Precise Survey of Reference Points at Crustal Movement Monitoring Stations Located in the Tokyo Metropolitan Area
1999 Survey Results

- Contents -

3. Reference Materials

3.1 SOKKIA NET2B / Leica TC2002 Inspection Reports

3.2 Certificate of Verification

3.3 Offset of Surveying Points

3.4 List of Instruments Used

3.5 Table of Telescope’s Attitude Data During its reference Point Observation

3.6 Table of Telescope’s Attitude Data During Penta-Prism Observation

3.7 Table of VLBI Antenna’s Attitude Data During its Reference Point Observation

1. Introduction

1.1 Outline of Survey Results

1.1.1 Table of survey results and lists of survey results

1) Survey results prepared in the year 1999 consist of six types of coordinate results where four are based respectively on local, horizontal, GRS80 and ITRF94 coordinate systems and two are based respectively on local and horizontal coordinate systems with geoid correction.

2) Furthermore, the results of the survey conducted once every year between 1996 and 1998 have been revised in the same format as the survey results in 1999 after recalculating.

3) As shown in Table 1 below, all of these aforesaid survey results are compiled in the following two formats: (1) “Table of Survey Results” where all the six types of survey results have been compiled separately by location (observation station) and by year; and (2) “List of Survey Results” where coordinate results based on horizontal coordinate system (with geoid correction) for the last four years (from 1996 to 1999) are displayed collectively.

Table 1 Types of survey results

Types	Coordinate system	Geoid correction	Coordinate value
Table of survey results	Local coordinate system	Without	X, Y, Z
	Ditto	With	X, Y, Z
	Horizontal coordinate system	Without	N, E, U
	Ditto	With	N, E, U
	GRS80 coordinate system	Without	Latitude, longitude, ellipsoidal height
	ITRF94 coordinate system	Without	X, Y, Z
List of survey results	Horizontal coordinate system	With	N, E, U

4) Four types of coordinate system shown in Table 1 are as follows:

(a) Local coordinate system

This is a coordinate system that ITRF94 earth-centered orthogonal coordinate system be shifted to the reference point ("station datum" from hereafter). There is one station datum for each station..

Station datum: Koganei station -> S3, Kashima station -> S3, Miura station -> S3, Tateyama station -> L2

(b) Horizontal coordinate system

Orthogonal coordinate system with plumb line axis set as U-axis, east as E-axis and north as N-axis at station datum (set in each observation station).

(c) GRS80 coordinate system

The geodetic coordinate system in ITRF94 frame where GRS80 ellipsoid is used.

(d) ITRF94 coordinate system

Earth-centered orthogonal coordinate system with Greenwich direction set as X-axis, earth's axis as Z-axis and the direction of X-axis around Z-axis that has been turned 90 degrees counterclockwise as Y-axis.

1.1.2 Observation values of SLR reference point (using survey method B)

1) Observation values of SLR reference point using survey method B are the observation values of horizontal angle, vertical angle and slope range of survey points shown in Table 2 below that have been measured by NET-2B.

Table 2 Horizontal angle reference directional point and survey points at each station (observation station)

	Koganei station	Kashima station	Miura station	Tateyama station
Horizontal angle reference directional point	H1	Survey suspended	S1	H1
Survey points	L1, L2, L3, H1, H2 SR-L1, SR-L2, SR-L3 SR-H1, SR-H2		L1, L2, L3, H1 SR-L1, SR-L2, SR-L3 SR-H1	L1, L2, L3, H1, H2 SR-L1, SR-L2, SR-L3 SR-H1, SR-H2

2) Survey points SR-** in Table 2 indicate the position of the center of the secondary mirror (reflection sheet for surveying has been attached) inside SLR telescope while SLR telescope aimed at long pillars L1, L2, L3 and leveling pillars H1, H2, respectively.

3) Horizontal angles shown in survey results are the values observed when the horizontal angle reference directional point was set as the reference point (0 deg. 0’ 0”) and the vertical angles are the values observed when the zenith was set as the reference point (0 deg. 0’ 0”).

4) L2 point at Koganei station was not possible to be measured by NET-2B, because of interference of visibility. Therefore, this point was measured by lifting a reflection sheet (241.8mm above L2 point).

5) At Kashima station, the survey was suspended due to some unknown reasons caused at the SLR telescope side that made it impossible to measure the range by NET2B.

6) At Miura station, observation values could not gained from H2 point due to a structure that blocked the optical route between SLR telescope and this point.

1.2 Determination of the orientation and positions of station datum

1.2.1 Determination of the orientation

1) At each station, a baseline between the station datum point and a far away point ("azimuth marker" hereafter) is set for determination of orientation. At the two end points of baseline, a 24-hours GPS survey is held and the azimuth is determined.

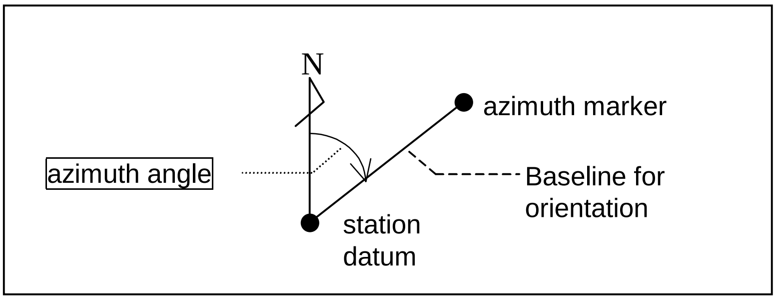


Figure 1 Determination of the orientation

2) Baseline for orientation is set to meet the following conditions:

(a) It must be long enough to keep the satisfaction of accuracy as the reference orientation.

