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Improved geodetic control by GPS for establishing a Geographic Information System for Sri Lanka

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1 INTRODUCTION

Sri Lanka is in the process of establishing a Geographic Information System (GIS). A GIS is a combination of spatial data which provides the relative location of points, lines or areas on the earth surface and attribute data related to these individual points, lines and polygons (Burrough, 1986).

Many base map layers, such as land use, elevation, soil type, transportation and cadastral layer showing property boundaries should be included in the GIS. Of these many layers, the cadastral map layer requires the highest level of spatial accuracy in the GIS because it is expected to resolve land registration, land transaction and boundary disputes in the country (Berugoda, 1987).

Currently, data for the base map layers are obtained from hard copy maps which are drawn to different scales. In Sri Lanka, a number of important data layers can be obtained from the 1 : 10,000 or 1 : 50,000 map series. Unfortunately, these series either do not cover the entire country or are not current. Therefore, many data layers have to be created using other map series; the 1 : 100,000 series can provide data for the land use layer, the 1 : 500,000 series for the transportation layer and the large scale series such as 1 : 1000 or 1 : 2000 for the cadastral layer.

When many different types of spatial data in the GIS are obtained from hard copy maps of different scales, a reliable spatial linkage mechanism is required for overlapping these digital layers. If the linkage mechanism is not reliable, the overlapping is not perfect and, as a result, the GIS will give inaccurate information at the data analysis stage.

Grid or projection coordinates obtained from hard copy maps are used in GIS as the linkage mechanism. In Sri Lanka a computerized GIS, even without property boundary information, is not available today. All the cadastral information requirements are obtained from hard copy maps (Manual GIS as opposed to computerized GIS). For cadastral layers in a GIS, this type of spatial linkage does not provide adequate coordinate control because the property boundaries, right of ways, utility lines etc, have to be determined within a tolerance of few centimeters. The solution to this problem is to develop an accurate geodetic control network covering the entire country, which can be used as the linkage mechanism of the GIS. The linkage mechanism of a GIS in a country is the geodetic control network (Committee on Geodesy, 1980). In "Arc/Info", a GIS software used in this study, this linkage mechanism is referred to as "tic file" in the master control coverage.

To develop an accurate linkage mechanism for the GIS, the geodetic control of Sri Lanka must be accurate for all data layers. The spatial accuracy needed in the GIS is dependent on the type of data layers that are required. Since the relative coordinates of property boundaries should be determined within an accuracy of few millimeters, the geodetic control must provide these with relative accuracy together with 5 cm absolute accuracy for cadastral surveying requirements of the GIS. Figure 1.1 shows how a geodetic control network functions in a GIS.

Today, using modern surveying instruments and Global Positioning System (GPS) of satellites, a cadastral surveying can be done with centimeter level relative accuracy and 5 cm level absolute accuracy.

In order to provide the centimeter level accuracy in parcel boundary surveys for GIS applications in Sri Lanka, we need to establish control stations at 10 km apart, having an absolute accuracy of 5 cm and relative accuracy of 1 :1,000,000. These control stations are the "C" order points in a modern GPS geodetic network.

The control network with "C" order points can be established using a primary control

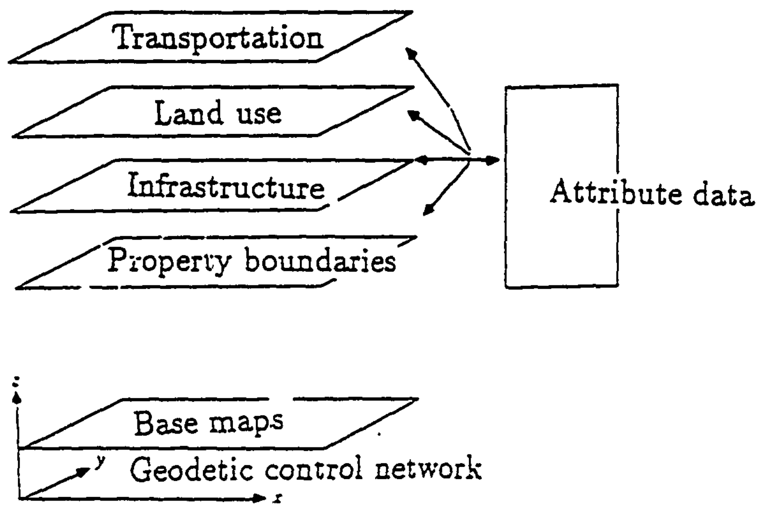


Figure 1.1 Geodetic control as the linkage mechanism of the GIS

network that has better relative and absolute accuracies. Such a network will have "A" order points within 1 cm absolute accuracy (100 km apart) and "B" order points with 2 cm absolute accuracy (25 km apart). Since this control network provides 1 part per million (ppm) relative accuracy in the "B" and "C" order points, it meets the specifications of the GPS network given by the Federal Geodetic Control Committee of the United States (FGCC, 1988).

