points in the Nankaido Earthquake

Section 3. Geodetic Control Points Survey Providing the Basis for Public Works

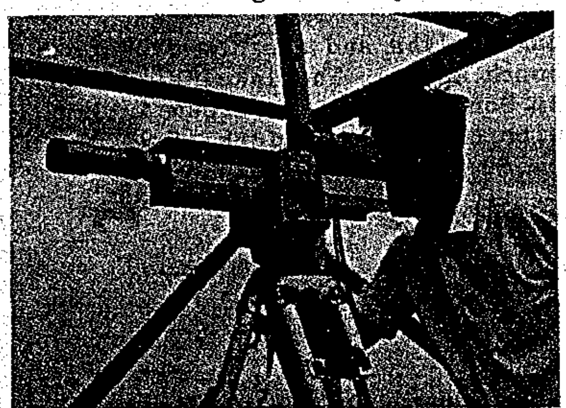
Precise geodetic survey contributes not only to natural sciences such as geophysics but also to precise civil engineering, in particular, large scale construction projects.

Precise geodetic instrumentation has been employed for follow-up survey of secular deformation of the dam in the survey on the safety of large scale dams. It is playing an increasingly important role in recent years in large scale public works such as the Seikan submarine tunnel and the bridge project linking Honshu and Shikoku. For instance, the dam survey, which is to study the deformation of the arch dam following the commencement of dam operation, is required to ascertain horizontal displacement with an accuracy of up to 1mm and 200∿300m of sight range. This is equivalent to the relative precision of $3\sim5 \times 10^{-6}$, requiring precise geodetic instrumentation techniques.

In the case of precise survey for the Seikan submarine tunnel commenced in the summer of 1964, horizontal error of 10cm was required for excavation from two points at a distance of 20km across the sea. This was a requirement for the relative precision of 5×10^{-6}, and oversea precise triangulation survey had to be conducted. In such a case, it is necessary to have geodetic control points with high precision in the vicinity of proposed construction sites.

As has been mentioned above, as recent construction projects require extremely high precision in survey, latest geodetic instrumentation techniques are employed. Further, as the survey area is often extensive, survey calculation is beyond the scope of plane local survey, and the theory of higher level of geodesy, taking account of the curvature of the earth and the deflection of the vertical, is required.

Thus, geodetic control points are expected to play an increasingly important role in future as they have to provide accurate coordinates for increasing extensive and precise public works such as the Super Express Railways, express highways, large scale bridges, submarine tunnels and large scale structures on the coast and also for the maintenance of completed facilities against accumulating diastrophism and ground deformation.

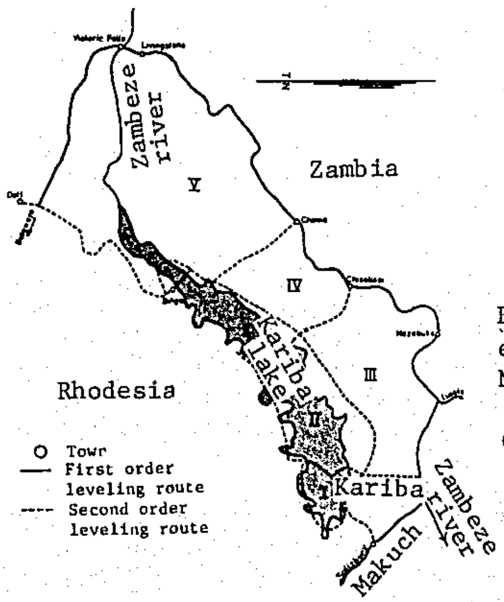


In particular, if these structures are expected to go through an active faults, it is important to ascertain if the fault is on the move and, if so, what effects it may have.

At some large scale dams in other countries such as Koina in India and Kaliba in South Africa, when water was stored in the reservoir, it activated the existing fault causing an earthquake. In one case the magnitude was over 6 causing extensive damages. The storing of water at a dam may thus trigger off an earthquake and also invite deformation of the crust which may be detected adequately by geodetic survey (Fig. 10).

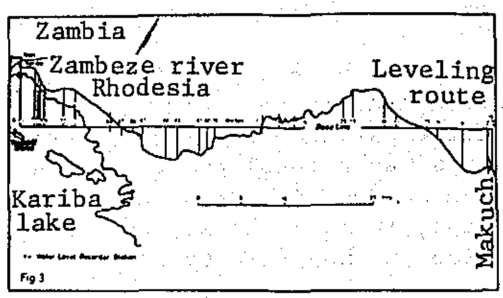
Since the increase in the size of the structure may thus invite large scale disaster if it is left without proper inspection of the safety. Accordingly, it is necessary to prevent such a disaster and, at the same time, conserve environment. All this is possible by detecting diastrophism and ground deformation. For that purpose, establishment of a geodetic network and periodical survey on a national basis are indispensable.

(a)



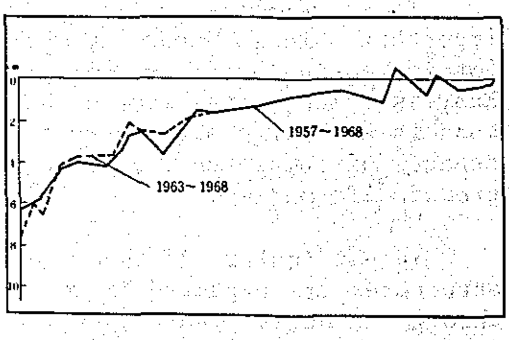
Lake Kaliba and the network of bench marks. The lake, situated in South Africa, is 250km in length with the maximum width of 40km.

(b)

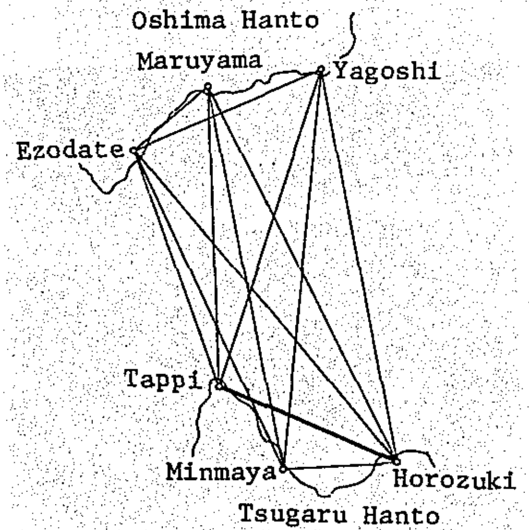


Bench marks between Kaliba at the east end of the Lake Kaliba and Makuchi.

(c)



Vertical movement along the leveling route on (b). The fact that fluctuations during the period from 1957 to 1968 are roughly the same as those 1963∿1968 shows that fluctuations occurred during the period of storing 1963∿1968.



Network of control points for the survey on the Seikan submarine tunnel.

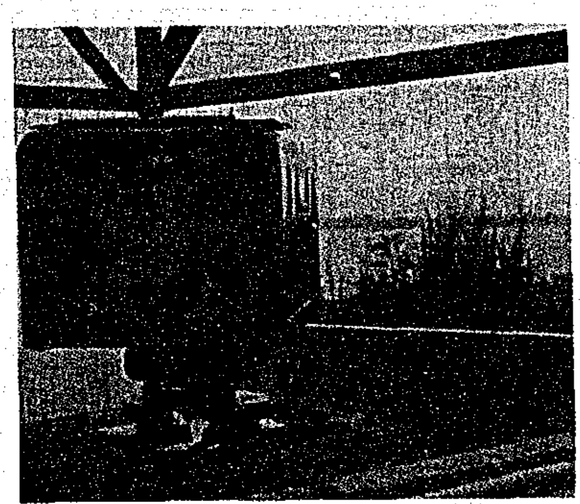
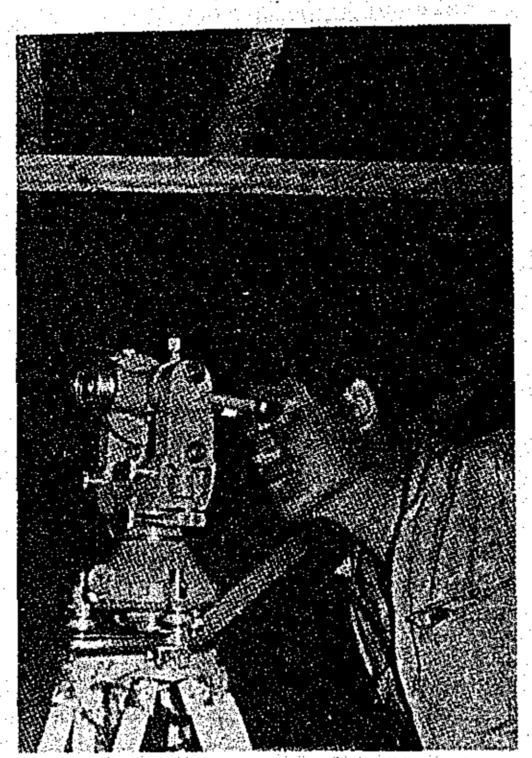
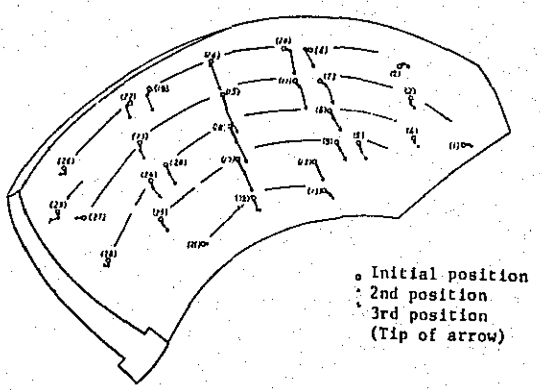


Fig. 10. Case of vertical diastrophism due to water storing at a dam

Horizontal shift of measuring marks on the dam

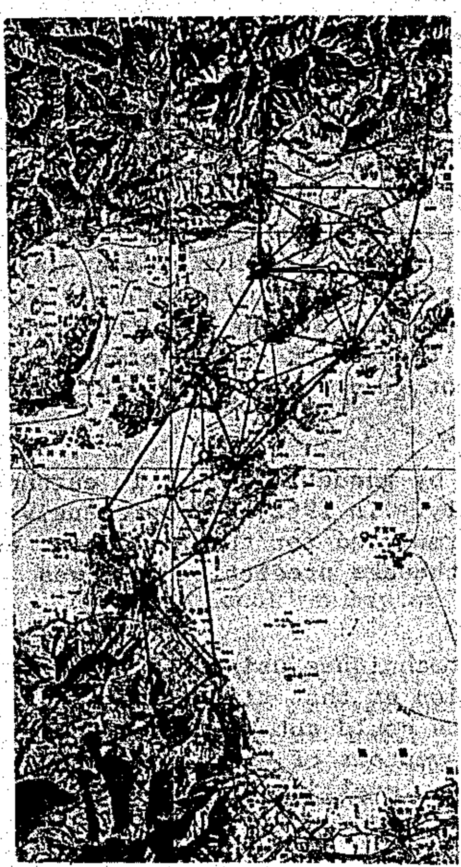
Measuring marks on the dam	2nd reading-1st reading (60%)			3rd-2nd (Full) (60%)		
	DX	DY	vector	DX	DY	Vector
	mm	mm	mm	mm	mm	mm
1	− 1	+ 5	101° 5	− 3	+ 1	162° 3
〃 2	+ 2	+ 5	68° 5	− 5	+ 5	135° 7
〃 3	− 5	+ 1	169° 5	− 7	+ 5	145° 9
〃 4	− 6	+ 1	171° 6	− 5	+ 4	141° 6
〃 5	− 8	+ 2	166° 8	− 10	+ 5	153° 11
〃 6	0	− 2	270° 2	− 22	+ 6	165° 23
〃 7	− 7	+ 5	145° 9	− 19	+ 6	163° 20
〃 8	− 10	+ 4	158° 11	− 15	+ 4	165° 16
〃 9	− 10	+ 4	158° 11	− 12	+ 5	157° 13
〃 10	− 3	+ 5	121° 6	− 24	+ 6	166° 25
〃 11	− 9	+ 5	151° 10	− 23	+ 6	165° 24
〃 12	− 13	+ 2	171° 13	− 9	+ 4	156° 10
〃 13	− 5	+ 5	135° 7	− 3	+ 2	146° 4
〃 14	− 1	− 1	225° 1	− 26	+ 7	165° 27
〃 15	− 13	+ 5	159° 14	− 24	+ 8	162° 25
〃 16	− 16	+ 5	163° 17	− 20	+ 7	161° 21
〃 17	− 15	+ 5	162° 16	− 17	+ 6	161° 18
〃 18	− 10	+ 2	169° 10	− 1	+ 3	109° 3
〃 19	− 5	− 2	202° 5	− 16	+ 5	163° 17
〃 20	− 10	+ 3	163° 10	− 11	+ 5	156° 12
〃 21				− 0	+ 3	90° 3
〃 22	− 3	− 3	225° 4	− 10	+ 2	169° 10
〃 23	− 6	0	180° 6	− 10	+ 4	158° 11
〃 24	− 7	+ 3	157° 8	− 8	+ 4	153° 9
〃 25	− 6	+ 3	153° 7	− 6	+ 3	153° 7
〃 26	− 4	− 1	194° 4	− 0	− 2	270° 2
〃 27	+ 1	− 3	289° 3	− 0	− 2	270° 2
〃 28	− 3	0	180° 3	+ 2	− 3	304° 4
〃 29	− 4	0	180° 4	− 1	− 6	261° 6

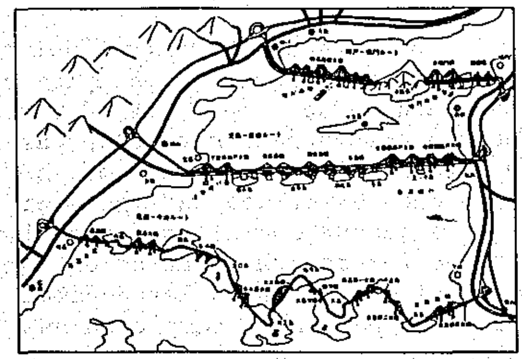




Control point damaged artificially

Fig. 11. Deformation of the arch dam





Above: Location map of the bridge connecting Honshu and Shikoku

Left : New geodetic network for Imabari-Onomichi (Proposed site for the Kurushima Ohashi)

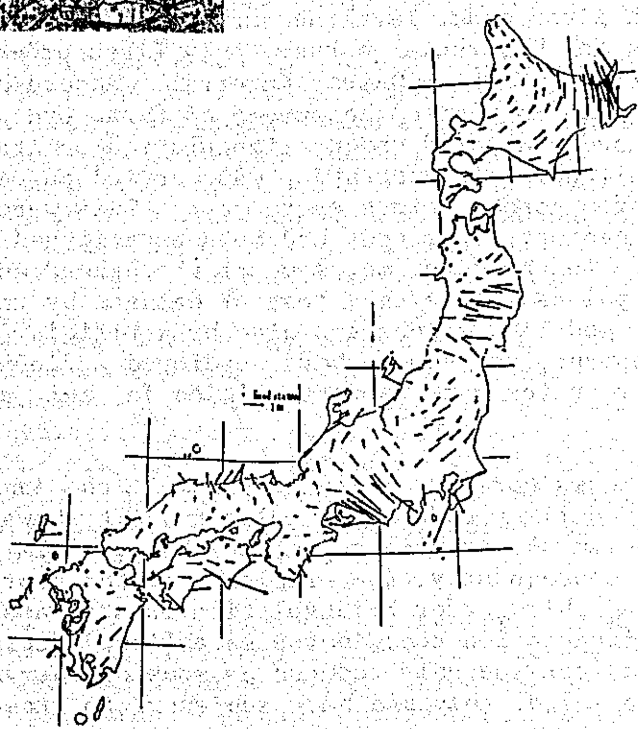


Fig. 12. Horizontal diastrophism in the last half century

CHAPTER 3. PRESENT CONDITION OF GEODETIC CONTROL POINTS

Though geodetic control points have become utilized for multiple purposes in response to changing circumstances, their control and maintenance have not been carried out except partial revision. The reason for the unfortunate situation was probably because it was not possible to obtain a long-term view of the operation of geodetic control points in as much as it was extremely difficult to foresee the present-day Japan amid the social confusion immediately after World War II. The surveys for the restoration of control points carried out after the Tonankai and Nankaido Earthquakes were, therefore, either of a temporary nature or partially completed and did not lead to a nation-wide geodetic survey.

However, the important role played by geodetic survey in earthquake prediction began to be recongnized widely. In 1962, a comprehensive plan based on a long-term vision was formulated for the first time as "*Geodetic operations with a view to earthquake prediction*" and was implemented partially from 1965.

However, since distance measuring techniques using light had not reached the stage of practical application at that time, precision in horizontal survey of triangulation points could not be regarded as adequate to capture minute premonitory symptoms of an earthquake. Consequently, expectations were placed on levelling which captures vertical diastrophism with the precision of several millionth. As a result, though first order leveling survey was to be repeated every five years, as for horizontal survey, only first order triangulation was to be carried out at a 10-year interval. Accordingly, there was no systematic plan to revise the survey of first order supplementary points and second and third order triangulation points totalling 38,000, and only partial restoration was carried out in respect of several tens of abnormal points every year. They were left in fact for nearly half a century after they had been established in the early Taisho period. In the meantime, some of the second and third order triangulation points were either lost or damaged by natural causes such as earthquakes and land slides and also by artificial obstacles such as land development, and some others developed serious errors in their coordinates, producing various difficulties in their practical application.

1) Fig. 12 shows horizontal diastrophism in the last 60 years revealed by repeated first order triangulation. It shows that the maximum diastrophism was as much as 4m and that of 1∿2m occurred in various places. Consequently, the precision of the coordinates of those second and third order triangulation points based on the first order triangulation coordinates established during the Meiji period was considerably reduced as some of them were left without correction after they had been subjected to extensive diastrophism.

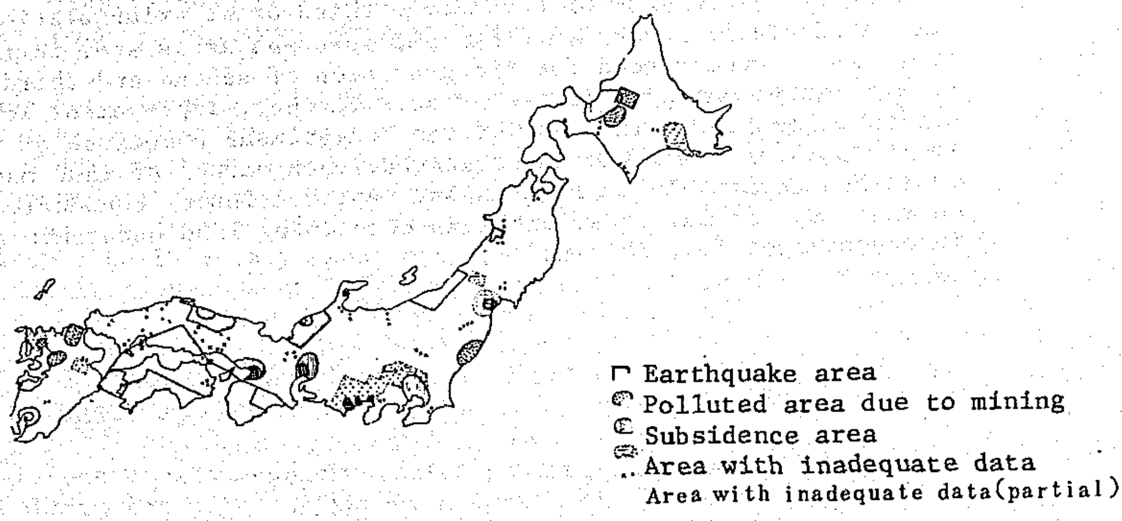


Fig. 13. Location map of unsatisfactory triangulation points

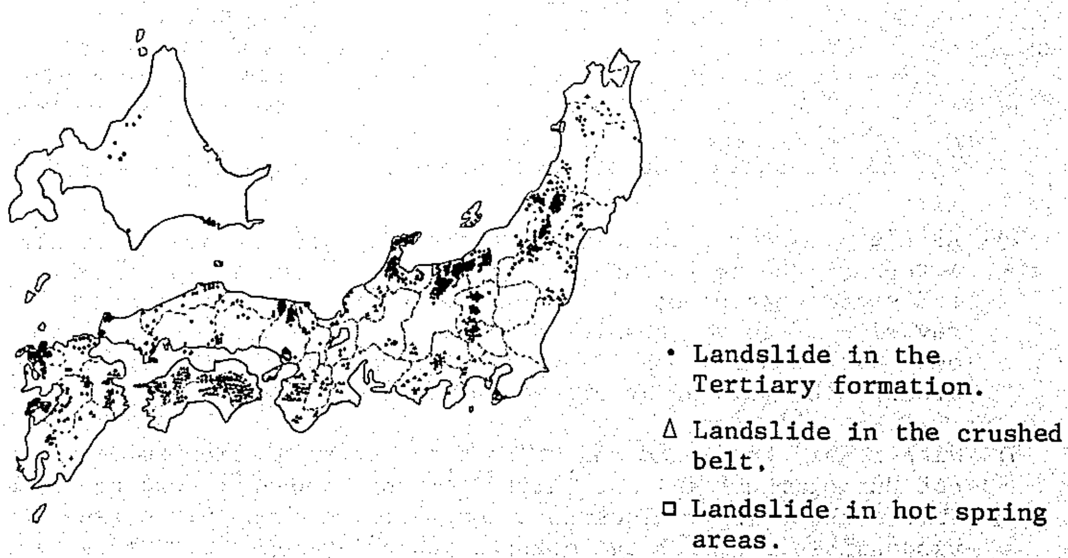
2) As Fig. 13 shows, those areas which have been subjected to diastrophism due to earthquakes, affected by mining, land slides and sedimentation cover the entire country, and local and partial revision of survey results by temporary survey for restoration only extended systematic error and irrationality to surrounding areas which may not be placed under control.

3) With regard to second and third order triangulation surveys carried out during the Meiji and Taisho periods, though it was understandable without large electronic computers, each survey officer was responsible only for the computation of averages for his area and concurrent computation for a wider region was not carried out. Consequently, those parts joining areas produced ununified results, and these unsatisfactory parts are scattered throughout the country, resulting in the survey results not unified for the country.

The ununified results of second and third order triangulation points due to the above reasons and the increase in the number of damaged points since the establishment are seriously affecting the establishment of fourth order triangulation points being carried out under the National Land Large Scale Map Project and other surveys for public works. In other words, they have lost their functions as geodetic control points which provide the standard for every type of survey and cannot be placed under control with mere partial revisions.

In particular, in connection with the concentration of population in large cities and their environs, there is an increasing need for cadastration of urban and suburban areas, which has been carried out mainly for rural areas under the Land Survey Act, and large scale and precise model surveys are in progress.

However, the loss of second and third order triangulation points was particularly marked in urban and suburban areas and, therefore, there is an urgent need for the provision of second and third order triangulation points in these areas. Further, improvement in the precision of control points has become indispensable because of the upsurge in land prices. If this is neglected, each parcel of land may not be described accurately in figures and reproduction of boundaries may not be ensured. It may cause problems concerning land ownership and thus inconveniences for citizens.



Source: River Bureau, Ministry of Construction

Fig. 14. Distribution of landslides

CHAPTER 4. RECENT ADVANCEMENT IN INSTRUMENTATION TECHNIQUES AND THE PRECISE GEODETIC NETWORK ESTABLISHMENT PLAN

As has been described in the previous chapter, the second and third order triangulation stations which have been left uncorrected since the middle of the Taisho period are closely related to the preparation of various maps, cadastration, urban re-development, large scale national land remoulding works and environmental conservation which affect actual society, and their nation-wide revision is urgently required for the development and conservation of the Japanese Islands.

Section 1. Epoch-Making Development of Distance Measuring Techniques Based on Light

The recent development of precise instrumentation techniques is revolutionizing the previous survey method with the history extending over a period of 200 years mainly through the rapid advancement of electronic engineering.

The conventional method of survey was mainly based on angle measurement with a theodolite, and triangulation based on angle measurement was the prevailing method of geodetic survey. Though triangulation has many merits, one of its demerits is that because of its nature there are many intervisible lines with neighboring points, thus making the topographical selection of triangulation points difficult.

It is clear that work efficiency will improve drastically with precise traversing thereby the position of a new point may be determined by obtaining only one intervisible line between two points and by measuring the included angle and the side length on the traverse route. Since triangulation requires costly high observing towers especially in flat urban areas or wooded areas, it will require a large amount of expenses and labor.