(b) The two end points must be good for total station observation, that is be visible from each other.

(c) The two end points must be sky-opened to make sure that GPS observation is possible.

3) Regarding the station datum (S3) at Koganei station, however, conditions for bearing-determining base line stated above as (b) and (c) could not be met. Therefore, an alternative point (X10) has been set on the upper part of SLR observation tower as the substitute of S3.

4) The azimuth angle and the range of the bearing-determining base line that have been actually set in each observation station are as listed in Table 3.

Table 3 Azimuth angle and range of bearing-determining base line

Observation station	Station datum	Azimuth angle (station datum -> azimuth marker)	Range (km)
Koganei	(X10)	119 deg. 59’ 41”	1.9
Kashima	S3	206 deg. 32’ 25”	1.6
Miura	S3	182 deg. 01’ 21”	4.3
Tateyama	L2	245 deg. 20’ 23”	2.0

* X10indicated in ( ) is the alternative point used as the substitute of S3.

5) For the purpose of verifying the precision of the measurement of the bearing-determining base line, Table 4 shows the comparison of the bearing-determining base line that has been measured respectively by GPS used for direction determination and TS used for azimuth angle setting.

Table 4 Comparison of the range of bearing-determining base line measured by GPS and TS

Observation station	GPS measurement	TS measurement (TC2002)	Difference
Koganei	1905.7218 m	1905.7153 m	6.5mm
Kashima	1569.2475 m	1569.2458 m	1.7mm
Miura	4317.0768 m	4317.0679 m	8.9mm
Tateyama	2090.7171 m	2090.7162 m	0.9mm

1.2.2 Determination of the coordinates of the the station datum

1) At each station, the station datum's coordinates in ITRF94 frame are determined by conducting a GPS network survey in which the station datum and three GSI (Geographical Survey Institute) 's permanent GPS stations nearby are included, and performing a 3 dimensional network adjustment with the three GSI's as the fixed..

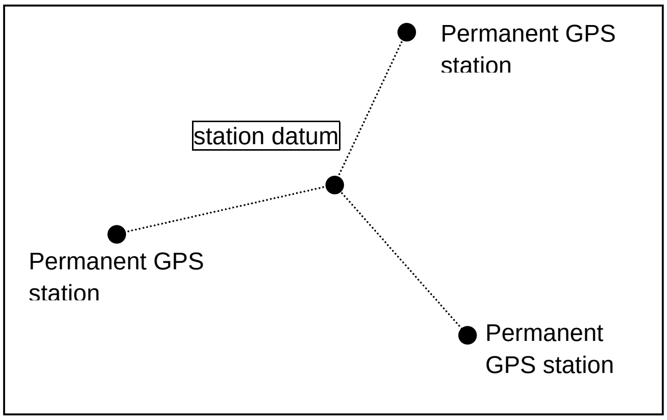


Figure 2 Station datum and electronic reference points

2) However, at Koganei station, the alternative point X10 is used for GPS survey, in the same way as how the reference azimuth angle was determined at this station.

3) As shown in Table 5, the standard deviation of the coordinates of GRS80 system of station datum that have been determined at four stations were found to be good at all the stations.

Table 5 Precision of the determined station datum

Observation station	Station datum	Standard deviation (mm)		
		$SD_{latitude}$	$SD_{longitude}$	SD_{height}
Koganei	X10	3.9	3.4	27.4
Kashima	S3	1.2	1.1	8.6
Miura	S3	2.6	2.2	18.1
Tateyama	L2	1.7	1.8	11.8

1.3 Coordinate Calculation

1.3.1 Coordinate calculation processing in 1999

Regarding the survey results in 1999, the results of the coordinates (latitude, longitude, ellipsoidal height) of GRS80 coordinate system have been first gained. Next, the results of the other five types of coordinate have been gained by calculation processing of coordinate conversion in the order of 2) -> 3) -> 4) -> 5) -> 6) on the basis of these coordinate results.

1) Calculation of the latitude, longitude and ellipsoidal height of GRS80 coordinate system
   The latitude, longitude and ellipsoidal height of GRS80 coordinate system of all points in the network are obtained through a three dimensional network adjustment in which the total station observations (range, horizontal angle and vertical angle), leveling observations are used as fixed point of height components, and the coordinates and orientaion at the station datum determined by GPS survey are used as the known.

2) Coordinate conversion from GRS80 coordinate system to ITRF94 coordinate system
   The latitude, longitude and ellipsoidal height of GRS80 coordinate system have been converted to X,Y and Z of ITRF94 coordinate system according to the following formula:

   $$X = (N+h) \cos(phi) \cos(lambda)$$
   $$Y = (N+h) \cos(phi) \sin(lambda)$$
   $$Z = \{N (1-e^2) +h\} \sin(lambda)$$

   Where, phi:latitude, lambda: longitude, h : ellipsoidal height, N : radius of east-west curvature, e : first eccentricity.

3) Conversion from ITRF94 coordinate system to local coordinate system (without geoid correction)
   X,Y and Z of ITRF94 coordinate system have been converted respectively to X_{local}, Y_{local} and Z_{local} of local coordinate system according to the following formula:

   $$X_{local} = X-X_0$$
   $$Y_{local} = Y-Y_0$$
   $$Z_{local} = Z-Z_0$$

Where, X_0, Y_0, and Z_0 are the coordinate values of the ITRF94 coordinate system of the station datum at each observation station.

4) Conversion from local coordinate system (without geoid correction) to horizontal coordinate system (without geoid correction)

X_{local}, Y_{local} and Z_{local} of local coordinate system have been converted to N, E and U of horizontal coordinate system according to the following formula:

$$\begin{bmatrix} N \\ E \\ U \end{bmatrix} = \begin{bmatrix} -\sin(phi)\cos(lambda) & -\sin(phi)\sin(lambda) & \cos(phi) \\ -\sin(lambda) & \cos(lambda) & 0 \\ \cos(phi)\cos(lambda) & \cos(phi)\sin(lambda) & \sin(phi) \end{bmatrix} \begin{bmatrix} X_{local} \\ Y_{local} \\ Z_{local} \end{bmatrix}$$

Where, phi : latitude, lambda : longitude.

5) Conversion from horizontal coordinate system (without geoid correction) to horizontal coordinate system (with geoid correction)

Geoid correction of horizontal coordinates has been executed according to the following formula by using the angle of inclination of the primary inclination constituent of geoid inclination (constituent xi for north-south direction and constituent eta for east-west direction). (Refer to 1.4 for the details regarding the primary inclination constituent of geoid.)

$$\begin{bmatrix} N_{geoid\_correction} \\ E_{geoid\_correction} \\ U_{geoid\_correction} \end{bmatrix} = \begin{bmatrix} \cos(xi) & 0 & -\sin(eta) \\ \sin(eta)\sin(xi) & \cos(eta) & \sin(eta)\cos(xi) \\ \cos(eta)\cos(xi) & -\sin(eta) & \cos(eta)\cos(xi) \end{bmatrix} \begin{bmatrix} N \\ E \\ U \end{bmatrix}$$

6) Conversion from horizontal coordinate system (with geoid correction) to local coordinate system (with geoid correction)

Horizontal coordinate system (with geoid correction) has been converted to local coordinate system (with geoid correction) by rotating the geoid correction ($N_{geoid\ correction}$, $E_{geoid\ correction}$ and $U_{geoid\ correction}$) of horizontal coordinate system to reverse direction according to the formula described in 4. The deviation that has been found with this rotation is only limited to the vertical deviation of the coordinate axis and had almost no effect at all to the conversion since the angle difference was very small.

$$\begin{bmatrix} X_{local\_geoid\_correction} \\ E_{local\_geoid\_correction} \\ U_{local\_geoid\_correction} \end{bmatrix}$$

$$= \begin{bmatrix} -\sin(phi)\cos(lambda) & -\sin(lambda) & \cos(phi)\cos(lambda) \\ -\sin(phi)\sin(lambda) & \cos(lambda) & \cos(phi)\sin(lambda) \\ \cos(phi) & 0 & \sin(phi) \end{bmatrix} \begin{bmatrix} N_{geoid\_correction} \\ E_{geoid\_correction} \\ U_{geoid\_correction} \end{bmatrix}$$

Where, phi : latitude, lambda : longitude.

1.3.2 Coordinate calculation processing in 1996 - 1998

1) The same procedure for coordinate calculation processing in gaining the survey results in 1999 is applied to data obtained once every year between 1996 - 1998.

2) However, as for the coordinates (latitude, longitude and ellipsoidal height) and the azimuth angle (station datum -> azimuth reference point) of the station datum (as shown in Table 6) which were necessary in the three-dimensional network adjustment for determining the latitude, longitude and ellipsoidal height of GRS80 coordinate system, the values gained in 1999 have been used.

Table 6 Station datum and Azimuth reference point

	Koganei station	Kashima station	Miura station	Tateyama station
Station datum	S3	S3	S3	L2
Azimuth reference point	S1	S1	L2	L1

1.4 Primary Level of Inclination of Geoid Correction

The primary level of inclination of geoid inclination used for geoid correction has been calculated in the order of 1) -> 2) -> 3).

1) Calculation of geoid undulation of WGS84 coordinate system
   By using the program “Geoid 96” developed by Geographical Survey Institute, Ministry of Construction, the geoid undulation of WGS84 coordinate system has been calculated respectively at the points approximately 500 meters north, south, east and west to the station datum at each observation station, so that all the surveying points at every observation station are within its range.

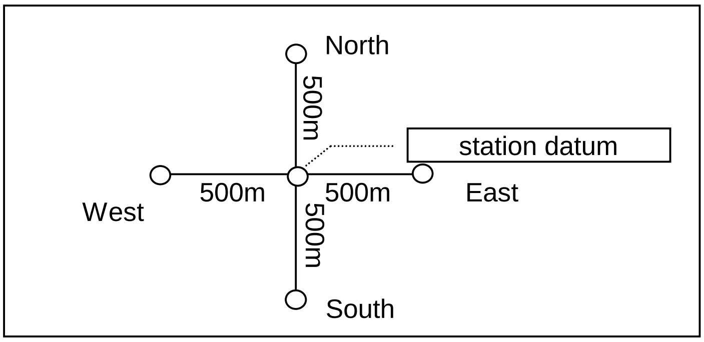


Figure 3 Map of geoid undulation calculation points

2) Determination of the geoid undulation of GRS80 coordinate system
   Since the ellipsoidal element and central position of both WGS 84 and GRS80 coordinate systems did not differ significantly, the same values calculated in 1) as the geoid undulations of WGS 84 coordinate system are used as the geoid undulations of GRS80 coordinate system.