Therefore, the invention of an efficient and precise distance measuring instrument has been a task in survey for a long time. In recent years, however, with the development of a electro-optical distance measuring instrument, it has become possible to measure the distance between two points with high precision and speed based on light velocity, a physical constant, by applying modulated light between the two points. Further, with the use of the laser beam, it has become possible to measure a long distance of 60∿70km with an error of less than ±2∿3cm. The appearance of such electro-optical distance measuring instrument has truly revolutionized geodetic survey and its significance may rank with the invention of the theodolite in the 17th century.

Since angle measurement with a theodolite is accompanied with an error, which is difficult to be corrected, because of abnormal refraction of light in the vicinity of the instrument, the improvement of the instrument is limited and further improvement is difficult to achieve.

With regard to distance measurement, however, even the present degree of precision excels that of angle measurement and the method of atmospheric correction is also easy. Further improvement on precision is still possible. For instance, a successful attempt has been made to ensure the instrumentation precision of 10 millionth by using different wave lengths of light to obtain automatically the accurate atmospheric correction, and the method will before long be put into practice. Reduction of the instrument in both size and weight is also being attempted in parallel with the development of electronic techniques, and marked improvement in work efficiency may be expected.

The Geographical Survey Institute has used the laser distance measuring instrument in the last few years for side length survey, arc length survey and precise distortion survey and proved its superiority. The Institute has also devised the method of operation to obtain high precision and efficiency. Fig. 15 gives, as an example, survey data on distortion in the southern part of Kanto area. Unlike previous triangulation survey, precise measurements of each side (average 20km) of the triangulation network were carried out repeatedly in this survey to follow the horizontal distortion of the crust to contribute to earthquake prediction.

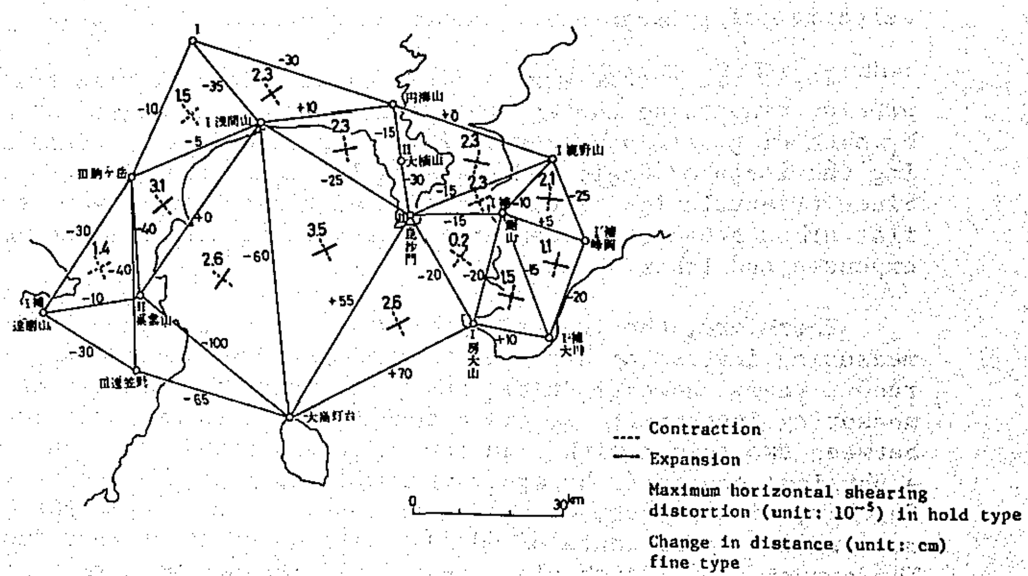


Fig. 15. Distance change shown by the precise distortion survey of the southern part of Kanto area (1924∿26-1970∿72) and maximum horizontal shearing distortion

The figure shows clearly that 1) Oshima moved approximately 1m to the west after the Great Earthquake in the southern part of Kanto area;

2) Boso and Miura Peninsulas are being compressed in the same direction as that of the expansion in the Great Earthquake but only reversed (SSE → NNW); and 3) present maximum shearing distortion applying to the crust is in the order of 3×10^{-5} and there is no immediate danger of a large scale earthquake of the Great Earthquake class (magnitude 8), though there seems to be an accumulation of energies sufficient to cause an earthquake of the magnitude 7.5 class similar to the Niigata Earthquake.

Precise distortion survey of the southern part of Kanto area is to be repeated completely for every 2∿3 years; but annual observation is carried out on the important side length. The results given by Table 1 show a surprising fact that the data obtained for different years are nearly identical. As a result, those concerned with survey have realized that a new age of survey has arrived with the geodimeter and those engaged geological research have felt strongly that survey of horizontal diastrophism by the electro-optical distance measuring method is the most reliable method for practical application of earthquake prediction and that the way has been opened to solve many unsolved problems concerning the techtonics of the Japanese Islands.

With the epoch-making advancement of survey techniques in the background, the Geographical Survey Institute has formulated the Precise Geodetic Network Project as an important part of the Third Long-term Program beginning in 1974, taking also account of the urgency of a large scale revision of the second and third order triangulation networks.

Table 1.

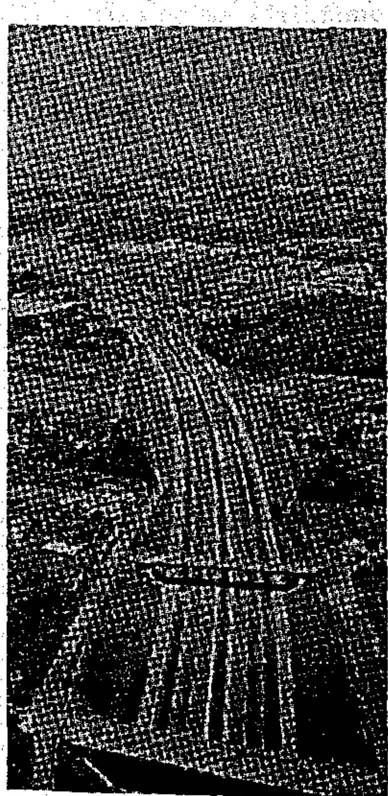
Survey point	1970. 8 I	1970. 12 II	1971. 12 III	Difference		Precision $\times 10^{-6}$
				II-I	III-II	
Bishamon - Oshima	51512.46m	.48m	-	+0.02m	-	0.4
" - Bodaisan	24045.83	.82	85m	-0.01	+0.03m	0.4.1.2
" - Nokogiriyama	17848.20	.18	-	-0.02	-	1.1
" - Ogusuyama	11038.59	.57	-	-0.02	-	1.8
" - Oshimatodai	-	46524.71	.75	-	+0.04	0.9
Oshimatodai - Sukumoyama	-	38289.98	.00	-	+0.02	0.5
" - Bodaisan	-	41434.73	.75	-	+0.02	0.5

Section 2. Scope of the Precise Geodetic Network Project

This project for the establishment of the precise geodetic network is, in short, to establish "Geodetic survey of the Showa period", different from that of the Meiji period. The difference between the two may be outlined as below.

(1) Since geodetic survey of the Meiji and Taisho periods was intended for the preparation of medium scale topographic maps, it was conducted in such a way that absolute precision of the triangulation survey points would invariably be approximately ±10cm through the orders of triangulation survey from the first order down to the third order. Consequently, relative precision fell from the first to second, and from the second to third, and the precision of several hundred thousandth with the first order triangulation survey fell to several ten thousandth with the third order triangulation survey.

In the case of the new "Precise geodetic network", however, because of the drastic improvement in distance measuring techniques, it is possible to ensure high relative precision regardless of the difference in distance without reducing efficiency. Especially with the second order triangulation survey network having the average distance of 8km for the side and above, it is possible to obtain practically every side with the precision of over 1 millionth by means of using the laser distance measuring instrument.



Above: Kuroyon Dam

Right: Express highway

(2) As for the third order triangulation points, the number totals 32,000, drastically more than the 6,000 with the second order triangulation points and above. Accordingly, it is virtually impossible to secure intervisible lines for all these points because of the collosal costs due to obstacles such as housing and trees both in plane areas and in mountain areas. For the third order triangulation points, therefore, traversing is to be employed for efficiency in operation, selecting the minimum number of intervisible routes to link triangulation points.

(3) As has been mentioned, survey computation during the Meiji period had to be carried out manually. Consequently, each survey officer had to carry out adjustment computation for each new point in his area (averaging 20 points). Thus, ideal adjustment, taking account of adequate error in coordinates of triangulation points survey, was not made for the joining sections of different survey areas, thus resulting in discrepancy ranging from several 10cm to 1∿2m. This was of course partly due to land movement over a period of several tens of years; but it was mainly due to the fact that the simplified method, based on small areas which had been adopted as adjustment computation for geodetic networks requiring an enormous amount of calculation in least square method, had to be carried out manually.

In the case of the new "Precise geodetic networks", however, since the large scale electronic computer has increased the computation capacity to the level which had been unconceivable previously, it has become possible to carry out adjustment computation for geodetic networks in an area of the size of Japan concurrently if necessary. Accordingly, the problem of ununified data or discrepancies in data due to the lack of rigidity in computation has been solved for the new geodetic networks.

(4) The theory of geodesy which provided the basis for geodetic survey during the Meiji period was "Two-dimensional geodesy" of the 19th century Europe. Geodetic coordinates are, needless to say, three-dimensional coordinates. However, the theory for obtaining three-dimensional coordinates accurately did not yet exist in the 19th century, nor was there demand for it.

From the present point of view, many of the survey data obtained during the Meiji period are in urgent need of revision. For instance, Japan's present system of geodetic coordinate system (Bessel's ellipsoid) shows a deviation of 12" in N-S and 10" in E-W from the geoid (terrestrial gravitational potential surface corresponding to the mean sea level) around the Japanese Islands. This is due to the fact that the astronomical latitude and longitude of Tokyo adopted as the geodetic latitude and longitude of the Tokyo datum station has a deviation of the plumb line resulting from the topography and the tectonics of South Kanto area.

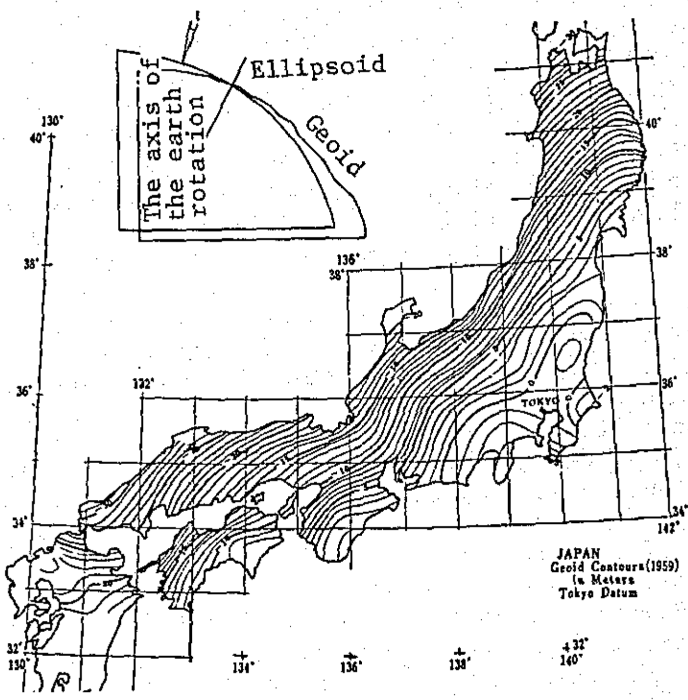


Fig. 16. Height of geoid to reference ellipsoid (unit: m).
The geoid deviates downward to several tens of meters below the Bessel's ellipsoid in proportion to the distance from the Tokyo datum station.

Consequently, as Fig. 16 shows, the reference ellipsoid corresponding to the geoid at the Tokyo datum station is off by several tens of meters above the geoid plane in Hokkaido and Kyushu, producing an error which cannot be ignored when reducing observed values to the ellipsoid.

After World War II, however, geodetic theories have made great progress in two fields. One was the completion of the theory for obtaining almost accurate three-dimensional geodetic coordinates by combining the traditional ellipsoidal geodesy and gravitational geodesy.

The other was that with the development of artificial satellites the global form of the geoid plane was ascertained and, at the same time, it became possible to unify regional and national systems of geodetic coordinates as a global system linking continents and oceans.

Accordingly, in establishing the new "Precise geodetic networks", the results of postwar modern geodesy are to be used to determine the position of Japan according to the global system of coordinates. Further, the reference ellipsoid corresponding to the mean geoid plane to establish correct three-dimensional geodetic coordinates.

(5) In order to promote practical application of earthquake prediction, first and second order triangulation networks of the precise geodetic networks are to be consolidated in observation.

In accordance with the approval of the Cabinet, the Earthquake Prediction Council was formed in 1969, comprising five national universities and six government agencies (Hokkaido University, Tohoku University, Tokyo University, Nagoya University, Kyoto University, National Research Center for Disaster Prevention, International Latitude Observatory, Japan Meteorological Agency, Hydrographic Department of the Maritime Safety Agency, Geological Survey Institute and the Geographical Survey Institute).

After conducting statistical study of geographical and time series of the past earthquakes with regard to future possibilities, the Council designated certain areas as Specified Observation Areas as shown by Fig. 17. The designated areas are those where there is a stronger possibility of earthquake judging from the above seismological statistics compared with other areas, thus requiring intensified observation.

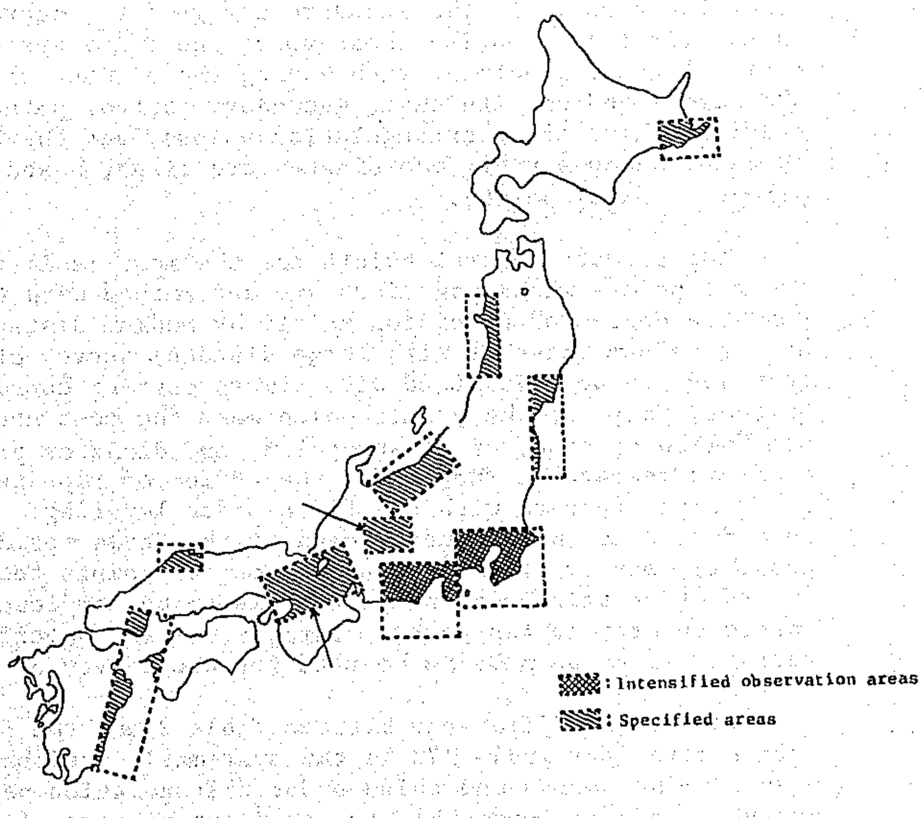


Fig. 17. Intensified and specific observation areas

The Intensified Observation Area in South Kanto shown by Fig. 17 was designated as such not only because of findings in seismological statistics but also because of abnormal phenomena observed in the area. It is, therefore, necessary to intensify various observations for follow-up study to ascertain whether these phenomena may lead to an earthquake.

From the standpoint of earthquake prediction, some areas of the Japanese Islands thus require special attention in observation compared with other areas, and geodetic survey forms an important part of the observation. Therefore, among the precise geodetic networks, those Specified Observation Areas require detailed survey of horizontal diastrophism. Further, as for the intensified Observation Area and the Concentrated Observation Area which is judged to be in a highly dangerous situation, the period of repeated observation has to be reduced further.

In the light of the above points, the over-all plan for the establishment of precise geodetic networks inaugurated in 1974 may be outlined as Table 2. The primary control points on Table 2 consist of the previous first order triangulation points and supplementary points, and second order triangulation points numbering 6,000. As the standard for geodetic coordinates, these points are, unlike those under the Meiji system, to be all national control points, each having the highest precision of the same standing. Further, secondary control points are the previous third order triangulation points, and they are called as such, because their coordinates are fixed, based on those of primary control points.

The primary control points are the most basic national control points whose positions are determined with the highest possible degree of precision mainly by medium distance survey averaging 8km together with large distance survey of 20∿50km. They have to be maintained with always correct coordinates. Further, they are the points which make the most contribution to the follow-up study of diastrophism, and disaster prevention and academic research. Therefore, the period of repeated survey is to be five years as with the first order leveling. Further, in order to establish survey data of the Showa period as soon as possible, survey of the secondary control points totalling 32,700 (third order triangulation points) is to be completed for the entire country in ten years. Further, as for specified areas, revision is to be made to be used for the survey of diastrophism.

The marked difference between Table 2 and the Second Long-term Program for 1964∿1973 is the systematic national revision plan for the second and third order triangulation networks. However, this is impossible to carry out because of the limited number of personnel. As a rule, therefore, it is to be carried out by the private sector.

Japan's survey industry has improved its technical level in recent years and perfected fourth order triangulation survey techniques (control point survey) with the electro-optical distance measuring instrument. Some firms have also carried out even more precise survey projects successfully, and it seems that they have sufficient capacity to carry out the precise geodetic network project. Under the guidance of the Geographical Survey Institute, therefore, they are expected to produce correct data which may be handed down to the next generation.

Table 2.

Type of survey	Previous name	Scope of survey	Remarks
Primary control point survey	First order triangulation survey Second order triangulation survey	The network of 8∿10km in average distance between points (corresponding to second triangulation survey) is to be the standard control point network. Further, Long distance survey of 20∿50km is to be conducted to consolidate the national standard network.	o Mainly under contract with the private sector; partially conducted by the Institute. o To be repeated at a 5-year cycle. o Target points: 6,000. o Long distance survey: 330 points at a 10-year cycle.
Secondary control point survey	Third order triangulation survey	The average distance between points is to be 4∿6km (corresponding to third order triangulation survey), and precise traversing is to be conducted with primary control points as the given points.	o As a rule under contract with the private sector. o A period of ten years is allocated tentatively for the survey of the entire country. Thereafter revision and restoration are to be carried out to maintain the data. o Target points: 32,700.

Section 3. Practical Results of the Precise Geodetic Network

The urgent need of the contemporary society for a precise national control points network has been partially described in the previous sections. The practical results of the precise geodetic network may be outlined as below.

(1) A new and accurate standard may be provided for the preparation of large scale domestic maps (scale 1/1,000∿1/5,000);

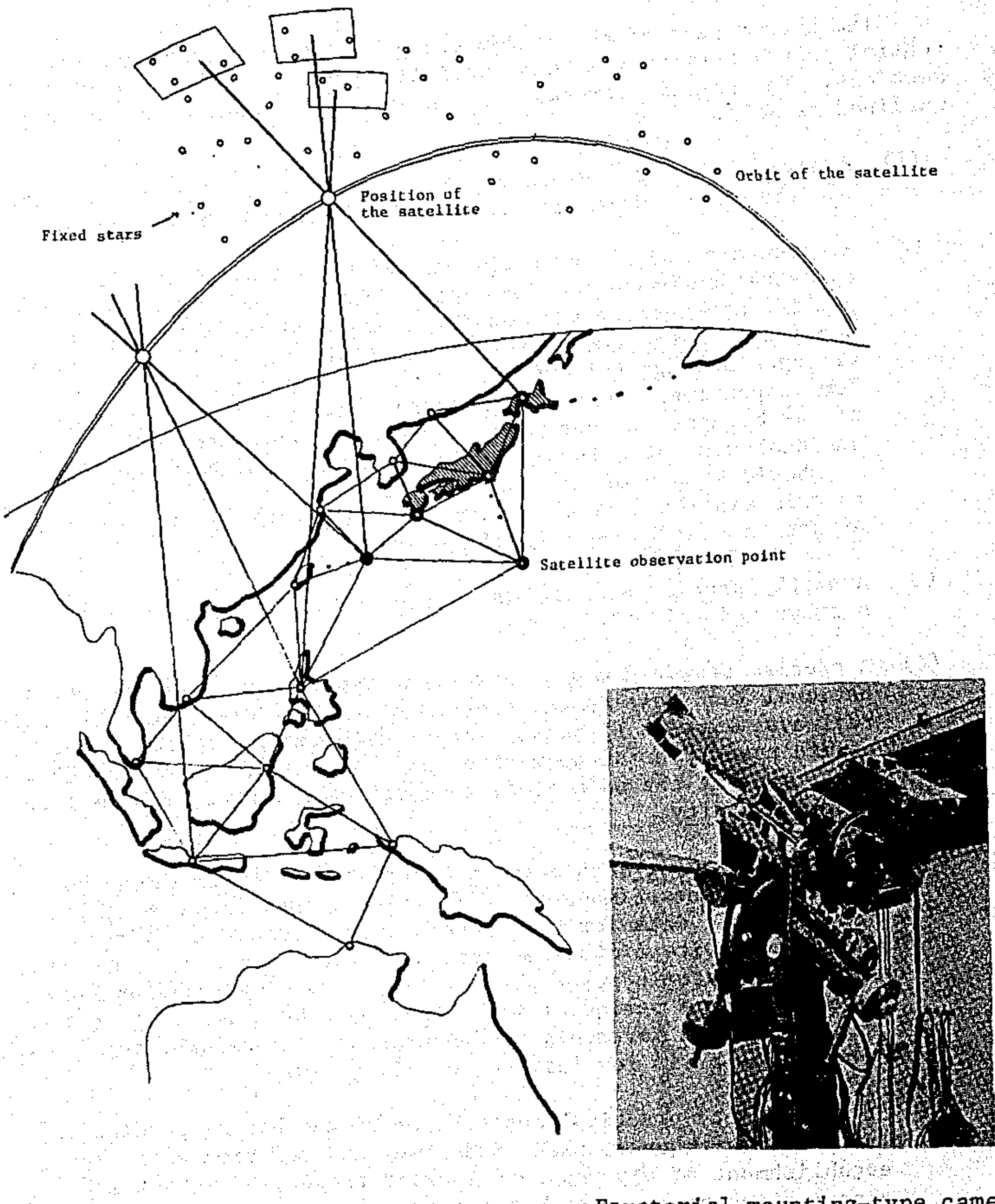
(2) An accurate and reproducible standard may be provided for cadastration under the Land Survey Act. Especially, since the precision regarding the boundary corner point in the urban section of large and medium cities (A-1), general urban areas, villages and agricultural areas (A-2) is prescribed as 2cm and 7cm respectively (mean square error) under the Act, the position of the boundary corner point may not be reproduced unless the national control points are maintained with higher precision. It should be one of the important responsibilities of the state to maintain the control points dada with high precision, which affect the land ownership of the citizens greatly;

(3) A highly precise standard may be provided for domestic public surveys and all other surveys;

(4) A precise standard may be given for especially large and precise national land development and remoulding projects (Super Express railways, express highways, large scale dams, oil pipe lines, atomic power plants, submarine tunnels, underwater structures, etc.) and, at the same time, geodetic coordinates with durable reproducibility;

(5) At the same time, land movement and deformation in the area surrounding these important facilities are to be surveyed to provide basic data for the conservation of those facilities which may invite a disaster such as large scale dams, submarine tunnels and atomic power plants. The survey of fractional land movement over a period of years may contribute directly to the study of ground movements (sedimentation, landslides, etc.) and provide the basis for further detailed survey;

(6) It may promote practical application of earthquake prediction (scale of earthquake, place and time) and contribute to the establishment of a permanent disaster prevention system and construction projects such as cities, ports, roads and plants. Since Japan will never be free from earthquake disaster, if scientific elucidation and prediction of earthquakes are possible, countermeasures may be formulated to keep the damage at a minimum and eliminate unnecessary social apprehension resulting from inaccurate information.