3) Calculation of the primary level of inclination of geoid undulation
   As shown in Table 7, the primary level of inclination of geoid undulation (component xi for north-south direction and component eta for east-west direction) have been calculated respectively on the basis of the difference of the geoid undulation at both ends of east-west direction and north-south direction (relative geoid undulation difference) at each observation station which respectively is one kilometer (1000m) long.

Table 7 Primary level of inclination of geoid undulation at each station

	Koganei station	Kashima station	Miura station	Tateyama station
Component xi for north-south direction	11.4 seconds	16.4 seconds	-1.9 seconds	-4.5 seconds
Component eta for east-west direction	-17.4 seconds	-24.3 seconds	-9.3 seconds	-13.6 seconds

4) Verification of the geoid undulation by “Geoid 96”

At Kashima station, the geoid undulation at point (X8) has been determined by calculating the difference between the relative ellipsoidal height by GPS survey and the orthometric height by leveling (first class leveling). The discrepancy between this value and the geoid undulation by “Geoid 96” has been confirmed to be just 8mm and therefore these two values are thought to be consistent.

1.5 Verification of Surveying Instruments

The surveying instruments used in 1999, as listed below in Table 8, have been verified and calibrated by the authorized verification bodies listed in Table 9, prior to the survey. (Refer to "3. Reference Materials" for the details regarding the results of the verification.)

Table 8 Surveying instruments used

Model type	S/N	Performance of the instrument
Total Station NET2B : Sokkia	31481 31788	Precision of the surveying range: 0.8mm+1ppm * observation range Precision of the surveying angle: 2 seconds (Minimum readable unit: 1 second)
Total Station TC2002 : Leica	359698	Precision of the surveying range: 1.0mm+1ppm * observation range Precision of the surveying angle: 0.5 seconds (Minimum readable unit: 0.1 seconds)
Level DiNi11:Carl Zeiss	100312	First class level Verified by Japanese Association of Surveyors
Staff Invar staff : Carl Zeiss	12562 12564	First class staff Verified by Japanese Association of Surveyors
GPS receivers 4000SSI : Trimble	01693 01741 01746 06732	First class GPS receivers Precision of the surveying range: 5mm+1ppm * range Verified by Japanese Association of Surveyors

Table 9　Verification method

instrument type	Verification method	Verification body
NET2B	Measuring apparatus : Laser length-measuring instrument SL-2000 Base line : 100m underground base line	Sokkia Co. Ltd
TC2002	Base line : 1km outdoor base line	Sokkia Co. Ltd
DiNi11	Observation value check Compensator functional check	Japanese Association of Surveyors
Invar staff	Scale error check	Japanese Association of Surveyors
GPS receivers	Base line GPS relative base line field at Geographical Survey Institute	Japanese Association of Surveyors

1.6 On-site Inspection of Surveying Instruments

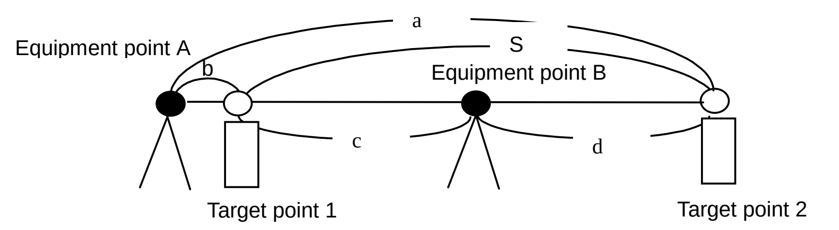
On-site inspection to check that all the surveying instruments work normally has been conducted at each observation station prior to the commencement of the survey in 1999.

1.6.1 On-site inspection of TS

(1) On-site inspection of the range measurement

On-site inspection of the range measurement has been conducted by applying 3-point method using the external geodetic survey monuments (short pillars).

(Refer to Figure 4.)



$$S \approx a - b \approx c + d$$

Figure 4 Drawing to explain the concept applied for on-site inspection of the range measurement (3-point method)

Compared to NET2B, the discrepancy was found to be quite significantly large with TC2002. The reason for this large discrepancy was that the reflection sheet used exclusively for NET2B could not be applied for observation using TC2002 and therefore, the inspection had to be conducted, using the reflection mirror (APS34) manufactured by Sokkia. According to explanation from Sokkia, the precision of their reflection mirror (APS34) may be affected slightly when it is used to measure a range that is closer than 30m.

Table 10 Results of the on-site inspection of the range measurement

station	Name of equip-ment	S/N	(a) s (m)	(b) a-b (m)	(c) c+d (m)	(a) – (c) Mean (m)	(a) – (c) Maximum discrepancy (mm)	Inspection date
Koganei	NET2B	31481	29.5935	29.5950	29.5934	29.5940	1.6	10/13
	NET2B	31788	29.5947	29.5946	29.5945	29.5946	0.2	10/13
	TC2002	359698	29.5959	29.5955	29.5974	29.5963	1.9	11/17
Kashima	NET2B	31481	24.5286	24.5282	24.5284	24.5284	0.4	10/23
	NET2B	31788	24.5285	24.5275	24.5285	24.5282	1.0	10/22
	TC2002	359698	24.5289	24.5289	24.5311	24.5296	2.1	10/22
Miura	NET2B	31481	47.9554	47.9557	47.9559	47.9557	0.3	11/12
	NET2B	31788	47.9557	47.9551	47.9566	47.9558	1.5	11/12
	TC2002	359698	47.9569	47.9552	47.9577	47.9566	2.5	11/12
Tateyama	NET2B	31481	8.3884	8.3880	8.3878	8.3881	0.6	11/09
	NET2B	31788	8.3886	8.3882	8.3891	8.3886	0.9	11/09
	TC2002	359698	8.3888	8.3879	8.3902	8.3890	2.3	11/09

(2) On-site inspection of the angle measurement

On-site inspection of the angle measurement has been conducted by (1) setting TS at the short pillars; (2) collimating other short pillars or long pillars with this TS; (3) measuring the angle of the same target with several other TS's (refer to Figure 5) and (4) comparing the measured values to confirm the performance of the angle measurement.

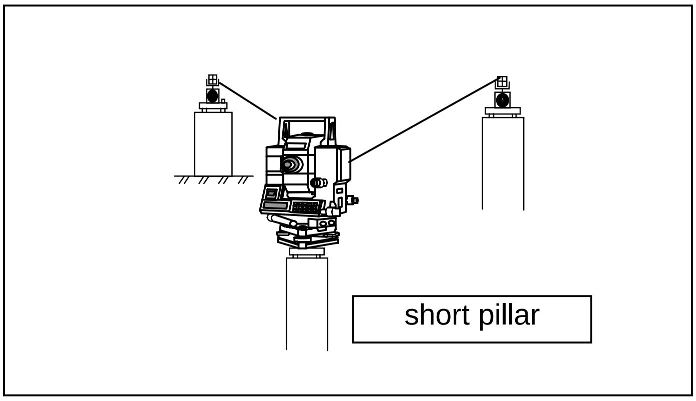


Figure 5 Drawing to explain the concept applied for on-site inspection of the angle measurement

Table 11　Results of the on-site inspection of the goniometric system

Observation station	Name of equipment	S/N	Horizontal angle (deg. ‘ “)	Vertical angle1 (deg. ‘ “)	Vertical angle 2 (deg. ‘ “)	Inspection date
Koganei	NET2B	31481	268.1804	-0.0454	2.1650	10/13
	NET2B	31788	268.1808	-0.0457	2.1650	10/13
	TC2002	359698	-	-	-	11/16
Kashima	NET2B	31481	303.3518	-0.0459	-0.0630	10/23
	NET2B	31788	303.3521	-0.0501	-0.0635	10/22
	TC2002	359698	303.3520	-0.0503	-0.0638	10/22
Miura	NET2B	31481	59.5740	-0.0324	-0.0857	11/12
	NET2B	31788	59.5743	-0.0324	-0.0856	11/12
	TC2002	359698	59.5742	-0.0329	-0.0906	11/12
Tateyama	NET2B	31481	258.1232	-0.1646	-0.1949	11/09
	NET2B	31788	258.1223	-0.1647	-0.1952	11/09
	TC2002	359698	258.1243	-0.1705	-0.2010	11/09

1) At Tateyama station, the discrepancy of the measurement of horizontal angle was found to be quite significantly large. The reason for this large discrepancy was because the target was only 8m away. The actual discrepancy is equivalent to only 0.7mm in distance and can therefore be considered to be within the tolerable range of measurement error.

2) At Koganei station, on-site inspection using TC2002 was not possible because the permission to enter the rooftop of Seiyu Stores’ s where azimuth marker was established, was restricted to a very limited amount of time. Therefore, this inspection was conducted at Miura station one day before the survey was scheduled to start at Koganei station (November 16th).

3) As a result of the on-site inspection of TC2002 for use in Koganei station that was conducted at Miura station, the discrepancy of horizontal angle was found to be 1 second.

1.6.2 On-site inspection of GPS

On-site inspection of GPS was conducted by setting a temporary base line randomly at a location where the sky was open above and comparing the range of the base line measured by GPS and TS. To secure the accuracy of the inspection, the measurements were compared after switching over the position of the antenna of GPS and the position of the reflection sheet and TS to prevent their positions from deviating (refer to Figure 6).

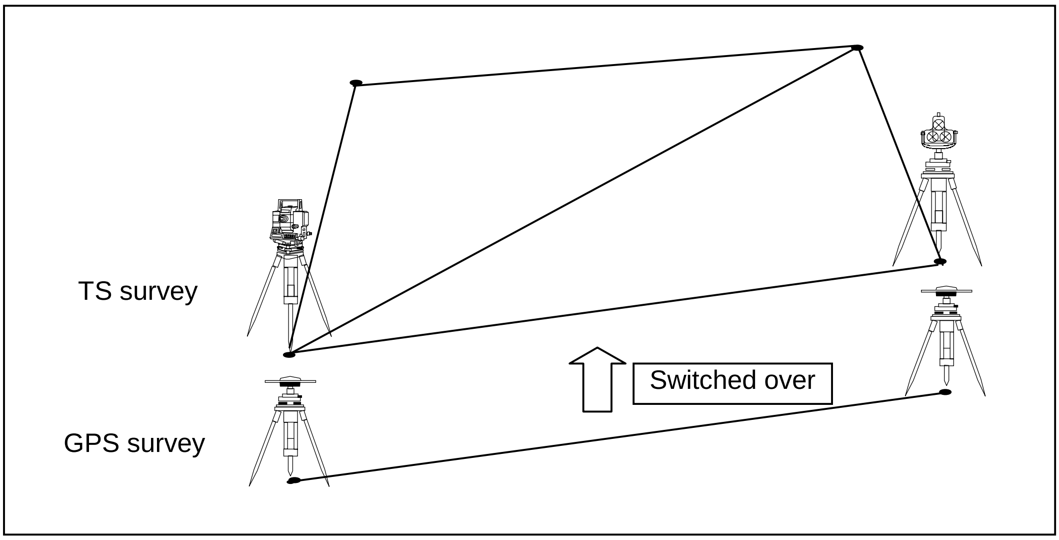


Figure 6 Drawing to explain the concept applied for the on-site inspection of GPS