Equatorial mounting-type camera

Fig. 18. If observation is made of the artificial satellite simultaneously at more than two known points and unknown points, positions of the unknown points may be accurately determined.

Though academic foundations of practical application of earthquake prediction by geodetic survey will be dealt with in the next section, repeated survey of the precise geodetic network will certainly detect signs of a coming large scale earthquake in conjunction with the survey of vertical diastrophism by leveling. For that purpose, it is essential to start observation and store data for the entire country before it is too late. Especially the South Kanto area which includes the Metropolitan Tokyo, literally the heart of Japan in politics, economy and culture, is estimated to have accumulated sufficient energy to cause an earthquake of the M-7 class after an elapse of half a century after the Great Earthquake of 1923. However, the present precise distortion survey (average distance 20km) is not adequate to predict earthquakes of this scale, and there is an urgent need for the establishment of a more intensive and precise geodetic network.

(7) Linkage with geodetic survey by satellites.

What has been given above is an account of the utilization of domestic geodetic coordinates. The precise geodetic network, however, may be linked with the rapidly developing geodetic survey method with artificial satellites to obtain the accurate position of the Japanese Islands, which is said to have a deviation of about 500m at present, and provide accurate geodetic coordinates for satellite observation stations in the country. This will contribute to the determination of geodetic coordinates of not only the remote islands around the Japanese Islands but also the several thousands of those islands belonging to the developing countries of Southeast Asia where precise survey and mapping have not yet been carried out.

Since these islands are mostly covered with thick woods, it is impossible to establish control points by ordinary land survey. However, as shown by Fig. 18, by linking the countries of Southeast Asia with Japan's precise geodetic network as the given point group by satellite triangulation, it is expected to make a great contribution to the survey and mapping operations which are indispenable for regional development in these countries.

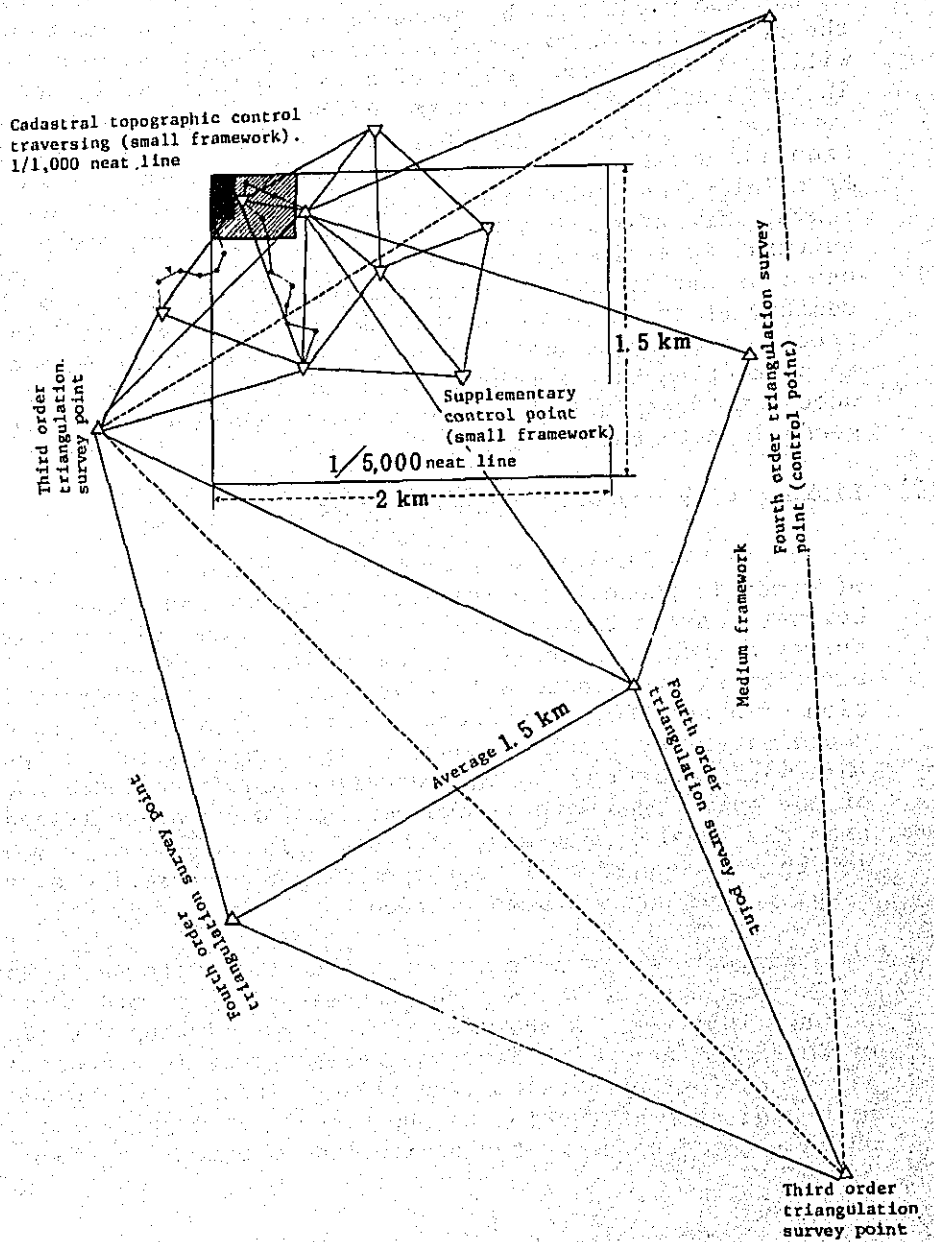
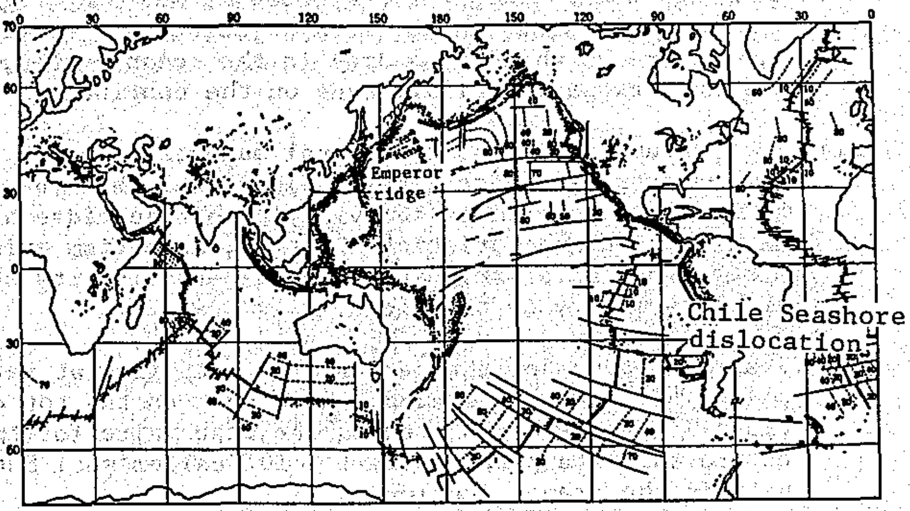


Fig. 19. Outline map of cadastration operation based on the national triangulation points

CHAPTER 5. ACADEMIC RESULTS OF PRECISE GEODETIC NETWORK

Section 1. Modern Earth Science and Precise Geodetic Network

Before discussing the contribution of Japan's precise geodetic network to modern natural sciences, particularly solid earth science, let us look at the present condition and problems faced by earth science.



Trench
Ridge
Transform fault
Epicenter
Direction of fault movement
Banded pattern of magnetism. Figure indicates the age (unit: 1 million years).

Fig. 20.

(1) Emergence of plate tectonics

As the saying "Immovable as the great earth" suggests, it was thought in the old days that there was nothing so stable and immovable as the earth. However, science has revealed that this very earth is moving, though slowly. As can be seen from the fact that fossil shells were found on the top of a mountain range in central Japan, where the elevation is over 2,000m, during the long geological period, the deep ocean bottom rose in many parts of the earth to form high mountains.

The concept of the earth has thus undergone a drastic change in recent years to that the earth is a living planet engaged in large scale movements. This change came with the drastic improvement in instrumentation techniques in marine observation after

World War II, discovering many important facts one after another which could not be explained according to the previous concept of the earth based on the observation of various phenomena on land.

i) With the extension of the topographic observation to the bottom of the ocean occupying the three-fourth of the surface of the earth, it was discovered that there was a long and extensive mountain range called mid-oceanic ridge running in the middle of the ocean as shown by Fig. 20. On the other hand, in the area adjacent to a continent runs a trench which is a drop in the ocean bottom; and a volcanic range invariably runs on the continental side;

ii) As instrumentation of terrestral magnetism on the ocean became extremely easy, it was discovered that the distribution of magnetism in the vicinity of the ridge had a surprising characteristic. That is, magnetism exists in a belt form of a stripe pattern, alternating its strength and in symmetry to the ridge.

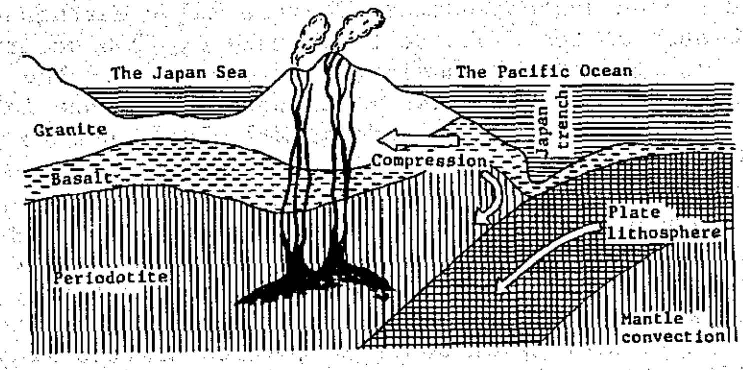
iii) Instrumentation of the terrestrial heat flow at various points of the ocean bottom showed that on the top of the ridge it was twice as much as the mean value for the entire earth (approximately 1×10^{-6}cal/cm^{2}sec) but was half in the trench section.

iv) It was also found out that even in the ocean area almost free from earthquakes numerous minor tremors invariably occur in the ridge section and that in the trench section such as the Japan trench numerous earthquakes concentrated in that part of the plane where it falls into the earth at an angle of several tens toward the side of the continent.

The above facts seem to be explainable with the ocean-floor spreading hypothesis — the ridge is where substance surfaces from the earth's interior and the trench is where substance subsides. Those substance surfacing at the ridge spreads to both sides and, at the same time, uplifts the ocean-floor near the ridge, causing minor tremors. The surfacing of hot substances from the interior increases the terretrial heat flow. The temperature of the substance then falls and magnetization occurs in the direction of the current terrestrial magnetism.

Though the direction of magnetism is north at present, the direction has never been stable. In fact, historical study of magnetism shows that the direction was reversed with a cycle of several hundred thousands of years. If in fact the ocean lithosphere surfacing, cooling and magnetizing at the ridge moves at a few centimeters per year in a horizontal direction, the abnormal distribution of terrestrial magnetism forming a stripe pattern may be explained satisfactorily. Further, the results of observation concerning the distribution of earthquakes and the earthquake mechanism show that it spreads to both sides at the ridge and obliquely at the trench section, supporting the ocean-floor spreading hypothesis.

The ocean lithosphere produced on the ocean-floor by the mantle convection thus forms a plate of about 100km in thickness. The new theory, Plate tectonics, has thus emerged to explain the dynamic behavior of the earth by assuming that the earth is covered with a few plates including both the continent and the ocean, which move and collide with each other with the result that one goes under the other.



a) The Japanese Islands are subjected to approximately horizontal compression from the Pacific.

b) The compression is caused by the plate of 70~100km in thickness known as lithosphere moving from the eastern part of the Pacific at a speed of few centimeters per year and subsiding at the Japan trench.

(2) Verification of plate tectonics by geodetic survey

A considerable number of scholars have come to think that the theory of plate tectonics can explain various observed facts satisfactorily.

In the case of the Japanese Islands and the surrounding area, the Japan trench runs from the southeastern part of Hokkaido along the eastern side of Northeast Japan, where the Pacific plate is assumed to submerge. Magma then rises from the top of the plate to form volcanoes. Since the Japanese Islands are subjected to pressure from the Pacific in this area, many earthquakes have occurred within the crust including those accompanied by tsunami such as the earthquake of 1896 and the great tsunami of 1933.

The Japan trench is assumed to be where the Pacific plate began its sudden subsidence in a relatively new geological period, which is still in progress. With this subsidence the Pacific coast of Northeast Japan is assumed to tilt because of the pull, and some survey results justify such an interpretation. At the same time, the Pacific coast is subjected to the compression from east to west and is expected to contract in that direction.

The Pacific side of Southwest Japan from the southern part of Kanto to the southern part of Shikoku, where many earthquakes have occurred from olden times, is assumed to be where the Philippine Sea plate and the Japanese Islands are in collision. It has been suggested recently that major earthquakes in this area such as the Nankaido Earthquake of 1946 occurred in repulsion against the subsidence of the plate. The frequency of earthquakes of this class is assumed to be once in 100~200 years and that of smaller ones of the M-7 class once in several tens of years.

Though the frequency of earthquakes occurring in the same area is lower on the Japan Sea side, there have been major earthquakes on that side. According to one theory, the Japanese Islands were once part of the continental mass situated at the eastern edge of Asian Continent, which drifted away to the Pacific side, producing the Japan Sea. If the expansion of the Japan Sea is still in progress, the Japanese Islands are also compressed from the Japan Sea, and those earthquakes occurring on that side may be the results of repulsion against the pressure from the Japan Sea.

These views based on modern earth science (plate tectonics) are still hypotheses at the present stage. However, they are expected to provide an important key to the solution of various problems, particularly those concerning the Japanese Islands. Accordingly, it is important for Japan to carry out verification and development of the plate tectonic approach by utilizing Japan's geological environment and elucidating the geology of the living Japanese Islands. Though various studies in earth science are of course important for that purpose, precise survey may be regarded as one of the most important methods as it can obtain without any hypothesis various symptoms of tectonic movements of the crust.

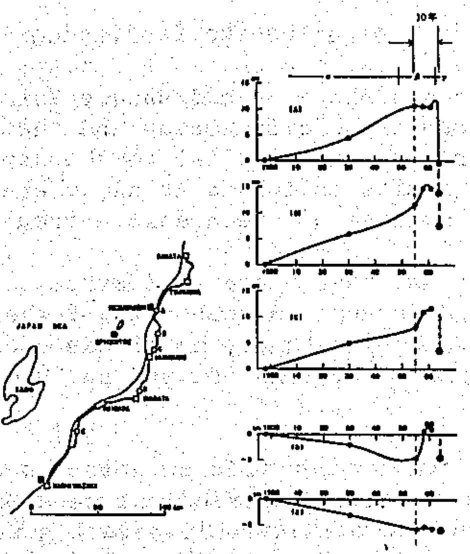


Fig. 21. Vertical movement at various places facing Awashima before and after the Niigata Earthquake (M7.5)

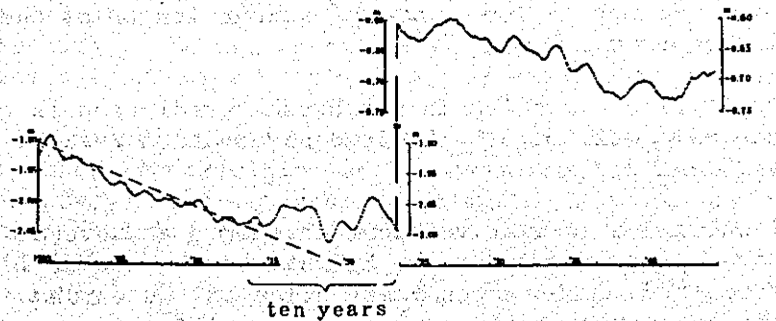


Fig. 22. Vertical crust movement shown by the moving average of monthly mean sea level at Aburatsubo (M7.9)

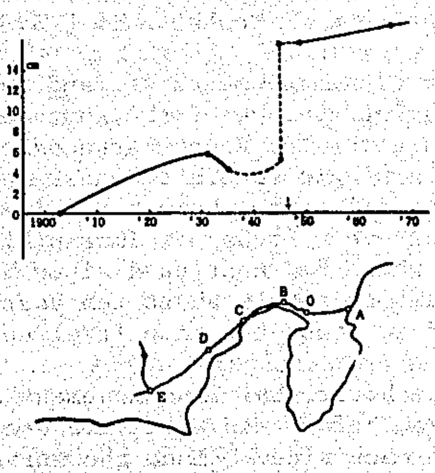


Fig. 23. Vertical movement at Kakegawa against Numazu before and after the Tonankai Earthquake (M8.0)

Section 2. Contribution to Earthquake Prediction

As has been described above, the Japanese Islands are subjected to compression from both the Pacific Ocean and the Japan Sea and, consequently, folded ridges run parallel to the Japanese Islands. There are many active faults in inland areas, where earthquakes causing damages have occurred because of the pressure from both sides.

It has been found out that there is a close relationship between the magnitude of the earthquake and the size of the area of the abnormal movement of the crust. For instance, magnitude 8 corresponds to a circle with the radius of 65km, magnitude 7 to 20km and magnitude 6 to 6km.

However, Japan's first order triangulation network (average length of one side 45km) is covered by a periodical repetition of survey, survey by such a coarse network may only detect abnormal movement of the crust producing extremely large earthquakes such as those of the M-8 class, and those of the M-7 class, though they are also large, may not be detected. Accordingly, in order to detect signs of large earthquakes of the M-7 class, repetition of survey has to be extended to the second order triangulation network (average length of the side 8km) throughout the country.

The most difficult problem in earthquake prediction is to predict when the earthquake will occur. Limited observation results are shown by Figs. 21, 22 and 23.

Fig. 21 shows the secular vertical movements at several points on leveling routes before and after the Niigata Earthquake of 1964. It shows that from 1898 to about ten years prior to the earthquake, the coast near the epicenter continued uniform rectilineal movements. Thereafter, they seemed to have assumed a different aspect, showing acceleration until they came to a halt for two or three years immediately before the earthquake. Similar cases have been reported wtih other earthquakes such as the one at Tashkent in U.S.S.R.

Fig. 22 shows vertical movements of land based on the mean sea level by using the moving average of the monthly mean sea level at Aburatsubo before and after the Great Earthquake of 1923 (M-7.9). Since it is based on a single record taken at Aburatsubo, effects of marine and weather conditions on the sea level may not have been sufficiently removed. However, it is not impossible to understand that the subsidying trend which had continued since before the Meiji period at an uniform rate came to a halt about ten years prior to the Great Earthquake and stayed at about the same level until the earthquake.

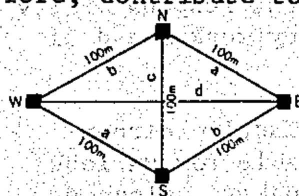
Fig. 24 shows the horizontal land movement at a diamond shaped base lines at Mitaka, Tokyo, before and after the Great Earthquake, having the length of 100m on each side. Since the survey began in 1916, later than at Aburatsubo, the mode of secular movement over a period of long time prior to the earthquake. However, it is surprising to see that, though Mitaka was far away from the epicenter, the mode of horizontal movement before and after the earthquake was very similar to the record of tidal observation near the epicenter. It shows that survey of horizontal diastrophism may rank with that of vertical diastrophism if the survey is carried out with high precision.

Fig. 23 shows the vertical movement at the Kakegawa bench mark before and after the Tonankai Earthquake of 1944 (M-8.0) on theassumption that Numazu was immovable as it was not affected by the earthquake. As is shown by the figure, Kakegawa area had shown an uniform secular rise until about 1933; but from ten and several years prior to the earthquake the mode of the movement changed to either leveling off or subdidying.

Judging from the above examples, it seems that in the case of a destructive large earthquake the secular diastrophism changes its mode about ten years prior to the earthquake. The follow-up survey on any change in the mode of diastrophism is the most important task in earthquake prediction. In this respect, repeated survey of 6,000 triangulation points of the second order and above with a cycle of five years means at least two observations obtainable during the period of abnormal movement, and makes it possible to capture premonitory signs at an early stage. It is, therefore, extremely important and affective in promoting the practical application of earthquake prediction.

Apart from extremely large earthquakes of the M-8 class occurring in the waters off the Pacific coast, prediction of those earthquakes of Tanna, Mikawa, Tottori, Fukui and Niigata class occurring in inland areas is also socially very important in Japan. Prediction of these earthquakes will be possible only with repeated survey with a five-year cycle on primary control points forming networks of 8km on average.

In addition, the establishment of a precise survey network is expected to contribute to the solution of Japan's various geological problems which have long been left unsolved. For instance, since the Meiji period various studies have been made of the Median tectonic line which divides Southwest Japan into the inner and outer belts. A view has been put forward that the Median tectonic line is a strike-slip fault and that the strike-slip has continued through the latest geological period and is still in progress. Thus, in order to verify this hypothesis, geodetic detection of the present strike-slip of the Median tectonic line is the most direct approach. However, the conventional repeated survey of first order triangulation is not adequate for that purpose. Repeated survey by the precise geodetic network will, therefore, contribute to the solution of this problem.



Diamond base line

Another important problem in earth science concerning Japan is the significance and present activities of the Fossa Magna, the great structural line dividing the Japanese Islands into Northeast and Southwest Japan. There are also the activities of large scale active faults seen in various parts of the country. Thus the Japanese Islands pose numerous geological problems yet to be solved, and most of them will be solved in the near future through carefully repeated survey by the precise geodetic network.

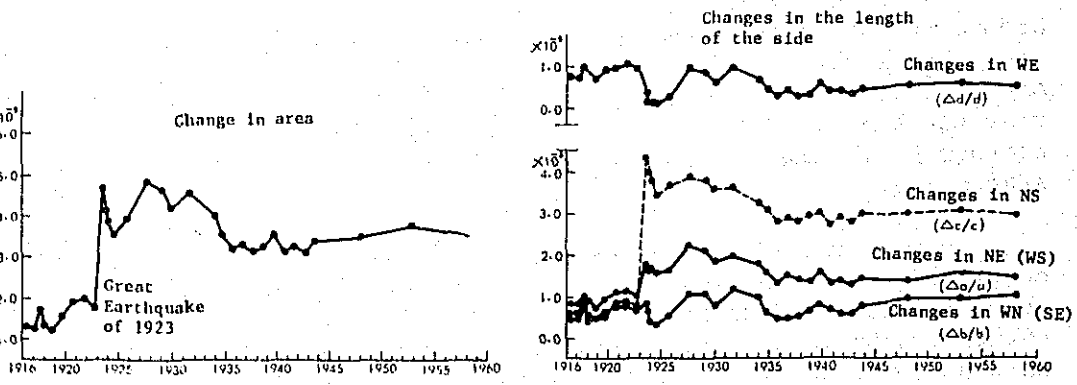


Fig. 24. Changes in the diamond base line at Mitaka

CHAPTER 6. JAPAN'S INTERNATIONAL RESPONSIBILITIES

It has been 100 years since the introduction of modern survey techniques into Japan. During the last century, the Japanese have tried their utmost to absorb the Western theories and techniques, completed the nation-wide geodetic network in the Meiji and early Taisho periods and carried out survey on the large earthquakes that had occurred, thus providing the most reliable data for the elucidation of vertical and horizontal diastrophism or subsidence.

Further, in recent years, they have come to realise that precise geodetic instrumentation is the most dependable method for the practical application of earthquake prediction, an epoch-making operation in the history of solid earth science. They have also commenced the geodetic verification operation concerning Plate tectonics, which is drawing the world-wide attention, under the International Geodynamics Project from 1972.

Since geodetic survey provides the basis for national land, it is primarily one of those national projects to be carried out by the State. In the case of Japan especially, the Government has the responsibility to improve and consolidate the precise geodetic survey work ahead of other countries for the following reasons:

(1) The Japanese Islands are situated in the most active circum-Pacific orogenic belt in the world, most suitable for various studies in solid earth science such as earthquakes, volcanoes, and diastrophism. Further, Japan has accumulated since the

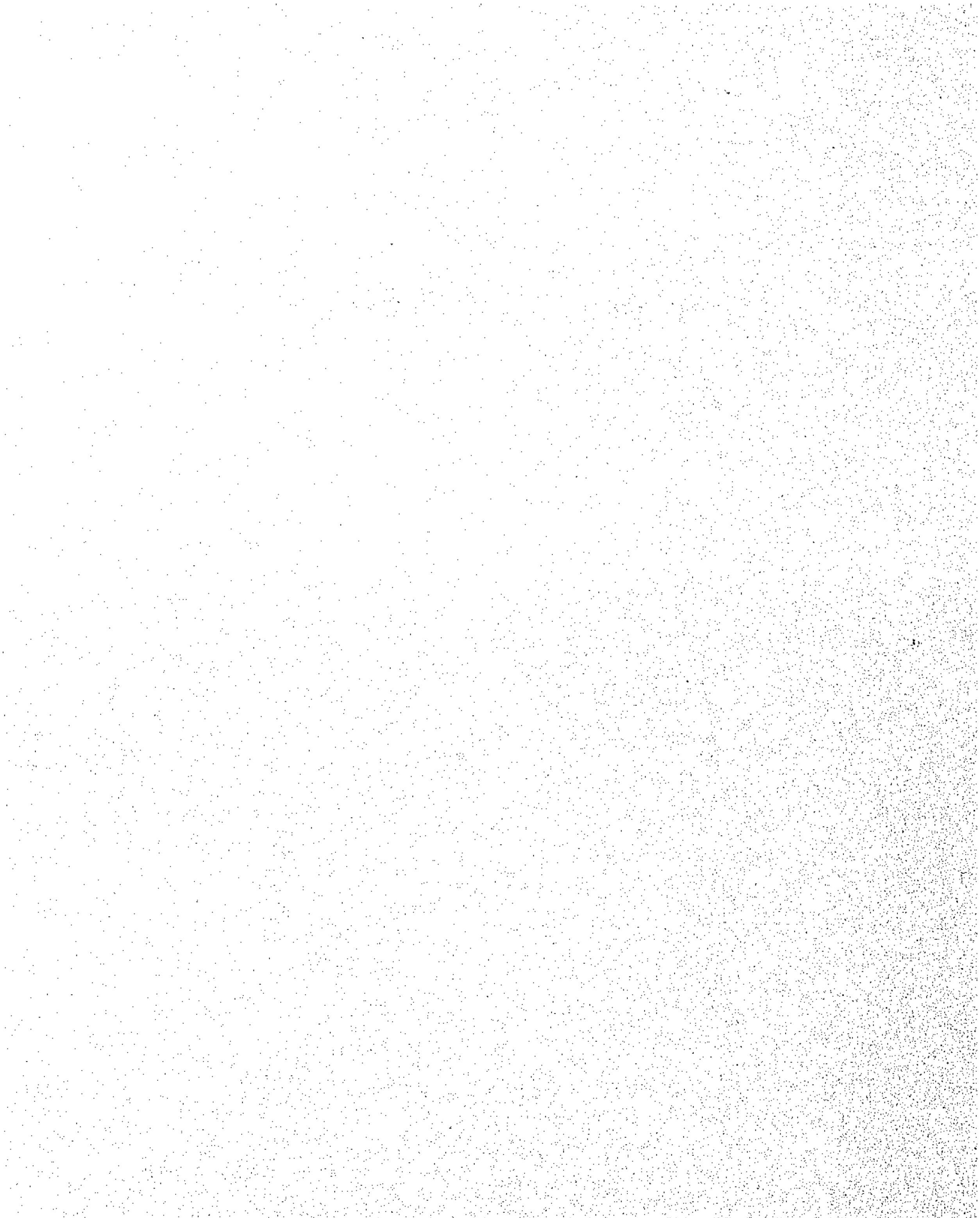
Meiji period a vast amount of data in earth science which are highly valued in the world. In fact no other advanced country has such geological environment and highly intensive observation network as possessed by Japan;

(2) With the population of 100 million in a limited area of land, there are strong social needs in Japan for development and conservation of national land disaster prevention. At the same time, however, Japan possesses technology and economic power surpassing many other countries, and is, therefore, capable of carrying out geodetic survey of a high precision with desired results.

In view of the above reasons, it seems that there is no other advanced country than Japan which has the need and the capacity to commence this work. In the past Japan adopted various cultures from other parts of the world, first from Continental Asia and then from the West. With Japan entering the centenary of the opening of the country to the outside world, the time has perhaps arrived and the background arranged for Japan to commence the Precise Geodetic Network project as a pioneering undertaking.

PART II.

PRECISE GEODETIC SURVEY



CHAPTER 1. OBJECTIVES AND CONCEPTION

The necessity for setting up a precise geodetic network and the outline of the project have already been described. Since the primary control survey as the uncleus of the project is in nature destined to succeed to the role of the conventional first order triangulation survey and to develop it further, thus having a very serious significance, it is important that those who are engaged in the project of setting up the precise geodetic network fully recognize it.

The primary control point network consists of first and second order triangulation points, and provides high density survey network as the base of the control points in our country, to supersede the conventional first order triangulation survey network. This corresponds to "the revolution from the method based on the angle observation to the survey system based on the length measurement by the appearance of practically usable electro-optical distance meters" as often said, and virtually changes the survey method in which a large triangulation survey network is reduced into smaller triangulation survey networks by turns. Therefore, it is necessary to think back to the valuable contents of experience by the first order triangulation survey made in pursuit of as precise measurements as possible until 1973, and to integrate the conceptions of engineers for precise survey based on angle measurement cultivated for the past about 100 years and the results of pioneering technical development in measurement techniques and precise measurement by electro-optical distance meters, in the effort to inherit them in the primary control survey and to create new ideas and contrivances based on it. In this way, we must devote utmost efforts to secure the precision of 1×10^{-6} which could not be realized by the first order triangulation survey. This must be also understood as the start of a new geodetic instrumentation project.

The first order triangulation survey was applied throughout Japan from 1882 to 1913, based on angle measurement, to form the first order triangulation survey network. For the absolute values of side lengths of first order triangulation survey network, base line measurement was made by Hilgard measuring rods, invar wires, etc. in 14 base line fields (base line length 3 to 10km) positioned at intervals, almost 200km of and the base line lengths were used to compose the base line triangulation survey network, for expansion to side lengths of approx. 45km between first order triangulation survey points. Recently, with the execution of repeated survey of first order triangulation survey, the length survey of 60 sides was being planned and executed. For this side length surveying, measuring instruments by lightwaves were used. With the recent progress of laser in practicality, rapid development is expected to be made in the project, and the technical development of measurement by laser has been positively promoted.

In the past, in the base line measurement to give reference for length, 25m invar wires of approx. 1.7mm dia. with very small expansion coefficient of 1 to 9×10^{-7} were used for connection by indexes on straight lines of 3 to 10km with little slopes, requiring considerable care, efforts and cost. With 1m Japanese standard meter No. 20 as standard length, 5m Guillaume standard scale almost equal to it in expansion coefficient is calibrated with relative precision of about 1μ, and using it, 25m invar wires are calibrated with relative precision of approx. 10μ. This work requires thorough-going preparations such as bisection by a microscope of 30 magnifications and projection by 1-second level, skill in measurement technique and discussions of various errors involved in the measurements. The experience in these severe precision measurements has provided a good foundation for the execution of first order triangulation survey. Examples of the measurement precisions are shown below.

Calibrated value of 5m Guillaume standard scale
(October, 1954) 5m + 67.74μ±0.94μ
Calibrated value of No. 56 25m invar wire
(February, 1958) 25m − 0.65mm±4μ
Tenjino base line wires (1953) 3,153.7955m±0.39mm
(Median value in shuttle measurements of 7 wires, the maximum difference in shuttle measurements of one wire ... 5.8mm, minimum difference ... 0.3mm) Aibano base line wires
(1954) 3,065.7281m±0.58mm
(Median value in shuttle measurements of 5 wires, the maximum difference in shuttle measurements of one wire ... 3.0mm, minimum difference ... 0.3mm)

As obvious from the above, the relative precision of base line surveying is as very high as 2×10^{-7}, but when it is magnified to a side length of approx. 45km of first order triangulation survey network, the precision drops to about 1/400,000, even if the standard deviation in the angle measurement (one direction) of the base line triangulation survey network is 0".3 to 0".4. Furthermore, in our country there are few places where base line surveying can be made again, in the past base line fields, etc. In this situation, the development of electro-optical distance meters with precision of 1×10^{-6} assuring practical use is very good news.

The precision of angle measurement depends upon stable measuring marks, selection of observation time, careful handling of measuring instruments, the promptness and minuteness of measurement (equipment and observation work to minimize the influence due to the rotation of measuring instrument, adjustment of level, and external force), etc. Therefore, column suspension type observation platforms of at least 3m or more were constructed one year before, and point light sources such as heliotropes and signal lamps were used as aiming points, with measurement made under different weather conditions of morning, evening and night, considering the lateral refraction of light. Such measurement took at least 2 to 3 days. However, the error by the lateral refraction of light

$$\Omega = (1/L)\int_0^L (L-\ell)/\sigma \cdot d\ell \qquad 1/\sigma = (16.5P/T^2)dT/dx$$

L ... distance, T ... absolute temperature, P ... atmospheric pressure, dT/dx ... horizontal temperature gradient

could not be removed entirely, and the mean error of one angle could never attain any precision beyond 2.5×10^{-6}. For this reason, much could not be expected on the relative precision of coordinates of first order triangulation survey points. On the contrary, in the length measurement using electro-optical distance meters, a relative precision of 1×10^{-6} can be obtained. The rigid survey based on side length measurement enables to improve the precision of the coodinates of national control points, and also in comparison with repeatedly directly measured values, this is greatly expected to be applied practically for earthquake prediction, etc.

CHAPTER 2. PRECISION OF MARKS

The objective of the primary control survey is to seek more precise geodetic coordinates, as mentioned before, and this requires as high precision as possible in the direct measurement of sides and angles. This enables to analyze the strain of crust from the change of measured values by surveying repeatedly. The precision of x-y coordinates can be evaluated in terms of error ellipse by

$$m_p^2 = m_x^2+m_y^2 = m_0^2\ Q_x+m_0^2\ Q_y = m_0^2\ (Q_x+Q_y) = m_0^2\ Q$$

In this equation, m_0 is determined by observation precision in the field, and Q is determined by the strength of geometrical figure and the quantity of observation. Therefore, how efficiently the $m_0 \sqrt{Q}$ can be made smaller is a large problem we face now. Let's discuss Q and m_0 a little more in detail which have such important meanings.

Section 1. Q

To make the adjustment computation of survey network by the observation equation of x-y coordinates, if an observed value is ℓ, its correction is υ, unknown most probable value is X, and its approximate value is X_0, then expressions $\ell+\upsilon = f(\overline{X})$ and $\overline{X} = X_0+X$ can be established. Hence the observation formula is $V = AX - L$, and from the condition of $V^T PV = \min$, a normal equation $BX = R$ ($B = A^TA$, $R = A^TL$) is obtained, with the correction of X_0 obtained from $X = B^{-1}R$. (With regard to adjustment computation, detailed description will be made under a special chapter devoted to it). The diagonal element in the elements of inverse matrix B^{-1} in this calculation equation is the Q which is going to be discussed here, and the summation is expressed by trace (Q). Since free adjustment computation gives $\det(B) = 0$, not enabling to obtain X, and method of, for example, (1) one-point one-azimuth fixing, (2) $\Sigma(X+Y) = 0$ and one-direction-fixing, (3) $\Sigma(X^2+Y^2) = \min$, etc. is taken as a minimum condition, and the value of Q depends upon the methods. Therefore, it is decided to use two methods; Mittermayer's method ($\Sigma(X^2+Y^2) = \min$) which is considered the best for evaluating the precision of the survey network as a whole, because of trace (Q) = min, and the one-point one-azimuth fixing method which is the simplest in computation, and does not involve complicated influence exerted on the variations of coordinates by hypothetical conditions.

As for methods of surveying, there are trilateration-triangulation survey, trilateration, triangulation survey, plygonal surveying, etc. Results of model computation for primary control survey, excluding polygonal surveying, are shown in Figs. 1.1 to 1.3. As obvious from them, it is naturally proven that trilateration-triangulation survey is the best, and it is known that triangulation survey is inferior to trilateration. As mentioned already, the distance measurement by electro-optical distance meter is far better than the angle measurement in precision, making the value of m_0 considerably small, and therefore, considering precision only, it is concluded that trilateration should be employed. However, trilateration is estimated to be more costly than triangulation survey, and is not quite advantageous as light of a project. Meanwhile, considering that if length measurement is increased in triangulation survey, Q becomes small gradually, and it is necessary to compare with trilateration in view of economy. Therefore, if, in the combination of length measurement and angle measurement, survey network of simple triangles is divided into triangle-connected polygons, with measurement of all the sides and angles made only at the central triangulation survey point of each polygon, observed points become about 30% of all the points, and this is surmised to be economically advantageous. The values of Q by such surveying method with angles and sides combined were confirmed to be about 20% different from those of trilateration, as shown in Figs. 2.1 to 2.4. Therefore, it was decided that the use of this method (umbrella method) for primary control survey be permitted.

Section 2. m_0

As described before, $m_0 = \sqrt{V^T PV/n-u}$ as the unit weight of measured value depends upon the performances of measuring instruments and quality of observation. The performances of electro-optical distance meters and theodolites used for primary control survey and the comprehensive precision are as follows:

(a) Electro-optical distance meters

Laser with light source output of 1mmW or more should be used.

Error sources and values of distance meters

(1)	Reading error (under stable weather conditions and good reflected light)	±2mm
(2)	Phase difference determining error	±3mm
(3)	Instrument constant error	±2mm
(4)	Main instrument centering error	±1mm
(5)	Reflector centering error	±1mm
(6)	Modulation frequency error (to be constant in the sign of error)	$\pm 4 \times 10^{-7}$
(7)	Frequency error (dispersion against a constant value)	$\pm 5 \times 10^{-7}$

If the meteorological correction error is collectively 1×10^{-6} D (D = measured distance), in addition to the above errors, the electro-optical distance meter has a measurement precision of $\pm 5mm \pm 1 \times 10^{-6}D$. Geodimeters Model 8, Model 6BL, etc. for long distance have the above performance.

(b) Theodolites

Theodolites should correspond to Special Grade Transit in accordance with the Surveying Instrument Performance Standard of Geographical Survey Institute (Classification Table by transit performance), and should be generally as follows. Wild T3, etc. have such performance.

Telescope	Effective aperture: approx. 60mm, Shortest sight distance: approx. 10m Magnification; Aperture (mm) × 1/2 × 1.5
Graduated circle	Dia. approx. 100mm Minimum graduated circle 0".2
Read method	Precision optical micrometer or equivalent
Plate level	Sensitivity 10" or more

Distance measurement includes a very important element of meteorological observation, in addition to the measurement techniques by distance meters, and therefore is described under a special chapter devoted to it. Since the errors of elements (temperature t, atmospheric pressure p, steam pressure e) for obtaining the refractive index on light path affect measured distance D by $dD = (\pm 1.0dt \mp 0.4db \pm 0.055dc) \times 10^{-6}D$, the comprehensive precision of measured distance is set as 2×10^{-6} for the time being to be attained by making 3 sets of measurement, considering also the above mentioned performance of electro-optical distance meters. For the secondary control survey, the relative precision is set as 5×10^{-6} to be attained by making 2 sets of measurement. In fourth order triangulation survey, the relative precision is set as 1×10^{-5} to be attained by making 1 set of measurement.

For angle measurement, a precision of 5×10^{-6} (mean error in one direction) has been decided to be secured by 6-pair times of direction method by a special grade transit, in reference to the data of first, second and third order triangulation survey. In secondary control survey, the standard precision of angle measurement (one direction) is set as 7×10^{-6} to be secured by making 3-pair observations of direction method by a 1st grade transit, and in fourth order triangulation survey, it is set as 9×10^{-6} to be attained by 2-pair times of direction method by a 1st grade transit.

Section 3 Expression of the Comprehensive Precision of Survey Network

The comprehensive precision of the survey network as a whole is expressed by $\overline{m}_p = m_0 \sqrt{\text{trace } Q/n}$, using the result of x-y network adjustment computation by Mittermayer's method ($\Sigma(X^2+Y^2) = \min$), and the target precision is 3cm. The results of model computation (x-y network adjustment by Mittermayer's method) to determine this value are shown in Figs. 3.1 and 3.2. Results by one-point one-azimuth fixing method are shown together for reference.

In secondary control survey, primary points are fixed as given points, unlike primary control survey, but $\overline{m}_p = m_0 \sqrt{\text{trace } Q/n} = 3\text{cm}$ is used as in the case of primary control survey. The results of the model computation (x-y network adjustment computation) are shown in Figs. 4.1 to 4.7.

To be noted here is that a large problem of estimating a weight arises for adjustment computation to handle quantities of different natures, viz. measured values of sides and angles simultaneously. In the survey by angle measurement only, the number of measuring times can be simply taken as a weight, but when length measurement and angle measurement are combined as in the case of precise survey network, the weight must be determined, using the respective measurement precisions.

In the primary control survey, the weight is determined with the mean error of measured angle in one direction as 1", and that of measured distance as $\pm 5\text{mm} \pm 2 \times 10^{-6}\text{D}$, and in the secondary control survey, with them respectively as 1".4 and $\pm 10\text{mm} \pm 3 \times 10^{-6}\text{D}$.

CHAPTER 3. RECONNAISSANCE

The reconnaissance is the work to finally determine a triangulation survey network by deciding the surveying method, for example, how to combine trilateration (including broken base lines) and umbrella method according to the topographic features and other field conditions, and also by deciding working method, means etc. such as the heights of observation platforms, heights of reflectors, types of measuring marks, locations of camping, and routes for climbing up and down mountains, confirming the intervisibility through triangulation survey points based on paper plant decided to obtain coordinates at the target precision. This is a very important field which determines the precision and the cost of surveying, and reconnaissance must be made carefully. To be concrete, this refers to what survey network is composed and what working steps are determined, considering the magnitude of Q and economy described in the previous chapter. For example, if high signal towers are erected as many as needed, with trees felled at random and any steep mountains selected as observation points, paper planning will be enough. This is the importance and difficulty in the reconnaissance for primary network. In case of observation work, etc. relatively detailed specifications are made out. and observation in accordance with the specifications results in no large mistakes in general. However, since reconnaissance depends greatly on the field conditions, only basic principles are specified, with much freedom entrusted to engineers. Umbrella method is a very economical method which enables to set steep mountains as reflecting points and to set intervisible sides with many trees to be felled and sides with no topographical intervisibility as sides to be calculated, and therefore how to employ this method intelligently is an important point in reconnaissance. The reconnaissance can be boiled down to the work of sheer walking and is the worthiest working process for engineers.

Section 1. Figure of Triangles

The magnitude of Q is determined by the strength of figure and the quantity of observation. However, since the quantity of observation is specified in the working regulations by various conditions, the strength of geometrical figure only (including the combination with measured angle) is discussed here.

The observation equation for x-y network adjustment is, as already mentioned

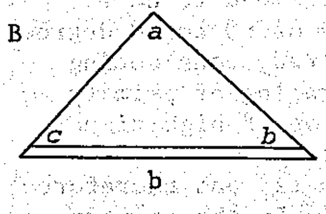
$$\upsilon ij = dzi+aidxi-bidyi-aidxj+bidyi-lij,\ \upsilon ij = -bidxi-aidyi +bidxj+aidyi-(Sij-S'ij)/S'ij,\ \text{where } ai = (yj-yi)/S^2ij \quad bi = (xj-xi)/S^2ij \quad \cdots\cdots \quad (3.1)$$

Hence, Q inverse number of the weight of x and y is a function of a and b, viz.

$$f\{(yi-yi)/S^2ij,\ (xj-xi)/S^2ij \quad \cdots\cdots \quad (3.1)'$$

This equation shows that the magnitude of Q is uniquely determined by the form of triangle, and the adjustment chart we decide by reconnaissance determines it. It must be fully remembered that however small m_0 may be with attention paid to observation precision only, large Q caused by loose reconnaissance spoils the effort made for m_0. What form of triangle will make Q small ? This question appears very complicated, as far as equation (3.1)' is concerned. However, unknown x and y are calculated, using the side lengths (S) and angles (a) of triangle. Therefore, if three interior angles of a triangle are measured with one side length given, the angles must be used to calculate the other side lengths, and if three side lengths only are measured, they must be used, to calculate the angles. In other words, that question can be replaced by a simple question: what triangle enables to minimize the errors of calculated values affected by the errors of measurement?

First of all, let's consider the errors affecting calculated side lengths when the interior angles of a triangle are measured.

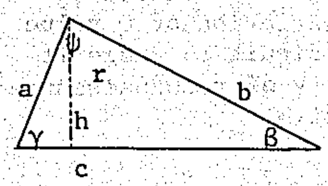


Calculated side length is $B = b \sin b/\sin a$, hence if the weight of each measured value of angle is 1, then

$$m_B = m_0\{(2/3)(\cot^2 a+\cot a\cdot\cot b+\cot^2 b)\}^{1/2} \qquad (3.2)$$

Therefore, the adjustment chart can be drawn, to have triangles which make $(\cot^2 a+\cot a\cdot\cot b+\cot^2 b)$ small. In reference to a simple triangle, this value is shown in Fig. 5.

Next, when three side lengths of a triangle are measured, the influence exerted by the measurement errors of side lengths on the included angle is expressed by the following equation.



$$c^2 = a^2+b^2-2ab\cdot\cos\psi$$

hence $m^2\psi = (c/h)^2\{1+(a-b\cos\psi)^2/c^2+(b-a\cos\psi)^2/c^2\}m^2$

where $dc/c \doteqdot da/c \doteqdot db/c$ is assumed, and the standard deviation is expressed by m.

$$\qquad (3.3)$$

Therefore, an adjustment chart can be determined to have triangles which make $(c/h)^2\{1+\frac{(a-b\cos\psi)^2}{c^2}+\frac{(b-a\cos\psi)^2}{c^2}\}$ small. With regard to a simple triangle, this value is shown in Fig. 6. Figs. 5 and 6 show that the figure close to regular triangles is the best form both in triangulation survey and trilateration. When the measurement of one side length of triangle is omitted by measuring two side lengths and one included angle, the error of the calculated length can be expressed by the following equation.

$$m_c^2 = [\{(a-b\cos\psi)/c\}^2+\{(b-a\cos\psi)/c\}^2]m_a^2\cdot b+(h/c)^2\ m_\psi^2 =$$
$$\Lambda^2\ m^2{}_{a\cdot b}i+B^2m^2\psi$$

where $da/c \doteq db/c$ is assumed, and the standard deviation is expressed by $m_{a\cdot b}$ .... (3.4)
The values of coefficients A, B, etc. are shown in Figs. 7.1 to 7.4. Using these values, the measurement precisions of two sides and the included angle of a triangle can be determined, in order to secure the required precision of the median value of two calculated side lengths obtained by the two-side included angle method from two different triangles. Furthermore, from the inverse calculation and the measurement precisions of sides and angles, the magnitudes of Q can be approximately compared between umbrella method and trilateration. The drawings show that triangles with vertex angles of 60 to 80 degrees give good precisions.

According to the above result, if one side is omitted in the two-side included angle method for regular triangles in trilateration and triangulation survey, survey network must be determined to have equilateral triangles with measured vertex angles of 60 to 80 degrees. For this reason, comprehensive judgement is required, considering precision and economy in relation with the downgrading of primary points, upgrading of secondary points, construction of high signal towers, tree felling, eccentricity, etc. The survey network once determined is used repeatedly for surveying as it is, and therefore the first reconnaissance is especially important. In the primary control survey, since first and second order triangulation survey points are already established, most of reconnaissance is already completed. Thus, final reconnaissance is to be made based on the above. On the other hand, the umbrella method has the advantage that triangles can be formed by planning to calculate the sides which do not provide intervisibility due to trees, topographic features, etc., enabling to set triangulation survey points easy to climb up and down with less felling as observation points, as mentioned before. Therefore, this advantage should be sufficiently exploited. Because of relatively large freedom in reconnaissance, the variation of Q value is estimated to be very large, and it is very difficult to judge it just from a general view. Therefore, the propriety of reconnaissance is judged, by calculating Q on an adjustment chart.

Section 2. Eccentricity and Broken Base Lines

Even if points are selected ideally, too much eccentricity and too many broken base lines considerably lower the comprehensive precision of angles and sides corrected around the stone marker as known from the propagation of errors by equations (5.2) and (5.3) described later. In addition, the measurements of eccentricity elements are very likely to involve errors more than estimated.

Furthermore, the complicated computation lowers efficiency and is surmised to cause errors, etc. Therefore, eccentricity and broken base lines are allowed only when they are inevitable. Eccentricity is allowed only when intervisibility cannot be obtained even with a 4m high observation platform, for the above reason as well as for the purpose of preserving the precision of observed values themselves. For this reason, it is necessary to clearly state the distance of eccentricity for eccentric points, broken base lines, etc. on the adjustment chart, for comprehensive judgement.