Table 12 Results of the on-site inspection of GPS

Observation station	Name of equipment	S/N	Base line 1 (m)	Base line 2 (m)	Base line 3 (m)	Base line 4 (m)	Base line 5 (m)	Inspection date
Koganei	GPS	1741, 1746, 6732	62.2393	46.4643	60.2012	-	-	10/13
	NET2B	31788	62.2410	46.4631	60.2012	-	-	
	Discrepancy (mm)		-1.7	1.2	0.0	-	-	
Kashima	GPS	1693, 1741, 1746, 6732	34.4697	43.0530	24.3169	20.6703	24.2735	10/19
	NET2B	31788	34.4687	43.0529	24.3194	20.6706	24.2726	
	Discrepancy (mm)		1.0	0.1	-2.5	-0.3	0.9	
Miura	GPS	1693, 1746, 6732	44.0392	32.3560	43.3745	-	-	11/05
	NET2B	31788	44.0368	32.3559	43.3759	-	-	
	Discrepancy (mm)		2.4	0.1	-1.4	-	-	
Tateyama	GPS	1693, 1746, 6732	15.8045	18.8596	-	-	-	11/08
	NET2B	31788	15.8055	18.8630	-	-	-	
	Discrepancy (mm)		-1.0	-3.4	-	-	-	

The discrepancy of all the base lines that have been inspected was less than 3.4mm when compared with TS measurements. Therefore, the surveying instruments of GPS were all confirmed to be meeting the specifications of required standard performance.

1.6.3 On-site inspection of the level

On-site inspection of the level has been conducted by checking the flat-levelness of the collimation line of the level and the compensator (automatic correction unit) respectively through the application of pile driving check method (refer to Figure 7).

1) Pile driving check method applied here was a method of checking and adjusting the flat-levelness of the collimation line of the level by applying the principle that the deviation of the collimation line increases proportionately as the distance between the level and staff gets farther away.

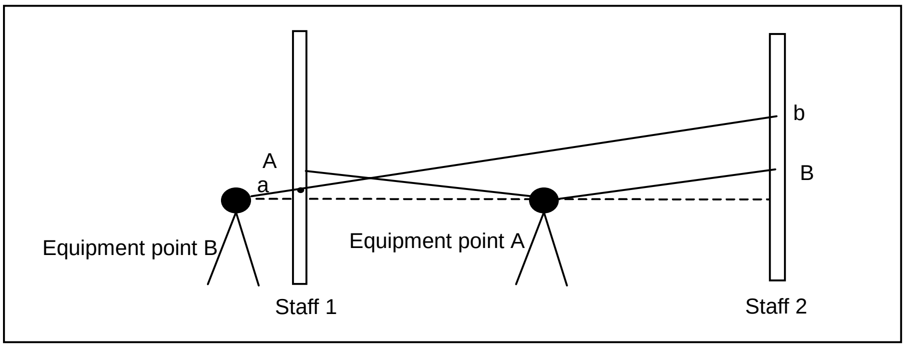


Figure 7 Drawing to explain the concept of pile driving check method

2) Compensator functional check method applied here was a method to compare the observation value gained from a level that has been adjusted to be placed at a position where the bubble is at the center and the observation values gained from a level that has been adjusted to be placed at a position where the bubble is deviated extremely to crosswise/lengthwise directions.

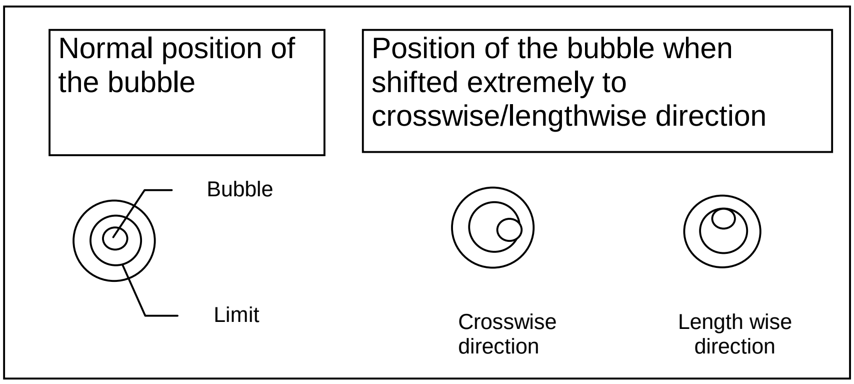


Figure 8 Drawing to explain the concept of compensator functional check method

Table 13 Results of the on-site inspection of the level

Observation station	Name of equipment : S/N	Pile driving check (mm)	Compensator functional check (mm)		Inspection date
Koganei	DiNi11 : 100312	0.13	Crosswise	0.01	10/18
	Invar staff : 12562,12564		Lengthwise	0.01	
Kashima	DiNi11 : 100312	0.01	Crosswise	0.13	10/23
	Invar staff : 12562,12564		Lengthwise	0.02	
Miura	DiNi11 : 100312	0.11	Crosswise	0.15	11/18
	Invar staff : 12562,12564		Lengthwise	0.09	
Tateyama	DiNi11 : 100312	0.04	Crosswise	0.06	11/09
	Invar staff : 12562,12564		Lengthwise	0.01	

(a) The result of the pile driving check showed evidently that the levels used for this survey all meet the standard specifications required for first class leveling.