CHAPTER 4. CONSTRUCTION OF TARGETS

After the points have been selected, the positions of survey points and heights of observation platforms and signal targets necessary for observation are determined. In conventional triangulation survey, all the points were observation points, and facilities with the structure of observation platform and signal target were constructed for all the points. However, the primary control survey is characterized by that targets must be built for only about 30% of points in trilateration and for only about 70%, in the umbrella method. Therefore, it is necessary to consider two types of target construction, and for a target, the installation of only signal lamp or reflector is sufficient, not requiring the conventional troublesome structure. Therefore, it will be efficient and economical to use portable signal targets together according to the field conditions of topographic features, etc.

Even if the height of target is about 1.50m, it is desirable to make a strong observation platform, but in the case of distance measurement at a rooftop observation point or ground observation point, it can be replaced by a tripod. If the height of measuring instrument is 1.50m or more, a frame for measuring instrument must be constructed by long logs, etc. Below are described details about the construction of targets.

(1) In the case of angle measurement, the observation platform must be strong enough to keep the slight vibration due to wind, etc. less than about 2/10 (1".5 to 2".0) of one graduation of the resolving power (approx. 2") plate level of telescope, since otherwise it propagates to the observed angle, to lower the precision of angle measurement, thus raising the resurvey rate. The vibration of a long period, for example, inclination or return of observation platform in a direction is a constant error. Therefore, it must be built firmly and symmetrically in thickness, form, etc. by straps, bracings, etc. using logs. Standard types of wooden targets satisfying these conditions as considered from the past experience are shown in Figs. 8.1 to 8.5. To make a portable metallic target, the structure must be considered especially. Stands of target and stands to support the tread board (in the case of ordinary target) must be erected as far away from the stands to support the observation platform as possible.

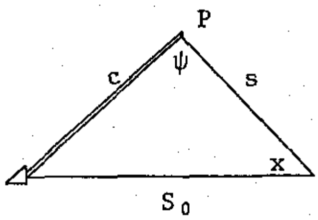
(2) In the case of distance measurement only, the slight vibration of observation platform can be within the range of beam width (approx. 20" for Model 8 or approx. 12" for Model 6BL) of emitted laser, and is not so rigid as in the case of angle measurement, being an advantageous point of trilateration.

(3) The signal target for a high target tower is a target platform to allow the action of the observer for observation. Therefore, a high target tower exceeding 10m in the height of observation platform, for example, must have sufficient strength, space and protective equipment to allow the safe observation of the observer. The allowance of slight vibration cannot be specified generally, in relation with the loss of light quantity, wind velocity to allow observation, etc., but will be sufficiently secured if the above mentioned condition is satisfied.

(4) As for an independent target platform (signal lamp platform, reflector platform), it can be very simplified for the reasons given above. Therefore, the development of portable targets is in need. The Geographical Survey Institute is now developing 25m single portable steel platform G tower, M tower.

CHAPTER 5. OBSERVATION

Section 1. Eccentricity of Angles and Sides



(a) When eccentric elements e and ψ are measured at the eccentric point P, to obtain an eccentric angle x, the errors of the eccentric elements affect the eccentric angle x are to be obtained. This is discussed below.

$$x = \mathrm{Sin}^{-1}(c/s \sin\psi)$$

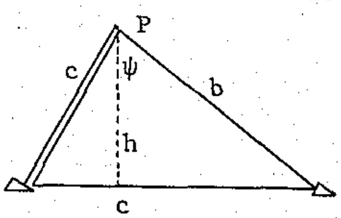
Hence $dx = (1/S_0)\sin\psi\cdot dc+(c/S_0)\cos\psi\cdot d\psi-(c/S_0^2)\sin\psi\cdot dS_0$ .... (5.1)

In the above equation, since the eccentric angle ψ depends upon the measuring direction at the same measuring point, the maximum values of $\sin\psi$ and $\cos\psi$ are taken for discussion. Thus, equation (5.1) is $dx \doteqdot (1/S_0)dc+(c/S_0)d\psi-(c/S_0^2)dS_0$. Hence, the error of the eccentric angle x to be obtained is

$$m_x^2 = (1/S_0)^2 m^2{}_c+(c/S_0^2)^2 m^2{}_s+(c/S_0)^2\ m_\psi^2 \quad \text{...........} \quad (5.2)$$

In equation (5.2), the first term should be made as small as possible by deciding a measuring method corresponding to the eccentric distance e, and since the second term is slight, due to the coefficient of (e/S_0^2), the error caused by ψ angle of third term only is considered. Thus, $m_x = (e/S_0)_{m\psi}$, and this is rewritten as $m_x(S_0/e) = m\psi$, from which if m_x is determined as a constant value, $m\psi$ is determined from the relation with (S_0/e). The $m\psi$ is a comprehensive measurement precision required for ψ angle, and the measurement unit, times of measuring pair, measuring method, etc. are determined to satisfy it. Not to lower the comprehensive precision (standard deviation) of corrected angle, $m_x = \leqq 1 \times 10^{-6}$ is determined for primary control survey and $m_x \leqq 1.25 \times 10^{-6}$, for secondary control survey. Since the standard deviation of eccentric angle for 1".4 standard deviation of one angle is set within 0".2 in the primary control survey, the comprehensive precision of one angle after correction does not exceed 1".41.

(b) When distance C between triangulation points is obtained by measuring b and eccentric elements e and ψ at the eccentric point P, the error on the side C to be obtained, by the measurement errors of e and ψ can be given by the following equation.



$$(dc/c) = (c-b\cos\psi/c)(dc/c)+(b-e\cos\psi/c)(db/c)+(cb\ \sin\psi/c^2)$$
$$(d\dot{\psi}/\rho,\ cb\ \sin\psi/c = h \quad\ldots\ldots\ldots\ldots\ldots\ldots\quad (5.3)$$

The first term is treated as done in the previous (a), and the second term is concerned with the measurement of C and can be neglected. Therefore, as in the case of the eccentricity of angle, only the error caused by ψ angle in the third term is to be considered. That is $(dc/C) = (eb \sin\psi/C^2)d\psi/\rho \doteq (e \sin\psi/c) d\psi/\rho$ where $b \doteq C$, hence $mc = (e/c)m$, giving the same form as in the case of the eccentricity of angle. As in the case of angle, $m\psi$ is determined and ψ measuring method is specified, not to make m_c exceed 1×10^{-6} in the primary control survey and 1.25×10^{-6} in the secondary control survey.

Section 2. Broken Base Lines

A side length between triangulation survey points is desirable to be measured directly, but under unavoidable circumstances due to topographic features, trees, structures, etc., broken base lines may be used. However, it is known from equation (5.3) that with regard to the precision of calculated side length (C) by the two-side included angle method, the error of measured angle exerts great influence in proportion to h/c, considerably lowering the precision of calculated side length. For this reason, if only the influence of measured angle error is considered in equation (5.3), then $(dS_0/S_0) = (h/S_0)d\psi$, and the error of side length (S) to be obtained is $m_s = (h/S_0)m\psi$. Using this equation, angle measurement is made not to allow m_s to exceed 1×10^{-6} in the primary control survey and 1.25×10^{-6} in the secondary control survey and to have 1".4 as $m\psi$ in the primary control survey and 2".0 in the secondary control survey, and the limit of $h \leq 0.1S_0$ is set.

Section 3. Distance Measurement

Distance measurement mainly includes the measurement work to operate electro-optical distance meters and the meteorological observation work to correct measured values, and it must be particularly noted that both are equally important. The survey network setting project was planned in the situation that highly precise distance measurement became possible, as already mentioned, but measurements to secure 2×10^{-6} in the field are not easy. Therefore, it is necessary to understand the theory and mechanism of distance measurement by light waves and to be well versed in the methods of handling electro-optical distance meters different according to models. This subject will be described in another textbook, and meteorological observation will be described under the special chapter for it. However, below are described the main points concerned with working regulations, though some contents are doubly described.

(Inspection and adjustment of electro-optical distance meters, etc.)

The first requirement is to perfectly service the main instrument and peripheral devices (device for meteorological observation, power supply, reflector). Before starting for fieldwork, the electro-optical distance meter must be calibrated in the modulation frequency, and the difference from the specified value must be adjusted to a precision within 5×10^{-7}, enabling to neglect $dD = (df/f)D$ due to the error of frequency. Furthermore, those who are engaged in fieldwork must check the instrument constant and comprehensive measurement precision and confirm the satisfactory performance of electro-optical distance meter, in the base line field, etc. They must also master the method of handling the electro-optical distance meter and distance measurement technique.

(Measurement by a distance meter)

The measuring method depends upon models, but common important matters are as follows:

(a) Since the variation of supply voltage lowers the precision of measurement, particular attention must be paid to the keeping of specified voltage. It is not allowed to continue measurement, assuming "this will be approximately good" for the psychological reason to finish measurement as soon as possible.

(b) It is desirable that the light intensify is equal between the measurement of reflector (R) and the measurement of conductor (C).

(c) Too intense reflected light in a short distance, etc. lowers the precision of measurement, and it is desirable to adjust the light intensity by the number of reflectors and a filter.

(Meteorological observation and distance measuring condition)

As described already, the error of refractive index (n) causes the following error to measured distance (D) due to dt, dp and de.

$$dD = dn \cdot D = (\pm 1.0dt \mp 0.4dp \pm 0.053de) \times 10^{-6}\ D \qquad \text{(5.4)}$$

Therefore, how do we find n on a light path extending over 10km? It refers to finding at precisions of 1 degree or less in temperature and 3mmHg or less in atmospheric pressure. For this reason, important studies as to how to measure t, p, and e at the observation point and the reflection point instead of direct measurement of t, p and e on the light path and whether or not they can be corrected by a function of elevation difference must be continued also in future to obtain a better measuring method. However, for the time being, the following method is taken in the primary control survey.

(a) Distance measurement must not be made for 10h to 14h, except when it is totally cloudy (cloudiness 10) and the wind is strong (wind velocity of 6m/sec or more, with sand dust and paper pieces blown up).

(b) When the measured temperatures at both the points are greatly different from the value calculated with a decrease rate of $dt = 0^\circ.5C/100m$, distance measurement has better be avoided.

(c) The measurement intervals between sets must be more than one hour. However, in the period from sunset to sunrise, the measurement intervals can be shortened to 20 minutes.

(d) Thermometers should be remote-controlled electric thermometers, and must be apart from planimetric features, vegetation, ground and structures by more than 3m, being suspended in places the most suitable for temperature observation on intervisible lines. And temperature observation should be made, minimizing the effect of solar heat and radiation from ground surface, human bodies, etc., as far as possible.

(e) Temperature observation should be made at intervals of 20 minutes from a time before starting the distance measurement, and should be finished by observing also after the distance measurement. The weather conditions (fine, cloudy, wind velocity) at the time of temperature observation should be recorded.

(f) Temperature observation should be made in principle at the same time both at the instrument point and the reflector point.

(g) Barometric observation should be made at the time of temperature observation only at the instrument point when the specific height difference is less than 1,000m, and the atmospheric pressure at the reflector point is obtained from the height measurement formula by atmospheric pressure, $\{P_2 = P_1 \times 10^{\Delta R/18464(1+0.003665t)}\}$, or the following formula obtained by modifying this formula. Barometers should be set up in lodgings, etc. and should not be moved frequently. Two types of standard barometer and measurement barometer should be used, and the change of the instrumental error of barometer should be watched.

$$P_2 = P_1 \mp 1.25\times10^{-4}(1-0.0037t)P_1\ \Delta h(m)$$
$$\text{(for field approximation, } P_2 = P_1 \mp 0.09\text{mmHg } \Delta h(m)) \quad \ldots\ldots (5.5)$$

where $t = (t_2+t_1)/2$, $\Delta h(m)$ ..... relative height
If $500m < \Delta h \leq 1,000m$, then, the correction of $dp = \{1.25\times10^{-4}(1-0.00366t) \times P_1\ \Delta h(m)\}^2/2P_1$ is made.

(h) The observation of humidity is omitted, and the correction of $0.6 \times 10^{-6}D(18^\circ C, 70\%)$ is made uniformly.

In the secondary control survey, t and P are observed only at the instrument point without limiting the distance measuring time, and t and P of the reflector point are obtained by using the respective decrease rate of ±0°.5C/100m and ±0.9mmHg/10m.

Section 4. Angle Measurement

As already described, the angle measurement precision greatly depends upon the performance of instrument, measurement technique and quality of observation platform, observation time and weather condition, etc. It must be again noted that the precision of calculated side in the umbrella method is lowered by $(h/S)m\psi$, and an effort must be made to secure 1".4 as precision of one angle.

(Inspection and adjustment of theodolites)

Before starting for fieldwork, it is necessary to discuss the functions and comprehensive precision of theodolite, using a collimator, etc. and to confirm whether it performs satisfactorily. Furthermore, it is necessary to master the measurement technique and to be familiar with the instrument. It is an effective method that those engaged in working calibrate the error of horizontal graduated circle, to master the measurement technique, since the comprehensive precision of instrument such as periodic error and accidental error of graduations, and the constant error of instrument can be known simultaneously. The periodic error of graduations is, according to Fourier analysis,

$$\Delta H = A\sin H \cos(\theta+\theta'+\delta_1)+B\sin 2H \cos\{2(\theta+\theta')+\delta_2\}$$

H... included angle, θ, θ^1 ... read values in first and second directions (there should be no relative height difference against the target) .... (5.6)

Since this value is contained in a double angle, it is useful to discuss double angle difference.

(Precausions for measurement)

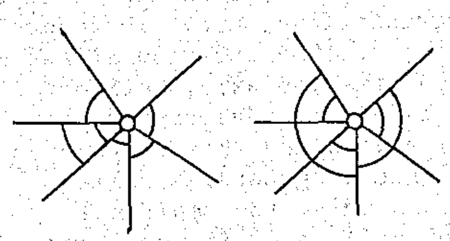
(a) The cardinal point of observation is to observe carefully and promptly.

(b) Observation is desirable to be made when light source can be seen clearly as a point, mostly about the time of sunset and sunrise.

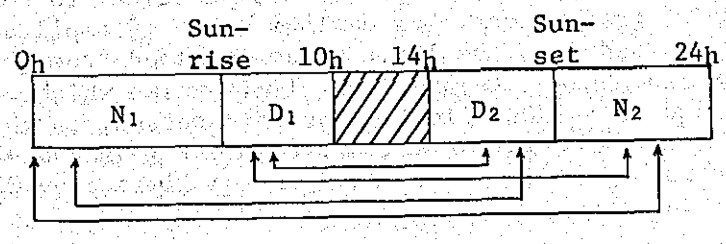
(c) Unlike the case of distance measurement, since it is substantially impossible to remove the influence of lateral refraction by meteorological observation, angle observation should be planned, considering that the mean value obtained by combining the measured values at time zones as different as possible is a more probable value.

(Observation methods)

Major observation methods of horizontal angle are angle method and direction method. In the primary control survey, the angle method as shown in Fig. (a) is surmised to be proper, but even this method requires a considerable number of observation days by the organization of surveying party now considered by us. For this reason, either the angle method or direction method may be used, as far as it does not cause station requirements in a given combination of two directions optionally selected from the light visible directions, to measure the included angle, as shown in Fig. (b).


(a) (b)

Observation order is established according to various conditions. At first, one set of observation is made on an included angle, and successively observation of 2nd set is made. In this method, observation is made for all (n-1) included angles (n is the number of directions), and then one set of measurement is made at a different time zone for each included angle, in the effort to replace the measured value obtained before. Desirable combinations of different time zones are N_1-D_2, N_1-N_2, D_1-D_2 and D_1-N_2 as shown below.



(Rejection of observed value, and double angle difference and observat observation difference)

In the observation of horizontal angle, it is a very difficult problem to judge the acceptability of observation and to reject poorly measured values. In precise measurement, with measurement of pair times as one set, the number of observation sets is decided according to the precision demanded. For example, in the conventional first order triangulation survey, with three-pair times (6 measured values) as one set, the standard deviation of one set is considered as

$$\sigma_x^2 = \sigma_u^2 + \sigma_\upsilon^2/6 \quad \ldots\ldots\ldots\ldots \quad (5.7)$$

and σ_u = 1".00 and σ_υ = 1".69 (these values are a little larger than those obtained by using all the conventional data) are obtained from measurement data. The value of σ_u is mainly due to lateral refraction and is commonly contained in the measured values of one set, and σ_υ is the standard deviation of one measured value in one set. These values and the measurement data of second and third order triangulation surveys are used to decide the standard precision of primary control survey as σ_X = 1".4, to obtain the limit between sets and the limits of observation difference and double angle difference.

The readings of normal state of transit (r) and reverse state of transit (l) contain the error of collimation axis affecting as a function of elevation angle, error of horizontal axis, error due to the difference (circumcenter) between the center of graduation circle and the center of rotary axis, error of graduations, etc. Therefore, since $(r - \ell)_{0°}$, $(r - \ell)_{60°}$ and $(r - \ell)_{120°}$, viz. observation difference contains the constant errors concerning collimation axis, horizontal axis and circumcenter commonly as constants, it is advantageous for judging the acceptability of observed values. Since this difference (the range of difference of observation; this is generally called observation difference) is accidental errors only, the empirical standard deviation obtained from equation (5.7), etc. is used, and the range distribution function (Numerical Statistics Table P. 206) is used, to specify the limit value of observation difference. Meanwhile, $(r + \ell)_{0°}$, $(r + \ell)_{60°}$ and $(r + \ell)_{120°}$, viz. double angles have periodic errors of graduations only though the constant errors contained in observation difference are almost eliminated, and therefore correction using equation (5.6) leaves accidental errors only (vertical axis error is considered as an accidental error in this case). Thus, the standard deviation of double angles can be obtained empirically from equation (5.7), etc., to obtain the limit value of double angle difference as in the case of observation difference. In the primary and secondary control surve survey, it is not practical to correct the periodic errors of graduations, the maximum value of amplitudes of periodic errors is added to the limit value obtained from the standard deviation of double angles, to provide a limit value which can be generally regarded as a loose limit. For this reason, in the conventional first order triangulation survey, the most reasonable observation difference only was specified, and there was no limit provided for double angle difference.

As the next problem, the dispersion among sets is greatly affected by the value of lateral refraction which cannot be estimated as an average value, in addition to observation errors. It corresponds to σ_u of equation (5.7), and though it is a problem to assume that the dispersion follows normal distribution, the limit value is provided, using σ_u = 1".00 as an empirical value. Therefore, when one is confident of the result of observation, one should use the observed value without rejecting it, irrespective of this limit, but based on experience for many years, the values exceeding this limit are checked by resurvey.

Thus, the acceptability of measured values of one set is carefully judged from both viewpoints of observation difference and double angle difference, and also with regard to the dispersion among sets, large errors are going to be removed by statistical judgement.

CHAPTER 6. COMPUTATION

In executing numerical computation, two elements of rapidity and accuracy are required even now when electronic computers are developed as well as in old times when logarithm was resorted to. It is a waste of cost and time, causing errors, to use more digits than required for computation and to make useless complicated computation. Matters to be noted in numerical computation mainly include the selection of computation method, proving method, and the relation between error range and significant figures.

Section 1. Correction of Measured Values

1) Meteorological correction of measured length by electro-optical distance meter

Since a measured distance is obtained by $D = (t/2)C$

t ... Time for light to shuttle in D (distance between instrument point and reflector point)

Light velocity C in the air on the light path must be obtained, to obtain the measured distance. The refractive index n of air on light path is $n = C_0/C$ $C_0 = 299,792.5$km/sec ..... Light velocity in vaccum

C ... Light velocity at the time of distance measurement

Hence, $D = (t/2)(C_0/n)$. In the meantime, in the electro-optical distance meter, intrinsic standard refractive index (n_s) is determined, and from the light velocity ($C_s = C_0/n_s$) determined based on it, the apparent measured length $D_s = (t/2)C_s$ is read. The above is expressed by

$$D = (t/2)(C_0/n) = (t/2)C_s(n_s/n) = D_s(n_s/n)$$

and if n is known, a correct distance D can be obtained, using the measured length D_s. This calculation is the meteorological correction of this section. Using that the refractive index is very close to 1, calculation is made as follows:

$$D = D_s(ns/n) = D_s(1+\Delta s)/(1+\Delta n) \fallingdotseq D_s + D_s(\Delta s - \Delta n) \quad \ldots\ldots \quad (6.1)$$

$$n = 1+\Delta n = 1+ \frac{ng-1}{1+\frac{t}{273.2}} \times \frac{P}{760} - \frac{15.02c}{273.2+t}$$

$$ng-1 = \{287,604 + \frac{3\times1.6288}{\lambda^2} + \frac{5\times0.0136}{\lambda^2}\} \times 10^{-6}$$

$n_s = 1 + \Delta s$ ..... Intrinsic standard refractive index of distance meter (1.0003086 in case of G8 and G6BL) p, t, e ...... Values by meteorological measurement at the time of distance measurement (mmHg, °C, mmHg)

λ .... Light wavelength expressed in μ (laser 0.6328μ, infrared ray 0.9μ, mercury 0.549μ, tungsten 0.55μ)

In the primary control survey, as humidity correction $(15.02 \times e/273.2 + t \times 10^{-6})$, the standard vlaue of 0.6×10^{-6} (temperature 18°C, relative humidity 70%) is used. For G8 and G6BL, calculation according to the above equation gives the following euqation.

$$\Delta s - \Delta n = (309.2-107.92 \times P/273.2t) \times 10^{-6}$$

2) Projection of measured distance (D) onto the reference ellipsoid

For the reasons that the detailed relation between reference ellipsoid and geoid is not known sufficiently, that the reference ellipsoid now used in our country may be revised in future, that the curved surface at the latitude of a certain site of reference ellipsoid is very approximate to the shpere with the average radius of curvature by the latitude of the site, and so on, the slope distance between two points is

$$\begin{aligned} S = {} & D-h^2/2D-h^4/8D^3-h^6/16D^5-5h^8/128D^7-7h^{10}/256D^9-2/h^{12}/1024D^{11} \\ & -Dhm/R+h^2hm/2RD+h^{11}hm/8RD^3+h^6hm/16RD^5 \\ & +D(5h_1^2+5h_2^2+6h_1h_2)/16R^2+D^3(1-K)^2/2\mu R^2 \quad \ldots\ldots\ldots \quad (6.2) \end{aligned}$$

where $h_1 = h_2 = h \leqq 3km$, $D = 5km$ to $50km$

In this equation, S can be sufficiently obtained to the unit of mm.

For the approximate calculation in the field in the primary control survey, the following equations can be used.

(1) $D_c = D - dD_1 \qquad dD_1 = D-D\sqrt{1-(k/D)^2}$

(2) $S = D_0 - dD_2 \qquad dD_2 = D_0-D_0R/(R+km)$

(3) If (h/D) is especially large, whether or not the correction of $(1/8)(hm/R)(h^4/D^3)$, etc. must be made should be discussed in reference to the following table.

(h/D)	$(1/8)(hm/R)(h/D)^4$	$(1/16)(hm/R)(hD)^6$
0.4	$1.5\times10^{-6}\times S$	1.2×10^{-7}
0.3	$5\times10^{-7}\times S$	—
0.2	$1\times10^{-7}\times S$	—

By averaging, the most probable values of h and a are obtained. Therefore, using the most probable values, S is corrected, as a measured side length used for x-y network adjustment. In elevation computation h = S tan a, the influence of error ds of S on h is expressed by dh = ds·tan a, but since tan a is normally small, the impact on the unit of cm is little caused, not requiring the correction of h.

3) Correction of measured angle to the reference ellipsoid

The included angle obtained by pointing aiming points B and C with a transit set up at point A is a spherical angle. This angle is an angle formed between the great circle containing the direction of vertical axis of the transit at point A and aiming point B and two great circles containing aiming point C. Therefore, when the direction of the vertical axis of the transit (vertical line at the point) does not coincide with the normal line of reference ellipsoid, viz. when there is vertical deviation θ, for the same reason as the vertical axis error due to the insufficient setup of the transit, the following correction must be made,

$$-\cot \zeta(\xi \sin a - \eta \cos a)$$

where ζ ..... Zenith distance from point A to point B

$\xi = \psi_a - \psi_g$ ..... Meridian distribution of θ

a ..... Azimuthal angle in pointing direction

$\eta = (\lambda a - \lambda g)\cos \psi$ ..... East-west line component of θ

but since θ is not known sufficiently, this correction is not made for the time being in the primary control survey. Also in measured distance, since the elevation H used for projection is not a value from the reference ellipsoid, it is assumed to be projected onto geoid. When the elevation of aiming point B is h ($\neq 0$), the great circle containing the normal line of point A and the aiming point B does not mostly contain the normal line of point B, causing an error of $da = (h/N_1)\eta^2 \cdot \rho'' \sin a \cdot \cos a$ as already mentioned. However, this is a negligible value in the primary control survey.

From the above, a measured included angle is treated as an angle on the reference ellipsoid, with no correction applied, for the time being in the primary control survey.

Section 2. Adjustment Computation of Measured Values

All the measured values are projected on the reference ellipsoid, and all the constant errors, even if corrected, contain accidental errors which follow a normal distribution. In general, since coordinates to be obtained are planned to be obtained from as many different routes and observed values as possible, the calculated coordinate values do not generally coincide, causing discrepancy. For this reason, adjustment computation of measured values is made for each survey block, to obtain the most probable values of coordinates. This is done by either method of condition equation or observation equation, and on a reference ellipsoid or a plane. Both the primary control survey and secondary control survey employ the method of observation equation on a plane. In the adjustment computation for a large block covering several survey areas of field work as in the case of primary control survey, B-L network adjustment with Laplace equation added is made. Theoretical details concerning these are described under the special chapter for them, and this section outlines the conception, methods, etc. of adjustment computation.

1) Objects and methods of adjustment computation

Adjustment computation is made for the objects of obtaining the most probable values of coordinates at the respective points, confirming the acceptability of measured values and the precision of coordinates, to judge whether or not the target precision is attained, discussing the necessity of renewing practical results, knowing more reliable variations of triangulation survey points, and so on. As mentioned before, the method by observation equation is more advantageous comprehensively than that by condition equation, but has a large difficulty that cannot be solved by perfect freedom. To solve this difficulty, such methods can be considered, as fixing higher order points, fixing one-point one-azimuth, fixing proximate points outside a survey area, Mittermayer's method ($\Sigma(X^2 + Y^2) = \min$), and $\Sigma(X + Y) = 0$ method. Considering the objects of the adjustment computation in the primary control survey, below are discussed the advantages and disadvantages of the methods.

To detect more reliable variations of coordinates, it is necessary to determine a computation area by collecting survey areas, etc., and to recalculate old coordinate values obtained by a different computation method, but in the computation involved in the fieldwork of primary control survey, comparison with practical results in each survey area block is made as practical processing. Therefore, the following discussion is made, recognizing the ambiguity due to them.

It is most important to obtain more reliable variation vectors and to determine the conditions necessary for adjustment computation, not to correct measured values as forced by the restriction of other ambiguous conditions, etc. For example, it is one of great difficulties, to determine the immovable point due to the influence by diastrophism, etc. This is, after all, what is the best connection method between practical results (coordinates) and new results (coordinates)? and Mittermayer's method $\Sigma(X^2 + Y^2) = \min$ and $\Sigma(X + Y) = 0$ method are intended to relate the old network to the new by these respective equations. Triangulation survey points do not vary according to the condition of $\Sigma(X^2 + Y^2) = \min$ or $\Sigma(X + Y) = 0$. This can be well understood, considering the variation due to an earthquake and an artificial local variation. Furthermore, in $\Sigma(X + Y) = 0$, the problem of rotation (lacking in one condition) requires to fix at least one of the remaining directions. The method of fixing proximate points outside a survey area (actually conceivable method is to fix two or three first order triangulation survey points outside a survey area) is to lightly correlate the old network with the new network, by relating the fixed points with observation points on the outside circumference of a survey area by assumed measured values and giving very small weights to them. In this method, the influence of these fixing conditions on variation vectors is complicated, not enabling to define clearly and make meaningful the detection of variation vectors unlike the former two methods.

Now, let's consider the method of fixing one-point one-azimuth. In this method, one point on the outside circumference of a survey area considered to be relatively less variable is fixed, and the azimuth from the point to one of the nearest triangulation survey points in the survey area, considered to be relatively less variable, is fixed (if the farthest triangulation survey point is used in fixing the azimuth, the fixing of the direction complicatedly affect the variation vectors of respective points, etc., and care should be taken in this regard). According to this method, observed values can be adjusted in completely free state, but if there is any error in the assumed azimuth, the error causes variation vectors to be rotated. If there is any error in the assumption of immovable point, it gives a constant error to the variation vectors at the respective points. However, it is significant that the variation vectors obtained by this method are the simplest variation vectors based on the propagation of simple assumed error, compared with it that the variation vectors obtained by the above other methods are complicated by assumption conditions. For this reason, the variation vectors are offered as basic data for the analysis of diastrophism.

Since the rotation by an error in the fixed azimuth and the constant error due to an error of immovable point are contained in the variation vectors of respective points as simple functions of those errors, the quantity can be estimated by the following simple calculation, using the variation vectors of respective points excluding the vectors of abnormal variation points.

$$\upsilon xi = +dx_0+x_i(ds/s)-y_i(d_{a0})-\Delta x(N-O)i$$

$$\upsilon yi = +dy_0+y_i(ds/s)+x_i(d_{a0})-\Delta y(N-O)i \quad \ldots\ldots\ldots\ldots\ldots \quad (6.4)$$

where dx_0, dy_0 ..... Errors to fixed coordinates x_0, y_0 of immovable point

da_0 ..... Error to fixed azimuth a_0

$\Delta x(N-O)i$, $\Delta y(N-O)i$ ..... x, y variations at point i

xi, yi ...Coordinate differences between point i and immovable point

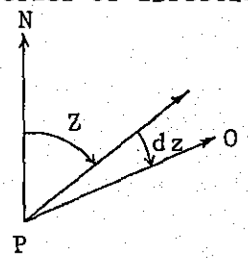
(ds/s) ...Average scale element in a survey area

The average scale element (ds/s) in a survey area obtained in equation (6.4) is a comprehensive value containing (1) difference between the length used for invar wire, etc. and the length by the use of the frequency by light waves of electro-optical distance meter, (2) expansion or contraction like constant error of first order triangulation survey network in a distance of approx. 200km closing from an extended base line length to another extended base line length, (3) average expansion or contraction due to diastrophism, and so on. To discuss the necessity of renewing practical results as one object of adjustment computation, it is planned to prepare data with the variation vectors of respective points corrected (called renewed result vectors), using dx_0, dy_0, da_0 and (ds/s) obtained from equation (6.4).

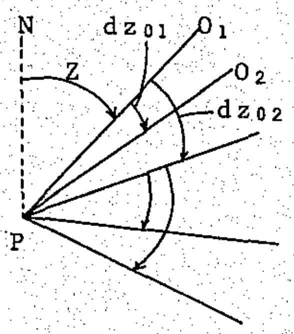
For evaluating the average precision of coordinates of all the points in a survey area, the expression of $m_0\sqrt{\text{trace }(Q)/n}$ is decided to be used, and as already mentioned, Q obtained by Mittermayer's method is used. In sub-contract surveying, it is decided to use the results of network adjustment based on one-point one-azimuth fixing method for the time being. Also with regard to the judgement as to the acceptability of precision of coordinates of respective points, there is similar difference between directly-operated surveying and sub-contract surveying.

Section 3. Orientation Errors

When an observation equation by x-y coordinates is made for each direction, a series of observation directions are arranged with zero direction as 0°0'0", O. A given angle added to zero direction, to rewrite it as an observed bearing of x-y coordinate system is called orientation angle (conventionally called standard angle) Z, and a correcting value for it (containing a pointing error like constant error for zero direction, etc.) is called orientation error dz. Therefore, dz is one of unknown quantities, and is not an accidental error which follows a normal distribution, as dz is contained commonly in the series of direction observation.



Direction angles $\vec{PO}$ are normally distributed around the average direction angle, with standard deviation σ_0. The angle to relate the average direction angle to Z given optionally is dz. Therefore, when direction observation at point P is divided into two groups, different "dz"s should be used for respective groups even for the same $\vec{PO}$ as zero direction. However, it is not



that with standard deviation of direction angles $\vec{PO}$ in group A as σ_{01} and that of direction angles PO_2 in group B as σ_{02}, two average direction angles $\vec{PO}$ are established, but that an average direction angle with only one σ_0 is established. Therefore, it can be said to be practical to consider an average direction angle $\vec{PO}$ with only one average σ_0 from the beginning, for introducing only one dz_0 like median value of dz_{01} and dz_{02}. Thus, it is allowed to unify zero direction, to lessen the number of "dz"s, for calculation. Furthermore, when angle observation is made as in case of primary control survey, respectively different orientation errors are required, since angle observation can be regarded as a kind of orientation observation. The number is one orientation error for each angle (two directions), increasing greatly the number of orientation errors as unknown quantities but since calculation can be made, using an error equation with orientation errors eliminated according to Schreiber's law, it is advantageous that computation volume does not become especially large.

CHAPTER 7. CONCLUSION

This chapter has described the basic conception of the Precise Geodetic Survey Operation Rules, with the hope that survey engineers sufficiently experienced in fourth order triangulation survey operation will understand the survey operation rules well and recognize the importance of various operation steps, studying more widely and deeply the precise geodetic survey. The survey engineers are requested to understand this description, before they master the operation rules and are engaged in the work.

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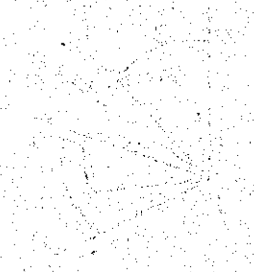

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PART III.

OPERATION RULES FOR PRECISE GEODETIC NETWORK PRIMARY CONTROL POINTS SURVEYS

Section 1. Outline

(Summary)

Article 1. The survey of precise geodetic network primary control points refers to the operation to establish geodetic coordinates for first and second order triangulation points.

2. The primary control point network consists of mono triangles and trilateration should be used as a rule.

However, if this is found to be impossible, in the case of the common side of two adjoining mono triangles, the distance measurement may be replaced by the observation of the two opposite angles.

3. Desirable precision in a horizontal position should be 3cm (mean value of mean square error of each point obtained by adjustment computation based on Trace (Q) = min.).

Section 2. Reconnaissance

(Summary)

Article 2. Reconnaissance is an important operation to determine the operational method and means depending on the topography and other local conditions while confirming the inter-visibility between triangulation points so that coordinates may be obtained with the desirable precision.

(Implementation of reconnaissance)

Article 3. Reconnaissance is to be made as prescribed below:

(1) Planning on the map is to be carried out with minute care on 1/50,000 topographic maps and 1/200,000 geographic maps taking full account of the strength of the triangulation network. The net adjustment plan is to be prepared using 1/200,000 geographic maps to compute $\sqrt{Q}$ showing the relative precision of coordinates of each point to be submitted.

(2) The method of measuring the lengths of surrounding sides is to be prescribed according to the geodetic network establishment plan.

(3) With regard to the observation point to install the electro-optical distance measuring instrument and the theodolite, selection is to be made locally according to the net adjustment plan to prepare the selection map. The side to be measured and the angle of measurement prescribed under the previous section are not to be changed.

(4) The geodetic network is to consist of approximately equilateral mono triangles as a rule and the included angle of less than 25° is to be avoided.

(5) In special cases, according to the geodetic network establishment plan, or because of topographical difficulties or for strengthening of the figure strength, downgrading of the primary control point or upgrading of the secondary control point (third order triangulation point) may be made.

(6) Eccentricity may be made to the extent of $e<s/10$ only when intervisibility is unobtainable with the height of 4m for the instrument, reflecter and the signal lamp.

(7) Broken base line may be used only when two sides of one triangle have to be measured regarding the other side.

(8) In the case of the loss of a triangulation point, it is to be restored at the original position as a rule. If this is found to be difficult, it is to be selected at a point with good intervisibility and suitable for use and conservation.

If it is in a mountain or hill area, it is to be selected on the top.

(9) All intervisible lines are to be entered on the reconnaissance map upon field survey.

(Survey of the present condition of the existing triangulation point)

Article 4. Survey of the present condition of the existing triangulation point is to be implemented as prescribed below.

(1) If the position of the triangulation point seems to be doubtful judging from the site condition and the measured value, a corner of the stone marker is to be excavated with caution to confirm and report on uplift and the base stone.

(2) If the stone marker is tilted, excessively exposed or buried, it is to be restored and reported accordingly.

(3) If the stone marker has been lost or damaged, it is to be reported promptly.

(4) The base stone of the first order triangulation point is not to be touched.

(The net adjustment plan and the description of point, agreement on the erection of the marker, etc.)

Article 5. The net adjustment plan is to be prepared according to the selection map, taking account of $\sqrt{Q}$.

(2) Description regarding the point are to include the location, cadaster, ownership, route and other matters for future information.

(3) If a new marker is to be installed on private land, agreement is to be obtained in writing from the owner or a site record is to be prepared with the approval of the administrator in the case of public land.

(4) A map of observation network is to be submitted using 1/200,000 geographic maps and 1/50,000 topographic maps.

Section 3. Installation of Permanent Markers and Observation Facilities

(Summary)

Article 6. Lost point is to be replaced by a permanent marker together with protective facilities.

2. Facilities necessary for observation are to be installed.

(Installation of permanent markers, etc.)

Article 7. Installation of a triangulation point is to implemented as prescribed below.

(1) Installation is to be carried out firmly according to the standard installation figure based on the Method of presentation of Operation Rules for Precise Geodetic Network Primary Control Points surveys (hereafter referred to as MOP) in such a way that the stone marker and the base stone meet at the center.

(2) In installation, the base stone is to be set horizontally. The face of the stone with the inscription "First order triangulation point" or "Second order...point" is to face south while the top of the stone is kept horizontally.

(3) Length of the stone is to be measured to the cm.

(4) In a special place, with the approval of the supervising official, it may be a underground marker or roof-top point.

(5) The triangulation point is to be provided with a marker.

(Observation facilities)

Article 8. Facilities for the installation of signal targets (observation frame, instrument base and target base) and thermometer are to be installed as prescribed below.

(1) The observation frame is to have a strong frame structure so that observation is easy and that the instrument may be protected from the direct sun and wind for precise observation.

(2) The instrument base is to be constructed strongly without eccentricity at a height of more than 2m above the ground for angle measurement and more than 1.5m for distance measuring. Further, if a tripod is used for distance measuring, supplementary facilities such as leg stakes and step plates are to be provided.

(3) The target base (including tripod) is to be constructed in such a way that there is as little eccentricity as possible and that the height of the signal lamp or the reflector is more than 1.5m above the ground. It may be installed together with the observation frame at an observation point.

(4) Facilities to install the thermometer are to be constructed at a position suitable for distance measuring at a distance of more than 3m from topographical obstacles, vegetation, ground and structures and is to be free from radiant heat.

Section 4. Observation

(Summary)

Article 9. Observation is an operation to measure the distance to the related point and the horizontal angle by setting the electro-optical distance meter and the theodolite at the observation point.

2. Measurement of the distance, observation of the horizontal angle and measurement of temperature, atmospheric pressure, etc. are to be made with minute care so that the error may be kept at a minimum.

(Capacity of instrument)

Article 10. The instrument to be used is to have the capacity described below.

Instrument	Capacity
Electro-optical distance meter	a. Laser beam of more than 1mW in output. b. Random error within ±5mm. c. Modulated frequency over approximately 30MHz with the permitted error as described below. i) Modulated frequency error within $\pm 4 \times 10^{-7}$. ii) Variation to the above within $\pm 5 \times 10^{-7}$.
Theodolite	Special grade transit (based on the standard capacity of survey instrument and equipment prescribed by the Geographical Survey Institute).
Thermometer	Minimum graduation value 0.2°C (thermistor type of the grade prescribed by the Geographical Survey Institute or above).
Barometer	Minimum graduation value 1mmHg or 1mb (Pauling, portable aneroid and regional barometer of the grade prescribed or above).
Level	Second-class level and invar rod (based on the standard capacity of survey instrument and equipment prescribed by the Geographical Survey Institute).
Power supply	Input: AC100V ± 10%. Output: DC12.5V ± 0.5V. Output current 10A. Power fluctuation and load fluctuation: 10mV or above.

2. Type of instrument, number and name are to be entered on the operational plan.

(Inspection of instrument and equipment)

Article 11. Electro-optical distance meter, theodolite, steel measuring tape, thermometer and barometer are to be inspected before and after the operation and also during the operation as necessary.

2. The weather correction formula is to be given on the operational plan.

(Inplementation of observation)

Article 12. Distance measuring is to be carried out as prescribed below.

(1) When the electro-optical distance meter (hereafter referred to as EDM) of the direct reading type, 10 readings are to form one set; in the case of EDM of the distance computation type, measurements by all fluctuating frequencies as one set.

(2) Distance measuring is to be conducted outside the period from 10:00 to 14:00 hours.

However, it may be conducted during the period if it is cloudy (cloud cover 10) or windy (velocity over 6m/sec with dust rising, paper flying, twigs moving and considerable surf line).

(3) Measurement interval of more than one hour between sets of measurements. However, it may be reduced to over 20 min between sunset and sunrise.

(4) Meteorological observation is to be carried out at the instrument point and the reflecting point at an interval of 20 min starting at 20 min prior to the commencement of distance measuring and ending with the observation following the end of the distance measuring operation.

Weather conditions during the measurement are to be entered.

(5) Temperature is to be measured simultaneously at both the instrument point and the reflecting point.

(6) As for barometer, the measurement barometer and the standard barometer kept at the living quarters are to be provided, which are to be compared at each measuring point.

(7) Pressure measurement is to be made together with temperature measurement at both the instrument point and the reflecting point. However, if the differential in elevation between

the instrument point and the reflecting point is less than 1,000m, the pressure at the latter may be obtained by the rate of diminution.

(8) Humidity measurement is not to be made as a rule.

(9) Cover is to be used to cut the sun and the wind.

(10) For combination of sets, measurements on days and times under different meteorological conditions are desirable.

(11) Tripod may be used.

(12) In addition to the above, instructions for each instrument are to be followed.

2. Observation of the horizontal angle is to be conducted as prescribed below.

(1) Observation is to be by the angle method without conditions or by the method of direction.

(2) One observation with a telescope is to be made in both the forward and backward positions to form one-pair observation. For each pair-observation the hroizontal graduation is to be set at a prescribed position and three pair-observations are to form one set.

(3) The time of observation is prescribed under the previous section (2) and (3).

(4) For the combination of sets, observation on days and times under different meteorological conditions is desirable.

(5) The target is to be a signal lamp or a reflector.

(6) Cover is to be used in observation to cut the sun and the wind.

(7) Stable tripod may be used for horizontal angle observation at roof-top.

3. Elevation of a restored point, broken line point or elevation inspection point is to be obtained from a primary point, nearest triangulation point or bench mark by vertical angle observation or leveling as prescribed below.

(1) In vertical angle observation, four observations in both forward and backward position of the telescope are to form one-pair observation.

(2) Height of a signal target or instrument is to be measured to the cm.

(3) Cover is to be used to cut the sun and the wind.

(4) Leveling is to be conducted in both ways.

(5) The target is to be a signal lamp or a reflector.

(Measurement of the eccentricity element)

Article 13. Measurement of the eccentric distance is to be made directly with a ruler, steel measuring tape or EDM.

2. Measurement of the eccentric angle is to be made with a protractor, Sehnen Table or theodolite.

3. Vertical angle observation for the computation of the relative height between the face of the stone marker and the eccentric point is to be made in the forward and backward directions (base point and eccentric point). However, if the eccentricity is less than 100m, it may be carried out through two observations at either the base point or the eccentric point at different heights of the instrument.

(Re-observation)

Article 14. If the results of observation are in excess of the acceptable range, observation is to be repeated.

Section 5. Computation and Arrangement

(Summary)

Article 15. Plane rectangular coordinate, latitude and longitude, movement vector, etc. of a triangulation point are to be computed on the basis of observed values by using the unified formula to the digit as shown below.

Plane rectangular coordinate	Latitude and longitude	Angle	Length of side
mm	0."0001	0."01	mm

2. Data and records are to be classified as observation notes, observation records, computation notes, description of point, data table, control point network, observation network, etc.

(On-site computation)

Article 16. On-site computation is to be carried out as prescribed below.

(1) Length of the side computed by two-sided inclined angle is to be checked by two side differentials.

(2) Center angle of the centered polygon is to be checked.

(3) Height computation is to be made and checked by appropriate methods.

(Computation)

Article 17. Computation is to be made according to the instructions for each observation point as prescribed below.

(1) Adjustment computation is to be XY or BL network adjustment computation with one direction fixed for one point.

(2) In the network adjustment computation, practical data are to be tentative fixed values. Further, the weight of one angle (one direction) is to be 1 to determine the weight of the distance.

(3) Computed value under (1) above is to be used for the computation of the vector of renewed data.

(4) For network adjustment computation and computation of latitude and longitude, electronic computer is to be used, employing an approved program.

(5) For lost points, points with metal marks and those requiring renewed data, practical data are to be computed.

2. Extensive adjustment computation is to be by BL network adjustment computation.

(Arrangement)

Article 18. Arrangement of observation notes, observation records, computation notes, description of point, data table, control point network map and observation network map are to be carried out as prescribed under MOP.

PART IV.

OPERATION RULES FOR CONTROL POINT SURVEYS

CHAPTER 1. CONTROL POINT SURVEY BY TRAVERSING

Section 1. Outline

(Summary)

Article 1. Control point survey by traversing is a survey operation to determine the geographical latitude and longitude, plane rectangular coordinate value and the height from the mean sea level in respect of a fourth order triangulation point by traversing on the basis of the existing first, second, third and fourth order triangulation points (hereafter referred to as the Known points).

2. Density of point allocation is to be 1 point per $2km^2$.

Section 2. Reconnaissance

(Summary)

Article 2. Reconnaissance refers to a series of operations to select on site the position of a new fourth order triangulation point (hereafter referred to as the New point) according to the net adjustment plan formulated by taking account of the allocation density on topographic maps and to decide on the method and means of operation according to the topography and other site conditions.

(Implementation of reconnaissance)

Article 3. Attempts are to be made to distribute new points evenly within the survey area and select favorable positions from the viewpoints of intervisibility, use and conservation.

2. The new point is to have a prefix, a selection number and a name.

3. The traverse network is to link known and new points and is to be in such a shape that error of closure may be checked only with observed values.

4. Triangulation may also be employed depending on local conditions.

(Survey of present condition of known points)

Article 4. Survey of present condition of known points is to be conducted as prescribed below.

(1) The level of the top of the stone marker is to be inspected, and if abnormality is noticed, a corner of the triangulation

point is excavated with caution to inspect the base stone and measure the length of the stone to the mm.

(2) If there is a discrepancy between the stone marker and the base stone at the center, tilting of the stone, excessive exposure or hiding, it is to be restored and any adjustment made in respect of the exposure or hiding is to be reported in the operational report or the source of survey data correction.

(3) If the stone has been lost or damaged, the report on abnormality of the control point is to be submitted promptly.

(4) The base stone of first order triangulation points is not to be touched.

(Net adjustment map, description of point and agreement on the erection of the marker, etc.)

Article 5. The net adjustment map is to be prepared on the basis of the reconnaissance map to be submitted accompanied by the reconnaissance map immediately after the completion of reconnaissance for selection.

2. Descriptions of the point are to include the location, cadaster, ownership or administrator, route, detailed drawing of the site and other matters for future information.

3. If the new point is to be installed on private land, agreement on the erection of the mark in writing is to be obtained from the owner; if it is on public land, the survey mark site record is to be prepared with the approval of the administrator.

Section 3. Installation of Stone Marker and Temporary Marks

(Summary)

Article 6. Permanent marker (fourth order triangulation point stone marker or metal marker; hereafter referred to as the Stone marker, etc. is to be installed at the selected position together with protective facilities.

2. If necessary, temporary marks (hereafter referred to as the Signal) are to be provided at known and new points.

(Installation of stone marker, etc.)

Article 7. Stone markers, etc. are to be installed as prescribed below.

(1) Installation is to be carried out firmly according to the standard installation plan and the stone marker and the base stone are to meet at the center.

(2) Installation is to be in such a way that the top of the stone marker and the base stone may be horizontal and the face of the stone marker with the inscription of Fourth order triangulation point may face south. Further, the metal marker is to be installed so that the inscribed letters can be read from the South side.

(3) The length of the stone marker is to be measured to the mm.

(Installation of signals)

Article 8. Attempts are to be made to place the center of the signal on that of the stone marker with the mandrel kept vertical and the lower side of the panel horizontal.

(Photographic control)

Article 9. Installation of the stone marker and others and that of the target signal are to be photographed.

Section 4. Observation

(Summary)

Article 10. In order to determine the position of a new point, the electro-optical distance meter, reflector, theodolite, signal lamp and the target board are to be arranged at necessary survey points to measure the distance between related points, horizontal angle and the rectangular angle. If necessary, leveling is to be conducted.