(b) The result of the compensator functional check showed evidently that the compensators used for this survey all function normally.

1.7 Confirmation of the Consistency with the Survey Results in 1996 - 1998

1) Regarding the coordinate results of SLR external geodetic survey monuments (short pillars, long pillars) observed in 1999 survey, the discrepancy of the coordinates have been examined to confirm the consistency with the coordinate results observed in the survey conducted respectively in 1996 - 1998.
Table 14 shows the discrepancy of the coordinate results of horizontal coordinate system (with geoid correction) gained at each observation station respectively in each of these years.

Table 14 Discrepancy between the coordinates observed in 1999 and 1996 - 1998

(unit: mm)

Name of point	Year	Koganei station			Kashima station			Miura station			Tateyama station		
		N	E	U	N	E	U	N	E	U	N	E	U
SLR-S1	1996	-2.4	0.3	0.8	-0.3	1.2	0.2	-1.8	0.9	0.2	0.4	2.6	-0.4
	1997	-2.7	0.4	0.1	-0.1	0.4	0.0	-0.6	0.6	0.0	0.7	2.4	-0.4
	1998	-2.5	0.4	-0.1	-0.1	0.2	0.0	-0.1	0.9	0.2	0.5	2.2	-0.3
SLR-S2	1996	-6.5	5.5	-0.2	0.9	-0.7	0.0	-1.0	2.3	-0.1	-0.1	0.3	0.4
	1997	-4.5	2.0	0.1	0.7	-0.8	0.1	-0.3	1.6	-0.2	0.3	0.9	0.2
	1998	-4.5	1.8	0.2	0.5	-0.6	-0.1	0.1	1.5	0.0	0.5	1.1	0.0
SLR-S3	1996	-	-	-	-	-	-	-	-	-	-1.5	0.8	-0.2
	1997	-	-	-	-	-	-	-	-	-	-1.9	1.4	-0.1
	1998	-	-	-	-	-	-	-	-	-	-2.4	2.4	-0.1
SLR-L1	1996	-2.5	-1.5	-0.6	0.7	0.9	0.1	-2.3	-0.4	0.1	3.0	3.6	-0.6
	1997	-1.9	-0.3	-0.1	0.3	0.2	-0.1	-1.3	0.0	-0.5	2.3	2.8	-0.3
	1998	-2.3	0.8	-0.3	0.3	0.1	0.0	-0.6	0.6	0.0	1.8	2.2	-0.1
SLR-L2	1996	-4.0	3.3	-0.4	2.1	-0.7	-0.1	-0.7	2.2	0.4	-	-	-
	1997	-3.0	1.8	0.1	1.0	-0.7	0.0	-0.3	1.1	-0.3	-	-	-
	1998	-3.6	2.3	-0.4	0.6	-0.6	-0.1	-0.4	1.1	0.0	-	-	-
SLR-L3	1996	0.0	0.9	0.2	-2.1	-2.0	-0.2	0.0	0.3	0.1	-1.2	0.6	-1.5
	1997	-0.5	0.6	0.2	-0.9	-1.0	-0.2	-0.3	-0.2	-0.2	-2.7	1.9	-0.1
	1998	-0.3	0.8	0.0	-0.7	-1.2	-0.1	-0.2	0.0	0.0	-1.5	2.3	0.1

2) “-“ represents that the point is the station datum.

3) At Miura station and Tateyama station, systematic discrepancy could not be found and the coordinate discrepancy is also insignificantly small.

4) At Koganei station, systematic discrepancy is found in southeast direction.

5) At Tateyama station, systematic discrepancy is found to broaden in fan shape from the station datum.

6) Since the causes of systematic discrepancy at Koganei station and Tateyama station have not been found to a sufficient level, no corrections are made.

1.8 Survey of SLR Telescope Reference Point

(1) Survey method

1) In 1999, the method of positioning the reference point of SLR telescope has been changed to “indirect method”, since the setting of survey target at the reference point has been found to be inadequate when the survey has been conducted by “direct method” in 1997 and 1998.

2) The indirect method applied here was a method to measure the survey targets placed near the reference point of SLR telescope from the three long pillars by using TS. AT each long pillar, the azimuth angle of SLR telescope has been adjusted to face the long pillar, and observations are made while SLR telescope is rotated to three positions along its elevation axis with an interval of approximately 45 deg. In this way, nine positions of the target on a sphere are gained. The method of least squares was applied to calculate the positioning coordinate of the spherical center (reference point) from these nine position’s coordinates.

3) However, at Koganei station, instead of using the long pillars (L1, L2, L3), base points (X1, X9, X10) have been used as the instrument points of TS. And at Tateyama station, base point (X1) has been used as the instrument point of TS instead of the long pillar (L3).

(2) Survey results

1) As displayed in Table 15, the result gained from the calculation of the spherical center (reference point) by applying the method of least squares to the nine coordinate values observed in 1999 survey by the “indirect method” showed evidently that the reference point was appropriate at all observation stations.

Table 15 Results of the calculation of the spherical center using the method of least squares

Observation station	Maximum value of the residual (mm)			Standard deviation of the spherical center (mm)		
	X	Y	H	SD_X	SD_Y	SD_H
Koganei	0.2	0.2	0.2	0.2	0.3	0.2
Kashima	0.2	0.1	0.1	0.1	0.1	0.1
Miura	0.1	0.1	0.1	0.1	0.1	0.1
Tateyama	0.2	0.2	0.2	0.2	0.1	0.1

* X,Y respectively represent the south-north and east-west component of the plane Cartesian coordinate system. H represents the vertical component.

2) Table 16 shows the comparison of the coordinates of SLR telescope reference point by “indirect method” in 1999 and those by “direct method” in 1997 and 1998 respectively.

Table 16 Difference of coordinates of SLR telescope reference point obtained in 1997 and 1998 compared to that in 1999.

(unit: mm)

Year	Koganei station			Kashima station			Miura station			Tateyama station		
	N	E	U	N	E	U	N	E	U	N	E	U
1997	0.4	0.4	1.7	0.2	-0.6	2.1	-0.7	0.2	5.0	0.2	2.5	2.0
1998	-1.6	-1.5	-1.9	0.6	-0.6	1.5	-0.2	0.0	1.9	-0.4	1.7	-1.4

1.9 Survey of KSP-VLBI Antenna Reference Point

(1) Survey method

1) In 1999, the survey of the reference point of KSP-VLBI antenna has been conducted by setting the survey target at the point of intersection of the axes of VLBI azimuth and elevation, and measuring this target from the survey points around VLBI antenna using TS, that is by the “direct method”.

2) For this survey, all the survey points in each station have been linked as an overall observation network and the positioning coordinates of the VLBI antenna reference point were determined by calculating the adjustment of this three-dimensional network.

(2) Survey results

1) As displayed in Table 17, the results of the three-dimensional network adjustment showed evidently that the precision of the position of the reference point of KSP-VLBI antenna in 1999 survey is satisfactory.

Table 17 Precision of the estimated position of KSP-VLBI antenna reference point by three-dimensional network adjustment.

Observation station	Standard deviation (mm)		
	SD_X	SD_Y	SD_H
Koganei	0.4	0.4	0.3
Kashima	0.6	1.0	0.3
Miura	0.7	0.8	0.6
Tateyama	0.4	0.5	0.2

2) Table 18 shows the comparison of the coordinates of KSP-VLBI antenna reference point gained in 1999 survey by “indirect method” and those gained respectively in 1996, 1997 and 1998 by “direct method”.

3) The same “direct method” applied in the survey conducted respectively in 1996 and 1998 was also applied in 1999 survey. Whereas in 1997, this survey was conducted by “indirect method” where the survey target set near the reference point was measured by rotating the azimuth angle and elevation angle of VLBI antenna.

Table 18 Coordinate discrepancies of KSP-VLBI antenna reference point gained in 1996, 1997 and 1998. compared to that gained in 1999.

(unit: mm)

Year	Koganei			Kashima			Miura			Tateyama		
	N	E	U	N	E	U	N	E	U	N	E	U
1996	-3.4	0.5	-0.8	4.4	-3.0	0.9	-7.8	-2.1	-1.4	0.4	-0.7	-0.3
1997	-2.6	-0.2	1.0	2.4	-1.9	1.6	-4.3	-0.8	-0.2	1.0	1.2	1.7
1998	-3.2	0.7	-1.5	1.7	-0.6	-0.9	-2.3	1.0	-1.4	1.0	0.7	2.2

1.10 Survey of 34mVLBI Antenna Reference Point

(1) Survey method

1) In 1999, the survey of the reference point of 34m VLBI antenna was conducted through the same “indirect method” applied in 1998 survey.

2) The indirect method applied here was a method to set a survey target near the reference point of VLBI antenna and measure this target with TS from base points located on two sides of VLBI antenna. From each of the base point, a total of nine positions of the target have been measured by rotating the azimuth angle and elevation angle of VLBI antenna respectively to three different directions. As a result, coordinates of the target at 18 different positions on a spherical surface have been gained. And from these coordinates, the coordinates of the spherical center (reference point) of VLBI antenna were estimated by applying the method of least squares.

3) For controlling the change of direction of the survey target whenever the azimuth angle and the angle of elevation of VLBI antenna have been shifted, remote radio-controlled method was used in 1999 instead of the manual method applied in the previous year (1998). Apparently, this new method brought significant improvement in terms of higher safety of the surveyors and better efficiency of their work.

(2) Survey results

1) As displayed in Table 19, the estimated coordinates of the spherical center (reference point) by applying the method of least squares to the eighteen (18) coordinate values gained in 1999 survey showed evidently that the reference point was appropriately determined.

Table 19 Results of the calculation of the spherical center using the method of least squares

Maximum value of the residual (mm)			Standard deviation of the spherical center (mm)		
X	Y	H	SD_X	SD_Y	SD_H
0.9	0.8	0.7	0.2	0.3	0.3

2) As compared in Table 20, the discrepancy between the coordinates of VLBI antenna reference point gained in 1999 survey and those gained in 1998 survey was insignificantly small.

Table 20 Discrepancy resulting from the comparison with the coordinates of VLBI antenna reference point gained in 1998 and in 1999

Year	N (mm)	E (mm)	U (mm)
1998	-1.1	0.6	0.7

1.11 Survey of 26mVLBI Antenna reference Point

1) In 1999, the survey of the reference point of 26m VLBI antenna was conducted through the same “indirect method” applied for the survey of the reference point of 36mVLBIantenna.

2) As displayed in Table 21, the results gained from the calculation of the spherical center (reference point) by applying the method of least squares to the eighteen (18) positioning coordinates that were set separately by shifting the positions around the spherical surface showed evidently that this reference point was appropriately determined.

Table 21 Results of the calculation of the spherical center using the method of least squares

Maximum value of the residual (mm)			Standard deviation of the spherical center (mm)		
X	Y	H	SD_X	SD_Y	SD_H
0.8	1.1	1.1	0.3	0.3	0.4