(Capacity of instruments)

Article 11. Instruments are to be of the capacity given below or above and are to be described in the operational plan.

Division	Capacity (nominal)	Remark
Theodolite	First grade transit based on the standard of survey instrument and equipment prescribed by the Geographical Survey Institute.	
Electro-optical distance meter	Within ± 20mm.	Range of over 2km.
Radio distance meter	Within ± 20mm + D/300,000.	Frequency 10GHz
Level	Bubble tube sensitivity over 40 sec.	Equivalent to the wooden folding leveling rod or over.

D: Distance to be measured.

Radio distance meter is not to be used with out an instruction.

(Inspection of instrument and equipment)

Article 12. Electro-optical distance meter, theodolite and steel measuring tape are to be used after inspection.

(Implementation of observation)

Article 13. Minute care is to be exercised in distance measuring, horizontal angle measurement and rectangular angle measurement to reduce the error as prescribed below.

(1) Meteorological observation (temperature and pressure) accompanying distance measuring is to be conducted at the site of the theodolite. Thermometer is to be suspended in the air at a place which is least affected by radiant heat from the topography and ground objects and protected from the direct sun.

(2) Measurement of the horizontal angle is to be by the direction observation method.

(3) For measuring the horizontal angle, observation with a telescope is to be made in both left and right positions to form one-pair observation, and the horizontal graduation is to be set at a prescribed position for each pair-observation.

(4) For measuring the rectangular angle, observation with a telescope is to be made in both left and right position to form one-pair observation which is to be made for each direction.

(5) For measuring the height in respect of the instrument, target lamp, reflector and target signal (or target board) is to be made to the cm.

(Measurement of the eccentricity element)

Article 14. The distance and the angle of eccentricity necessary for the computation of eccentric correction in the case of discrepancy between the instrument, stone marker and target in respect of the center are called elements of eccentricity which are to be measured with a prescribed precision.

(Re-observation)

Article 15. In the measurement of distance, horizontal angle and rectangular angle, if the data obtained exceed the accepted range, measurement is to be repeated.

Section 5. Computation and Arrangement

(Summary)

Article 16. Plane rectangular coordinate, latitude and longitude, elevation and related corrective computations are to be made according to the unified formula on the basis of observed values.

2. According to the results of observation and computation, observation notes, observation records, computation notes, description of point, data table, control point network map, etc. are to be prepared.

(Implementation of computation)

Article 17. Computation is to be made according to the instructions for each item as prescribed below.

2. In on-site computation, by the closing error of the traverse, angle measurement, plane rectangular coordinate and relative height are to be examined and estimates are to be obtained for precise computation.

3. Precise computation is to be made as prescribed below.

(1) Horizontal position is to be computed by XY network adjustment computation employing plane rectangular coordinates.

(2) Latitude and longitude are to be computed from the plane rectangular coordinate values obtained by XY network adjustment computation.

(3) Elevation is to be computed by the elevation adjustment computation.

(4) Electronic computer is to be used for XY network adjustment computation, computation of latitude and longitude, and elevation adjustment computation, using an inspected and registered program.

(5) As for plane rectangular coordinates of a new point within 2km from the boundary of the coordinate system, in addition to coordinate values by the coordinate system concerned, coordinate values of neighboring coordinate systems are to be computed.

(6) If an existing point shows abnormality, it is to be regarded as a new point and computation is to be made accordingly to renew the data.

(Arrangement)

Article 18. Arrangement of observation notes, observation records, computation notes, description of point, data table, control point network map, etc. is to be carried out by prescribed methods.

CHAPTER 2. SURVEY OF CONTROL POINTS BY TRIANGULATION

Section 1. Outline

(Summary)

Article 19. Rules prescribed under Article 1 to be applied. However, "polygon" in Article 1 is to be read "triangular" in this case.

Section 2. Reconnaissance

(Summary)

Article 20. Rules prescribed under Article 2 to be applied.

(Implementation of reconnaissance)

Article 21. Rules prescribed under Article 3, Paragraphs 1 and 2 are to be applied.

2. The triangulation network is to consist of triangles formed by both existing and new points and points are to be selected in such a way that each triangle is close to a regular triangle.

3. Points are to be selected in such a way that each triangle may be formed by only observed values.

However, if this is not possible due to the topography and others, one angle may be a supplement.

4. The traversing survey method may also be employed depending on the topography.

(Survey of present condition of known points), (Adjustment map, description of point and the agreement on the erection of the mark)

Article 22. Rules prescribed under Articles 4 and 5 are to be applied.

Section 3. Installation of stone markers and others and temporary marks

(Summary), (Installation of stone markers and others), (Provision of signals)

Article 23. Rules prescribed under Articles 6-9 are to be applied.

Section 4. Observation

(Summary), (Capacity of instrument), (Inspection of instrument and equipment), (Implementation of observation), (Measurement of eccentric elements), (Reobservation)

Article 24. Rules prescribed under Articles 10-15 are to be applied.

Section 5. Computation and Arrangement

(Summary), (Implementation of computation), (Arrangement)

Article 25. Rules prescribed under Articles 16-18 are to be applied. However, in this case, "polygon" in Article 17 is to be read "triangular".

PART V.

OPERATION RULES FOR LEVELING--BASIC SURVEY

CHAPTER 1. GENERAL RULES

Section 1. General Description

(Summary)

Article 1. These operation rules describe methods, regulations and so forth for 1st and 2nd order leveling and sea (river)-crossing leveling to be performed directly by the Geographical Survey Institute or by outside subcontractors.

2. The 1st order leveling described in these operation rules is the operation, as defined here, by which elevations of bench marks to be newly established or already in sites along national highways or main local highways in many areas are determined with prescribed accuracy by using level and rod.

3. The 2nd order leveling described in these operation rules is the operation, as defined here, by which elevations of bench marks to be newly established or already in sites along national highways or main local highways are determined with prescribed accuracy by using level and rod in order to supplement the 1st order leveling network consisting of 1st order bench marks.

4. The sea (river)-crossing leveling described in these operation rules is, as defined here, the operation to determine the difference in elevation between two points at both banks separated by a river or strait (hereafter referred to as "crossing point"), with prescribed accuracy by using level or theodolite and rod.

(Leveling route and leveling net)

Article 2. The route that sequentially connects the bench marks is called a leveling route.

2. The 1st order leveling route must have Japan standard datum of leveling or 1st order bench marks as starting point and ending point and, as a rule, must form a circuit.

3. The length of the 1st leveling route forming a circuit must be 400km as a standard.

4. The 2nd order leveling route must be formed in such a manner that it is connected to 1st order bench marks or existing 2nd order bench marks.

5. The length of a 2nd order leveling route shall be less than 200km approximately.

6. The leveling net is the net-shaped route consisting of more than two connected circuits of leveling routes.

(Classification and density of bench marks)

Article 3. The bench marks are classified into fundamental bench mark, sub-fundamental bench mark, 1st order bench mark, 2nd order bench mark, junction bench mark, tidal bench mark and sea-crossing bench mark.

2. The density of the bench marks shown in the table below should be considered as standard.

Item	Fundamental bench mark	Sub-fundamental bench mark	1st & 2nd order bench marks
Density	Interval between bench marks is 100∿150km	Interval between bench marks is 7 ∿ 10km	Interval between bench marks is 2km except where accompanied with kilo-post mark.

Item	Junction bench mark	Tidal bench mark	Sea-crossing bench mark
Density	To be provided at junction point of 1st order leveling routes	To be provided in adjacent with tide station	To be provided at sea(river)-crossing point

(Bench mark numbers)

Article 4. Serial numbers shall be assigned for each classification of fundamental bench marks, sub-fundamental bench marks, 1st & 2nd bench marks and tidal bench marks, serial numbers for each district for sea-crossing bench marks, and serial numbers of 1st order bench marks or serial numbers of junction bench marks for junction bench marks. In addition, words such as "fundamental", "sub-fundamental", "junction", "tidal" or "crossing" shall be attached before each number of bench mark except for 1st and 2nd bench marks.

2. For 1st order bench mark or 2nd order bench mark accompanied with kilo-post mark (this will be called "kilo-post bench mark" hereinafter), each number shall be accompanied with 3 upper digits as road number and 3 lower digits as distance from road starting point in kilometers with fractions discarded. For 2nd order bench mark to be installed in pre-

fectural roads, prefectural number (refer to accompanied table) should be placed over the road number.

(Restriction on observation)

Article 5. Discrepancy between fore and back leveling, error of closure when a circuit is formed or a connection is made to an existing point, and root mean square error for whole observation in sea(river)-crossing leveling shall have the values within the restriction shown in the table below.

Classification	1st order leveling	2nd order leveling	Sea(River)-crossing leveling	
			Connecting 1st order leveling route	Connecting 2nd order leveling route
Discrepancy between fore and back leveling	$2.5\sqrt{S}$ mm	$5.0\sqrt{S}$ mm		
Error of closure	$2.0\sqrt{S}$ mm	$5.0\sqrt{S}$ mm		
Root mean square error			$\pm 2.0\sqrt{S}$ mm	$\pm 2.5\sqrt{S}$ mm

S means distance of observation in km.

2. The restriction for the difference between the relative height by check leveling and the previously observed relative height shall be determined using discrepancy between fore and back leveling described in previous paragraph.

3. The elevations of bench marks newly installed or reinstalled during leveling shall be determined by the observed value based on adjacent bench mark, and the error of closure when connected shall be within $15\text{mm}\sqrt{S}$. (S is observation distance in km).

(Multiple use of bench mark)

Article 6. The bench mark can be also used as a control point for other purposes.

CHAPTER 2. 1ST AND 2ND LEVELING

Section 1. Preparation for operation

(Summary)

Article 7. Persons who engage in survey must prepare for the operation in detail before commencement for performing the survey operation smoothly and effectively, and also for improving accuracy of survey.

Section 2. Reconnaissance

(Summary)

Article 8. The reconnaissance the operation for selecting the most desirable locations with thorough investigation for properness of bench mark locations planned on map in view of conservation and utilization of bench marks as survey markers.

(Execution of field inspection and reconnaissance)

Article 9. Location of bench marks and leveling routes planned on the map shall be inspected at field and then their properness shall be determined. Particularly, the projects that involve alteration, improvement or new construction of the roads shall be referred to government authorities in order to investigate and select the proper locations which will not require any reinstallation after survey is completed.

2. Necessity of new installation of bench marks and revision work should be determined through investigation of conditions of existing bench marks.

3. The location of bench marks to be selected shall be such that the ground is stable and appropriate for conserving the bench marks as survey markers.

4. The selction of location of the fundamental bench marks shall be performed in accordance with procedure for installing Fundamental Bench Marks prescribed in other paragraphs.

5. The location of the sub-fundamental bench marks, when selected, shall have solid ground and be appropriate for conserving them as survey markers.

6. The kilo-post bench mark shall be located adjacent to 100-meter sign post marked 5 on it. However, if its location is inappropriate, the kilo-post bench mark can be located to nearest 100-meter sign post.

7. When using the site selected for survey markers, it is required to obtain approval by owner or person in charge of the site.

(Map of selected points, description of point and preparing written approval)

Article 10. When bench mark location is selected, the location shall be indicated on 1/50,000 or 1/25,000 scale topographic map and necessary notes should be accompanied. This map is called "map of selected points". (This map is made only for new installation.)

2. In description of point, name of topographic map (1/200,000 or 1/50,000 scale) including the name, number and address of bench mark, classification of land, owner of land and way to land, detailed vicinity map and other information useful for future operation should be indicated.

3. For all 1st order bench marks used for observation in normal leveling, the description of point shall be made.

4. A written approval for installing bench mark shall be made if land of the bench mark is privately owned, and station site investigation report shall be made if land is owned by government. The latter, however, is not needed for installing kilo-post bench mark.

Section 3. Installation of Permanent Markers

(Summary)

Article 11. In installing the permanent marker, prescribed permanent marker shall be buried at a location selected by the point-selecting operation, and protecting means for the marker shall be also provided. Installation shall be done either with normal or underground burying method.

(Installation)

Article 12. Installation shall be performed securely in accordance with standard installation details.

2. Installation of fundamental bench mark shall be made in accordance with the Procedure for Installing Fundamental Bench Marks prescribed in other paragraphs.

Section 4. Observation

(Summary)

Article 13. Observation is to measure the height difference between two rod points by reading rods with a level set up at the center between them, and by repeating this procedure successively to determine the height difference in elevations between bench marks.

2. Since accuracy of observation will directly affect the elevations resulted, extra precautions will be required in handling instrument and selecting observation method.

(Observation instruments)

Article 14. Instruments to be used shall have the capacity better than those shown in table below and they shall be clearly designated in operation plan.

Item	1st order leveling	2nd order leveling
Level	1st class	2nd class
Rod	1st class	1st class

However, these instruments must conform to the survey instrument capacity standard prepared by the Geographical Survey Institute.

(Preparation for observation)

Article 15. The level and rod shall be inspected and adjusted for the items shown in the following paragraphs before starting the observation and during the operation whenever appropriate.

(1) Inspection and adjustment of parallelism between the collimation axis and axis of main level tube and circular level.

(2) Inspection and adjustment of circular level for automatic level.

(3) Inspection and adjustment of bubble level attached to rod.

(Execution of observation)

Article 16. Observation team shall consist of one observer and three assistants as standard.

2. Describing in original draft result table, revision of

description and computations shall be made in conformity with the method of presentation.

3. Observation shall consist of fore and back leveling.

4. Two rods shall be used as a set, one of them being numbered with No. I and the other with No. II. They shall be used in the reverse order for the back leveling. Number of survey stations during one-direction leveling between bench marks must be an even number.

5. Distance between level and backsight rod shall be equal to the distance between level and foresight rod, and the level shall be set up on a straight line between two rods as possible.

6. Length of sight and unit of reading shall satisfy the requirements shown in table below.

Item	1st order leveling	2nd order leveling
Length of sight	50m maximum	60m maximum
Unit of reading	0.1mm	1 mm

7. Two specific legs of level shall be always in parallel with line of sight and set up alternatively at right side and then at left side of forward direction at every survey station.

8. In the case of releveling due to discrepancies exceeding tolerable limit between fore and back reveling between bench marks, it shall not be allowed to accept only the data of the same direction.

9. Starting point and ending point of the leveling route shall be determined after examining the records of movements in the past, and their relation to existing adjacent bench marks shall be checked before starting observation in order to check possible abnormality that may exist in these points. If abnormal condition is found, it is required to extend checking route.

Section 5. Computation, Arrangement, Review & Inspection

(Summary)

Article 17. Elevation of bench mark shall be obtained from observed data after rod correction, normal orthometric correction and, if necessary, correction for movement due to difference of observation time and after performing adjustment computations for leveling net based upon these corrections.

2. Results of observation and computations shall be separately filed generally into result table, table for correction due to movements and leveling adjustment computation table.

(Descriptions and computations in observation fieldbook, result table, table for correction due to movement and leveling adjustment computation table)

Article 18. Descriptions and computations for the observation fieldbook shall be made in accordance with Method of Presentation.

2. Descriptions and computations for the result table shall be made in accordance with Method of Presentation.

3. For the survey in the area where considerable ground movement is expected, computations for correction due to movement shall be made.

4. Adjustment computations shall be performed with weights inversely proportional to the lengths of routes, using observation equation or condition equation.

(Arrangement)

Article 19. The arrangement for observation fieldbook, adjustment computation table, description of bench mark, result table, control network diagram, precision control table, station site investigation report or written approval for installing bench marks, and notice of location of installed station markers shall be made in accordance with the Method of Presentation.

(Review and inspection)

Article 20. Members of survey team shall submit various documents related to the operation to chief of section for review and inspection.

(Final results)

Article 21. Final results and data shall consist of original draft result table, result table, adjustment computation table description of point, precision control table, written approval for installing bench marks, station site investigation report and notice of location of installed station markers, new & old details, and other materials needed.

CHAPTER 3. SEA-CROSSING LEVELING

Section 1. General Description

(Classification)

Article 22. Sea-crossing leveling shall be conducted in accordance with the table shown below as standard, depending upon the distance to be observed.

Classification	Method of survey	Standard distance to be observed
Reciprocal method (5m method)	Attach a mark board on rod, align it with line of sight of level by moving the board up and down, read scale of rod, and find difference in elevations.	Up to about 300m
Tilting screw method	Attach to mark boards in rod, and observe the interval between the boards by using scales of tilting screw of level or, instead of using level, observe elevation angles of both upper and lower boards by theodolite for computing difference of elevation. However, if level is to be used, two levels are required in this method.	Up to about 5km
Theodolite method (trigonometry)	Instead of using rod, install signal target mark and observe zenith distance using theodolite for computing the difference of elevation. Distance between crossing points should be obtained using distance meter.	

Section 2. Reconnaissance

(Summary)

Article 23. In the reconnaissance, crossing points planned on map are checked whether they are appropriate for actual survey on the basis of topographic conditions on site and the most suited location must be selected.

(Execution of reconnaissance)

Article 24. The following items should be taken into consideration when performing the reconnaissance for surveying by means of reciprocal method or tilting screw method.

(1) When using level, the relative height between survey stations on both banks is to be less than 1 meter.

(2) The both survey stations are to be as close as possible to the banks, and the topographic features at both stations are to be similar to each other with firm ground condition.

(3) Line of sight is to be at least three meters higher above the surface of the water.

2. When performing the point reconnaissance by using theodolite method for the observation distance exceeding 5km, the above items must be taken into consideration in addition to the items listed below.

(1) Survey stations to be selected at both banks shall be as high as possible from the surface of the water level, and part of the line of sight passing on land shall be kept to a minimum.

(2) When observation distance exceeds nearly 10km, two survey stations with a difference of about 50 meters between relative heights at both banks shall be selected.

Section 3. Installation of Permanent Markers

(Installation)

Article 25. Installation of the permanent markers shall be performed in accordance with Section 3, Chapter 2.

Section 4. Observation

(Summary)

Article 26. Observation is defined here as the operation to be performed in accordance with survey classification shown in Art. 21 for determining difference in elevations between crossing points at both banks.

(Instruments to be used)

Article 27. Instruments shall have the capacity better than those shown in the following table:

Classification	Level	Theodolite	Distance meter	Rod	Remarks
1st order leveling route	1st class	Special class	±30mm+D/300,000	1st class	D means observation distance.
2nd order leveling route	2nd class	1st class	" "	"	

However, these instruments must be recognized for the conformity with surveying instrument capacity standards prepared by Geographical Survey Institute.

(Inspection and adjustment of instruments)

Article 28. Level, theodolite, distance meter, etc. shall be inspected and adjusted for their functions before commencement of operation or during operation whenever possible, and only normally operative instruments shall be used.

(Preparation for observation)

Article 29. The following considerations will be necessary for preparing observation:

(1) Leg stake and footboard shall be installed for securely setting up the instrument.

(2) When surveying with tilting screw method, rod shall be fixed by support frame about 5 meters away from observation point.

(3) Width of white line on mark board shall be so adjusted that sighting can be easily performed depending upon the observation distance and instruments.

(4) On each rod on both banks, one mark board shall be attached for reciprocal method and two mark boards for tilting screw method.

(5) When observation distance exceeds about 5km, a signal target shall be installed as mark.

(Execution of observation)

Article 30. Observation shall be performed in the following manner:

(1) Communication between both banks shall be well maintained during observation, and operation shall be divided into morning and afternoon portions.

(2) Number of observations shall be determined in accordance with Management Standards.

(3) The observation shall be divided into several portions in accordance with Management Standards, depending upon observation distance, each portion being performed in a particular day designated.

2. Observation by reciprocal method shall be performed in the following manner:

(1) Observation shall consist of 5 pairs of operation as a set. Each pair of operation comprizes this-side rod observation plus opposite-side rod observation and opposite-side rod observation plus this-side rod observation. Number of sets of observation prescribed in Management Standards shall be performed.

(2) In opposite-side observation, an assistant shall move the mark board attached to the rod up and down as directed by observer until it align with the line of sight, then he should read rod scale in millimeter when alignment is made and record the results.

3. Observation by tilting screw method (using two levels) shall be performed in the following manner:

(1) Install base board horizontally on tripod, and set two levels on the board.

(2) After adjusting sight of two levels to opposite-side mark board, turn both levels until both objective lens of levels face each other.

(3) By adjusting main bubble levels, make line of sight of each level horizontal.

(4) Make an adjustment until cross hairs of one level coincides with the image of cross hair at other level.

(5) Make lines of sight of two levels horizontal on the base board by using main bubble levels, and read the scale of this-side rod.

(6) Aim two levels at opposite-side mark board, and make the levels horizontal by using the main bubble levels.

(7) Give a sign to start simultaneous observation to persons working at opposite bank.

(8) Sight lower mark board at opposite bank through the left level by turning tilting screw and read scale (m_1) of the screw.

(9) Properly match the image of bubble and read the scale (m_0) of the tilting screw.

(10) Turn the tilting screw to sight the upper mark board and read the scale (m_2) of the tilting screw.

(11) Repeat the observation described in (8), (9) and (10) and a pair of observation will be completed.

(12) Then, using the same level, perform additional four pairs of observation.

(13) Using the other level, repeat the observation described from (8) to (12).

(14) Aim both levels at this-side rod, make the lines of sight of levels horizontal by using the main bubble levels, and read the rod scale in a manner described in Paragraph (5).

(15) Perform observation described from (5) to (14) at both banks to complete 1/2 set of observation.

(16) One set of observation should be completed by performing 1/2 set before 13:00 and remaining 1/2 set after 13:00.

4. When using theodolite instead of level, provide one theodolite at each bank and perform the zenith distance observation for both upper and lower mark boards in accordance with steps in Paragraphs (1) to (16) shown above.

5. Observation by the theodolite method shall be performed in the following manner:

(1) Measure instrument height and mark board height.

(2) Signal the sign to start observation at both banks and perform simultaneous observation.

(3) Complete one pair of observation by performing sighting once at each of left and right telescopes. A total of five pairs of observation is required.

(4) Whe upper and lower sighting points (two) are provided, one pair of observation is needed for each of upper and lower mark, and five pairs of observation are required in total.

(5) Signal the sign of end of observation to observer at opposite bank

(6) Steps (1) to (5) form 1/2 set of observation. And one set of observation should be performed and completed in accordance with Paragraphs (5) to (16) described above.

Section 5. Computations, Arrangement & Inspection

(Computations)

Article 31. Computations for difference of elevations of crossing points shall be done by general computations based upon the observed data in accordance with the Method of Presentation.

(Execution of computations, arrangement and inspection)

Article 32. Computations, arrangement and inspection shall be carried out in accordance with provisions shown in Section 5, Chapter 2.

PART VI.

OPERATION RULES FOR LEVELING — PUBLIC SURVEY

Section 1. Summary

(Summary)

Article 1. Leveling is the operation, as defined herewith, in which difference in elevation (relative height) between bench marks shall be obtained and elevations of newly installed bench marks including previously observed bench marks shall be determined basing upon the existing bench marks by using levels and leveling rods.

2. The leveling can be classified, depending upon the accuracy, into 1st order leveling, 2nd order leveling, 3rd order leveling, 4th order leveling and simplified leveling.

Section 2. Planning

(Preparation of operation plan)

Article 2. Bench mark arrangement plan shall be made on map and operation plan shall be prepared considering instruments to be used, period of operation, members of team and so forth.

2. When making plan for releveling (including survey for ground subsidence), it is required to investigate process of movement of bench marks in the past on the graph and to determine the starting point and ending point of the observation.

3. The leveling route shall be planned, as a rule, to start from an existing bench mark and to close at another existing bench mark.

Section 3. Reconnaissance

(Summary)

Article 3. Reconnaissance means the operation for selecting the most desirable locations for bench marks on site after fully checking whether the bench mark locations planned on the map are appropriate in view of conservation and utilization of the bench marks are survey markers.

(Field reconaissance survey)

Article 4. Locations of bench marks and leveling route planned on the map shall be investigated on the spots and their appropriateness shall be determined.

2. The field reconaissance survey of existing bench marks shall be performed in the following manner:

(1) Verify whether top of the bench mark is horizontal and check possible abnormal condition.

(2) Abnormal conditions such as loss of bench marks, and slanted, excessively exposed or broken bench marks shall be, if found, immediately reported in writing to planning authorities.

(Execution of reconnaisance)

Article 5. Location of bench mark must be selected at such a place where the ground is stable and suited for the conservation of the bench mark as survey marker.

2. Particularly, the projects that involve alteration, improvement or new construction of the roads shall be referred to government authorities in order to investigate and select the proper location which will not require any installation afterward.

3. When using the site selected for installation of bench mark, it is required to obtain approval by owner or person in charge of the site.

(Map of reconnaissance, description of points, and preparing written approval)

Article 6. When bench mark locations are selected, map showing the locations shall be provided.

2. In describing the bench marks, address of bench marks, classification of land, owner of land, way to land, detailed vicinity map, and other information useful for future operation should be indicated.

3. When site for installing the bench marks (permanent markers) is privately owned, owner's approval shall be obtained and written approval for installing bench mark shall be prepared. For government owned land, approval by person in change of the land shall be obtained and station site investigation report shall be prepared.

4. When permanent markers or the like are installed, the person in charge of planning authority shall immediately report it to the head of Geographical Survey Institute and also to prefectural governor by using designated forms.

Section 4. Installation of Stone Markers

(Summary)

Article 7. Stone markers shall be installed by the predetermined method at the location selected by the operation of point selection, and their protective means shall be also provided.

(Execution of installation)

Article 8. Installation shall be conducted securely in accordance with standard installation detail shown in Method of Presentation.

2. When performing the installation in heavy traffic area, traffic safety precautions such as posting sign of "in operation" shall be taken.

Section 5. Observation

(Summary)

Article 9. Observation means the finding of difference of elevations between two points after reading the scales of rods dtanding at these two points by means of a level set up at the center between them.

(Capacity of instruments)

Article 10. Instruments to be used shall have the capacity better than those shown in the table below, and they must be indicated on the operation plan.

(1) Capacity of instruments used

Classification	Sensitivity of level	Remarks
1st class level	10"/20mm (coincidence type)	With plane parallel plate and reading accuracy up to 0.1mm (0.01mm by eye estimation).
2nd class level	20"/2mm (coincidence type)	With plane parallel plate an accessory and accuracy up to 0.5mm (0.1mm by eye estimation).
3rd class level	40"/2mm	1mm by eye estimation

(2) Instruments used can be classified depending upon the survey accuracy into those shown in the table below.

Classification	1st class leveling	2nd class leveling	3rd class leveling	4th class leveling	Simplified leveling
Level	1st class	2nd class	3rd class	3rd class	3rd class
Rod	1st class	1st class	2nd class	2nd class	Telescopic rod

(Inspection of instruments)

Article 11. Levels and rods shall be inspected and adjusted by checking functions and by measuring.

(Execution of observation)

Article 12. Since accuracy of observation directly affect the results, extra precautions shall be necessary when handling instruments and selecting observation method.

2. Description of original draft result table, revision of described items and computations shall be performed in accordance with Method of Presentation.

3. Observation shall be performed by fore and back leveling

4. Two rods shall be used as a set, one of them being numbered with No. I and the other with No. II. They shall be used in the reverse order for the back leveling. Number of survey stations during one-direction leveling between bench marks must be an even number.

5. Distance between level and backsight rod shall be equal to the distance between level and foresight rod, and the level shall be set up on a straight line between two rods as much as possible. (This distance will be called "length of sight" hereinafter.)

6. Length of sight and unit of reading shall be as indicated in the table below.

Classifi-cation	1st class leveling	2nd class leveling	3rd class leveling	4th class leveling	Simplified leveling
Length of sight	Maximum 50m	Maximum 60m	Maximum 70m	Maximum 70m	Maximum 80m
Unit of reading	0.1mm	1mm	1mm	1mm	1mm

7. Two specific legs of level shall be always in parallel with line of sight and set up alternately at right side and then at left side of forward direction at every survey station.

8. When measured values exceed the allowable limits shown in the table below, releveling is required.

Classific-cation	1st class leveling	2nd class leveling	3rd class leveling	4th class leveling	Simplified leveling
Discre-pancy between fore and back trip	2,5mm$\sqrt{S}$	5mm$\sqrt{S}$	10mm$\sqrt{S}$	20mm$\sqrt{S}$	40mm$\sqrt{S}$
Closure error of circuit	2,5mm$\sqrt{S}$	5mm$\sqrt{S}$	10mm$\sqrt{S}$	20mm$\sqrt{S}$	mm mm 50+40$\sqrt{S}$

S: Observation distance (one-direction) in km

9. When releveling is conducted, it shall not be allowed to accept only measured values of the same direction for 1st and 2nd order leveling.

10. Starting point and ending point of the leveling route shall be determined after examining the records of movements in the past, and their relation to existing adjacent stations shall be checked before starting observation in order to check possible abnormality that may exist in these points.

Section 6. Computation and Arrangement

(Summary)

Article 13. Elevation of bench mark shall be obtained from observed data after rod correction (only for 1st and 2nd class levelings), normal orthometric correction (only for 1st, 2nd and 3rd class levelings) and, if necessary, correction for movement due to difference of observation time and after performing adjustment computations for leveling net based upon these corrections.

2. Results of observation and computations shall be separately filed generally into the result table, table for correction due to movements of ground and leveling adjustment computation table.

(Descriptions and computations in observation fieldbook, result table, table for correction due to movement of leveling route, and leveling adjustment computation table)

Article 14. Descriptions and computations for the observation fieldbook shall be made in accordance with Method of Presentation.

2. Descriptions and computations for the result table shall be made in accordance with Method of Presentation.

3. Correction due to movement for the area where ground movement is observed shall be computed based upon the specified standard date.

4. Adjustment computations shall be performed with weights inversely proportional to the length of routes, using observation equation or condition equation.

(Arrangement)

Article 15. The arrangement for observation fieldbook, adjustment computation table, description of points, result table, adjustment result table, precision control table, leveling route map, station site investigation report, written approval for installing bench marks, and notice of location of installed station markers and so forth shall be made in accordance with the Method of Presentation.

Section 7. Final Results

(Final results)

Article 16. Final results are:

(1) Observation fieldbook

(2) Computation table

(3) Description of points

(4) Observation result table and Adjustment result table

(5) Leveling route map

(6) Written approval for installing bench marks and Station site investigation report

(7) Precision control table

(8) Field inspection book

(9) Adjustment map

(10) Photographs of survey markers taken on ground

(11) Report on abnormal control points

PART VII.

OPERATION RULES FOR ASTRONOMICAL SURVEYS

CHAPTER 1. GENERAL RULES

Section 1. General

(Summary)

Article 1. These operation rules prescribe the method of observation, computation and arrangement for 1st and 2nd order astronomical surveys to be performed by the Geographical Survey Institute.

2. 1st order astronomical survey is the operation for observing the astronomical latitude and longitude and the astronomical azimuth of primary control points.

3. 2nd order astronomical survey is the operation for observing astronomical latitude and longitude of primary or secondary control points.

(Purpose)

Article 2. The purposes of the astronomical surveys are to determine the shape of geoid as the base of geodetic survey from the difference (vertical deflections) between the astronomical latitude and longitude and the geodetic latitude and longitude obtained from the precise geodetic survey network, and to correct the accumulative errors of the geodetic survey network by observing the astronomical azimuth.

CHAPTER 2. 1ST & 2ND ASTRONOMICAL SURVEYS

Section 1. Preparation for Operation

(Summary)

Article 3. Members of a survey team shall perform necessary preparation in detail prior to the commencement of operation in order to assure the smooth flow of survey operation and to improve the accuracy of surveys.

Section 2. Selection of Stations

(Summary)

Article 4. The selection of stations is the operation to investigate and select astronomical observation points by thoroughly considering the density of astronomical points and the degree of facility of operation.

2. When astronomical observation points are selected, their positions shall be indicated on 1/200,000 or 1/50,000 scale map to make a selection map with necessary items noted.

3. When using astronomical observation points selected, the use of the site for the points must be approved by owners or representatives of property.

Section 3. Preparation of Tables of Stars

(Summary)

Article 5. For performing smooth observation, tables of starts shall be produced which includes time when each observed star reaches a certain altitude, and data necessary for observation such as star azimuth at that time and start magnitude.

Section 4. Observation

(Summary)

Article 6. Observation is the operation to find the astronomical latitude and longitude of triangulation survey points with equal altitude method using astrolabes and to determine the astronomical azimuth by using special-class transits.

2. Since the accuracy of the observation directly reflects on the results, extra precautions must be taken in handling instruments and in obeying the observation method.

(Observation points)

Article 7. Observation shall be performed on the traingulation survey points as a rule.

(Inspection and adjustment of instruments)

Article 8. Astrolabes must be inspected and adjusted for each item listed below prior to the observation.

(1) Horizontal adjustment of the main body by using the main level.

(2) Horizontal adjustment of the prism section.

(3) Inspection for horizontal in the knife edge direction.

(4) Horizontal adjustment of the knife position indicating lines in the guide telescope.

(5) Inspection for reversal angle by the graduated circle.

(6) Adjustment for collimation axes of the main telescope and the guide telescope.

(7) Adjustment for electrical equipments.

2. Special-class transits shall conform to surveying instrument performance standards of Geographical Survey Institute and necessary inspection and adjustment shall be performed prior to operation.

(Execution of observation)

Article 9. In the 1st order astronomical surveying operation, observation consisting of more than 6 sets as standard (20 to 24 starts for one set) shall be performed at each survey station, and 2 sets of observation (each set comprizes 12 pairs of observation) for the astronomical azimuth shall be performed using Polaris.

2. In the 2nd order astronomical surveying operation, observation consisting of more than 3 sets (20 to 24 stars for one set) shall be performed at each survey station.

(Measuring eccentric elements)

Article 10. When observation is performed with certain eccentricity, eccentric elements required for the eccentric computations must be measured.

Section 5. Computations and Arrangement

(Summary)

Article 11. The astronomical latitude and longitude and the astronomical azimuth shall be computed up to the digits shown in the table below, using general computing equations with observed values.

Astronomical latitude & longitude	Astronomical azimuth
0."01	0."01

2. Results and records shall be arranged by separating into record books, computation books, 1st order astronomical azimuth record books and computation books, deflection of vertical computation books, eccentricity measurement books and various maps.

(Field rough computations)

Article 12. Field rough computations shall be performed in the following manners:

(1) Find approximate values of astronomical latitude and longitude for each set, and check the properness of observed results.

(2) Compute approximate values of astronomical azimuth for each pair of observation and check the properness of observed values.

(Execution of computations)

Article 13. Computations shall be performed in the following manners in conformity with Method of Presentation.

(1) For computing astronomical latitude and longitude, time of passing fixed altitude of each star should be read from the records of observed starts and used as observation data, and adjustment computations shall be made to find latitude and longitude from the observed data.

(2) For computing the deflection of vertical, it can be determined from the astronomical latitude and longitude found from paragraph above and from the geodetic latitude and longitude.

(3) Astronomical azimuth shall be computed for each pair of observation and mean values be determined.

(Adjustment)

Article 14. Arrangement for various tables, books and maps shall be conducted in conformity with Method of Presentation.

(Inspection)

Article 15. Members of the survey team shall submit fieldbooks, computation books and maps to the head of section at an appropriate time for his inspection and checking.

PART VIII.

OPERATION RULES FOR 1ST & 2ND ORDER GRAVITY SURVEYS

CHAPTER 1. GENERAL RULES

Section 1. General

(Summary)

Article 1. These operation rules describe the methods and the scope of 1st and 2nd order gravity surveys to be conducted by the Geographical Survey Institute.

2. 1st order gravity survey prescribed in these operation rules is the operation for determining the values of gravity at newly installed or existing 1st order gravity stations in major cities etc. throughout the nation with prescribed accuracy, using a gravity meter and basing upon the fundamental gravity stations in order to utilize the results as standards for gravity surveys.

3. 2nd order gravity survey prescribed in these operation rules is the operation for determining the values of gravity at existing or newly installed gravity stations for which elevations are known by bench marks, or triangulation survey points or traverse points (this will be called "control point" hereinafter), with prescribed accuracy using a gravity meter and basing upon the fundamental gravity stations, 1st order gravity stations and existing 2nd order gravity stations in order to utilize the results as geodetic materials.

(Density of gravity stations)

Article 2. Density of 1st order gravity stations shall be such that about one to two stations be located in each prefecture as a standard.

2. 2nd order gravity stations shall be distributed uniformly with density as high as possible.

(Station name of gravity station)

Article 3. The station name for 1st order gravity station shall be equal to the name of place where it is located as a rule.

2. For the name of 2nd order gravity station, the station name should be used for the control point but other appropriate names can be used for other points.

(Accuracy of gravity station)

article 4. 1st order gravity station must have highly accurate value of gravity at all time by repetitive observations.

2. 2nd order gravity station must also have designated accuracy based upon 1st order gravity station at any time.

3. The scope or limit of the observation will be described in other paragraphs.

CHAPTER 2. 1ST & 2ND ORDER GRAVITY SURVEYS

Section 1. Preparation for Operation

(Summary)

Article 5. Members of the survey team shall make preparation in detail for the survey before the commencement of operation for effectively and smoothly carrying out the survey operation and for improving survey accuracy.

Section 2. Reconnaissance

(Summary)

Article 6. Reconnaissance is the operation to thoroughly check whether the location of gravity stations planned on the map is appropriate in view of conservation and utilization of markers and of accessibility to them and to select the most desirable location.

(Setting up gravity stations)

Article 7. Setting up gravity stations means the determination of values of gravity at the stations with markers or stations with good reproducability without markers.

2. The location of 1st order gravity station should have specially solid ground and be appropriate for conserving and utilizing the markers.

3. For 2nd order gravity stations, existing control points may be used and also the location where its elevation has been determined with prescribed accuracy (this will be called "spot elevation" hereinafter) can be used.

4. When the gravity station is set up, the description of point should be produced for the station. The address of gravity station, the name of owner, the route to the station and the detailed vicinity map shall be shown in the description of point. However, the description of point is not required when the control point is used.

(Selection of survey stations)

Article 8. In 1st order gravity survey, survey stations shall be selected in such a manner that the difference in gravity between survey stations is kept small and the transportation time will not become too long.

2. In 2nd order gravity survey, control points should be

selected as control points as a rule but, if they are not available, spot elevation can be selected.

Section 3. Installation of Permanent Markers

(Summary)

Article 9. When 1st order gravity station is newly installed, prescribed permanent markers shall be installed.

2. For 2nd order gravity station, new markers should not be installed if there are existing markers. Spot elevation should not be installed unless especially needed.

Section 4. Observation

(Summary)

Article 10. Observation means a sequential reading of gravity meter in accordance with the predetermined method and finding relative difference in values of gravity among survey stations.

2. In 1st order gravity survey, a closed round-trip observation shall be made from the fundamental gravity station to the same fundamental gravity station as a rule.

3. In 2nd order gravity survey, a closed round-trip observation shall be made from an existing gravity station to the same gravity station. It should be closed within a day as a rule but, if necessary, a one-way trip observation from an existing gravity station to another gravity station, or a closure after more than two days may be allowed.

4. A team of observation shall have more than two persons including an observer and the same observer for each gravity meter shall be involved untill the observation is closed.

5. Since accuracy of observation directly affects the results of gravity, extra precautions shall be paid in handling instruments and selecting a transporting method and observation method.

(Observation instruments)

Article 11. The gravity meter shall be LaCoste & Romberg gravimeter hereafter referred to as "Lacoste gravimeter". Since this gravity meter is a highly accurate, sensitive instrument, it must be handled with extra precautions.

2. Inspection of the Lacoste gravimeter will be prescribed later.

(Preparation for observation)

Article 12. Charge a battery for a period of time sufficient for the observation hours.

2. In 1st order gravity survey, perform a test observation for several days before going to field and upon completion of operation in order to grasp the drift conditions.

3. In 2nd order gravity survey, perform a test observation before going out for a daily operation and after completing a daily operation for verifying that it is capable to operate normally.

(Execution of observation)

Article 13. More than two gravimeters shall be used for 1st order gravity survey and more than one for 2nd order gravity survey.

2. When arrived at a survey station, verify the station by referring to maps and the description of point.

3. Gravimeter shall be set up on top of the marker. Use a measuring table if required. However, if it is difficult to place the gravimeter on top of a marker, the observation can be made at nearby stable point and relative position between a marker and an observation point shall be clearly indicated on the fieldbook.

4. If an extra high precision is required, special considerations must be taken for the number of reading of gravimeter, the reading time and the time spent for finding stone markers.

5. The observation fieldbook shall be described in accordance with Method of Presentation.

(Restrictions for observation)

Article 14. In 1st order gravity survey, a resurveying will not be performed as a rule. If a large tear occures, it should be handled by the method prescribed in other documents.

2. In 2nd order gravity survey, a resurveying will be necessary as a rule if the difference of observed values between going and returning observation exceeds a predetermined value after earth tide correction.

(Tying between 1st and 2nd order gravity stations)

Article 15. As a rule, more than one auxiliary station tied to 1st order gravity station shall be provided at each 1st order gravity station. More than two gravimeters shall be used for an observing auxiliary station. As a rule, the round trip observation tied from 1st order gravity station to the same station is required. As auxiliary station, a bench mark near 1st order gravity station on the solid ground proper for a long-term preservation can be used.

2. In 2nd order gravity survey, 1st order gravity station must be tied to 2nd order gravity station used as a standard for the gravity survey within an operation area.

(Checking, moving and reinstalling of gravity station)

Article 16. If a change of the gravity value is suspected due to a considerable change of environment near the 1st order gravity station, the gravity value of this station must be checked. If a change in gravity is detected, the new gravity value must be determined and the description of point be newly made.

2. If 1st order gravity station is lost, it must be reinstalled or moved to the other location if its loss in future is expected.

3. For 2nd order gravity station, the reinstallation is not required if its loss is negligible.

Section 5. Computations, Arrangement, Test and Inspection

(Summary)

Article 17. The gravity value at a gravity station shall be computed from the gravity value at an existing gravity station after finding the difference of gravity between survey stations after making corrections for earth tide, instrument height and drift to the observed values. Gravity anomaly shall be found from these results. Types of gravity anomaly are free air anomaly, simple Bouguer anomaly, and Bouguer anomaly.

2. Results of observation and computations shall be arranged and generally divided into the gravity difference result table and the observation result table for each operation.

3. When the gravity net is completed, a re-computation shall be made by the net adjustment.

(Descriptions and computation in observation fieldbook, the gravity difference computation table and observation result table)

Article 18. Descriptions and computations in observation fieldbook, gravity difference computation table and observation result table shall be made in accordance with Method of Presentation.

(Arrangement)

Article 19. Observation fieldbook, gravity difference computation table, observation result table, description of point, and gravity station layout map shall be arranged in accordance with Method of Presentation.

(Check and inspection)

Article 20. Survey team members shall submit various tables and books to the head of section for his inspection and check.

(Results)

Article 21. Required results are:

Observation fieldbook, gravity difference computation table, observation result table, description of point, gravity station layout map and other necessary materials.

PART IX.

OPERATION RULES FOR 1ST ORDER MAGNETIC SURVEYS

CHAPTER 1. GENERAL RULES

Section 1. General

(Summary)

Article 1. These operation rules prescribe the observation method for 1st order magnetic survey to be performed by the Geographical Survey Institute and the restrictions thereof.

2. 1st order magnetic survey under the operation rules is the operation for measuring three elements of terrestrial magnetism at 1st order magnetic stations or similar ones in order to find the earth's magnetic field and its secular change.

3. Three elements of terrestrial magnetism are magnetic declination, dip of magnetism and geomagnetic total force.

(Number and name of magnetic station)

Article 2. Magnetic stations shall be named and numbered. Numbers shall be serial numbers throughout the country, and main place name in nearby location shall be used as the name of the station.

CHAPTER 2. 1ST ORDER MAGNETIC SURVEY

Section 1. Selection of Stations

(Summary)

Article 3. Selection of stations is the operation of selecting stations which satisfy the requirements for measuring natural magnetic field without being affected by artificial magnetic disturbance and where such requirements will be expected to be met also in future.

(Execution of reconnaissance)

Article 4. Reconnaissance shall be performed by the following manners:

1. In reconnaissance on the maps, it is required that magnetic stations be uniformly distributed throughout country and that a required minimum distance be maintained from cities and railroads since they often cause the disturbance of terrestrial magnetism.

2. In field, location planned and shown on the map shall be investigated, and a place where there is no source of magnetic disturbance shall be selected as the magnetic station. For preventing possible occurrence of necessity of moving or reinstallation in future, special consideration will be needed by verifying development plan or land utilization plan in the area with governmental authorities concerned before making final selection.

(Description of points and written approval for installing bench marks)

Article 5. When a magnetic station is selected, the description of the point shall be coordinated, the owner's or the representative's approval be obtained, and written approval for installing bench marks or station site investigation report be provided using standard forms.

Section 2. Installation of Permanent Markers Marking Stakes

(Summary)

Article 6. In installing permanent markers, prescribed permanent markers shall be installed at the location selected by the selecting operation and, in addition, marking stakes shall be provided.

(Installation of permanent markers)

Article 7. Installation of permanent markers at the magnetic stations shall be conducted in the following manners:

1. As a rule, installation of permanent markers shall be made securely before the observation in accordance with standard installation of permanent markers details.

2. When installing, the face marked as "1st order magnetic station" shall be directed to south and the top surface must be in level when installed.

(Installing marking stakes)

Article 8. Marking stakes shall be provided at the magnetic stations.

2. If existing stakes are rotten, they have to be replaced by new ones.

Section 3. Observation

(Summary)

Article 9. Observation is the operation for measuring three elements of terrestrial magnetism at the station marked above ground by using magnetic compasses and magnetometers.

2. Observation of terrestrial magnetism shall be carefully performed since it is easily affected by nearby artificial magnetic field.

3. Observation must be postponed when magnetic field is greatly disturbed by magnetic storm or others.

(Calibration)

Article 10. Magnetic compasses and magnetometers to be used for the observation shall be calibrated by comparing to standard in Geomagnetic Observatory before and after the operation and whenever necessary.

(Preparation for observation)

Article 11. Before the commencement of observations, the following preparations are needed:

1. Clean the area of the magnetic station which is about 3 meters to south and north and about 2 meters to east and west, and make sure that there is no source of magnetic disturbance such as steel material for a distance of about several dozen meters.

2. Leg stakes shall be securely driven and the tripod be installed on them.

3. For measuring geomagnetic total force, an auxiliary station shall be installed and the detector of magnetometer be fixed on the station.

4. Azimuth marker shall be securely installed at the point more than 50 meters away from the magnetic station.

(Descriptions on observation fieldbook)

Article 12. Descriptions on observation fieldbook, the revision of descriptions and computations shall be conducted in accordance with Method of Presentation.

(Observation for terrestrial magnetism)

Article 13. Observation for terrestrial magnetism shall be conducted at 00 minute every hour from 6:00 to 22:00 and also at 9:30.

2. Measurement of magnetic declination and dip by magnetic compasses shall be conducted in conformity with Magnetic Compass Instruction Manual.

3. Measurement of geomagnetic total force by magnetometer shall be conducted before and after the observation described in above paragraph and during the reversing of the magnetic compass.

(Observation of azimuth marker)

Article 14. Observation of the azimuth marker must be made before or after the observation by magnetic compass and after each revision of level.

(Measurement of local difference)

Article 15. Local difference in geomagnetic total force between the auxiliary station and the central point of magnetic compass from observed value of geomagnetic total force taken at the auxiliary station.

2. Measurement shall be conducted within two days before the observation and within two days after the observation. One set of measurement is needed at each time. In addition, the difference in geomagnetic total force between the central point of magnetic compass and the point about 50cm below the central point shall be measured.

(Observation of Azimuth)

Article 16. Azimuth observation shall be conducted for more than 6 pairs by using Polaris.

2. Observation can be made at any time.

3. The magnetic compass shall be properly installed on the same location as used for geomagnetic observation.

(Precautions for observation)

Article 17. Observation shall be conducted after verifying that there is no magnetic material around the magnetic station and its auxiliary station. Observers shall not have any magnetic material on their clothes and accessories. Since the magnetic compass is easily affected by electrical disturbance, it must be carefully handled and always well maintained. Even fine dusts are not allowed.

2. Observers shall bring previous observation results with them and carry out the operation by checking at all time whether observed values are appropriate compared to the previous results.

Section 4. Computations and Arrangement

(Computations)

Article 18. From the measured values of the three elements of terrestrial magnetism at the magnetic station, other components shall be computed.

2. Mean values of measured results of prescribed number shall be computed of each component. These mean values are to be then corrected using observed values taken by the standard observatory in order to unify them and reduce to values of standard year.

(Arrangement, inspection and check)

Article 19. Observation fieldbook, observation result table, observation adjustment table, description of point, etc. shall be arranged in accordance with the Method of Presentation.

2. Team members shall submit these tables and books to the head of section for his check and inspection.