1.1 Research objectives

The objectives of this research are to identify and describe the procedures of establishing a GIS for Sri Lanka and to find solutions for the ensuing problems. Specifically,

- it investigates the accuracy of the existing geodetic control network which was established in 1932 (Jackson and Price, 1933). In this stage of research we shall determine the suitability of the existing geodetic control as the linkage mechanism of the GIS.
- it develops the best procedure for establishing a new GPS geodetic network to

satisfy the needs of the comprehensive GIS, including an appropriate procedure for collection of GPS data which will provide the expected spatial accuracy after the adjustment of the network.

- it develops a suitable procedure for transformation between old and new coordinate system. It will transform latitudes and longitudes calculated on two different reference ellipsoids enabling us to combine the future with the past. This transformation methodology will also facilitate the incorporation of maps of the old plane coordinate system in the newly calculated linkage mechanism.
- it recommends a suitable cost effective, accurate and fast methodology to get cadastral level spatial information into the GIS from large scale cadastral maps and spatial information from small scale maps. A procedure for using scanners to produce raster data from hard copy maps and converting these raster data to vector is described here. These vector data are improved by a sequential least squares adjustment.

1.2 Present available accuracy of the network

The present geodetic control of the country was analyzed. It was found to provide only about 1 : 30,000 accuracy in the central regions of the country and the accuracy is even much weaker in the northern and north-eastern regions. Therefore, the existing geodetic network is not acceptable as the linkage mechanism of the GIS. It is also not suitable basis for the data collection for the cadastral map layer of the GIS because the desired accuracy is 1 : 1,000,000 for points which are 25 km apart (FGCC,1988). A detailed analysis of the present geodetic network is given in Chapter 3.

Due to the poor accuracy of the existing network, it is necessary to establish a new geodetic network before creating the GIS. The old geodetic network was established by a triangulation method. Today, the Global Positioning Systems (GPS) provide the best technology for establishing highly accurate geodetic networks (Leick, 1995). A suitable GPS data collection procedure, which will provide the expected accuracy of the new geodetic network, has to be developed. The GPS data collection procedure and recommendations are discussed in section 4.3 and 5.1.

1.3 GPS data collection procedure for a new adjustment

Working with simulated data, and adjusting the network with the least squares adjustment software "Geolab" (Bitwise Ideas, Inc.), it was found that a relative accuracy better than 1 cm can be achieved for "A" order points, 100 km apart, if random errors can be limited to 1 mm and the GPS observation procedure designed to limit ppm errors to 10^{-8}. These "A" order points can be taken as fixed points for adjusting "B" order points in the network. Details of this analysis are given in section 5.3.2.

For "B" order points, 25 km apart, the Survey Department of Sri Lanka proposes to take GPS observations, according to triangulation lines which were used in the old geodetic control network. Again, working with simulated data, it was found that only a 25 cm average absolute accuracy can be obtained with 5 mm random errors and 10^{-8} ppm errors with proposed procedure. This accuracy can be improved to 2 cm average relative accuracy by using the procedure explained in section 5.3.4.

1.4 Transformation parameters

Transformation parameters are necessary to transform coordinates between old and new systems in order to incorporate all the old maps into the new system. For this purpose, it is required to calculate a suitable set of transformation parameters for the country. Using "Geolab" and all currently available data, a new set of coordinates were obtained for the purpose of calculating transformation parameters. These parameters are expected to be comparable with parameters to be calculated after a complete adjustment proposed in this research because the differences between new and old coordinates will be mainly due to the adjustment procedure used in the old adjustment, discussed in Chapter 3.

The calculated parameters show that a single set of transformation parameters for the entire country provides 1 m accuracy for coordinate transformations. When dealing with small scale maps, this transformation accuracy is considered to be satisfactory according to the spatial accuracy of small scale maps given in Table 7.2. Nine sets of parameters calculated at provincial levels provided an accuracy of 30 cm; and 25 sets of parameters calculated at district levels provided 14 cm accuracy for coordinate transformations. Therefore, the district level parameters are suitable for coordinate transformation of large scale maps. Details of the calculation of transformation parameters are given in chapter 6. A software "*Con_cord*" was developed for the purpose of transforming many different types of coordinates to and from each other. The main menu of "*Con_cord*" is given in Appendix D, and subroutines developed for this purpose are given in Appendix J.

1.5 Spatial data for the cadastral map layer

Highly accurate spatial data are required for the cadastral map layer of the GIS, because it provides coordinates for most of the cadastral and engineering needs of the country. The best possible way of getting highly accurate data into the GIS is by direct entry of coordinates and measurements obtained by GPS or surveying (Byrene, 1991). As cost and time factors play a major role in establishing a GIS, it is not advisable to neglect already available cadastral and town survey maps in the country. The common method of converting these large scale maps to digital form is by manual digitizing, but this procedure is very costly, tedious and time consuming. Also, it can introduce more human errors to digitized coordinates. As the large scale cadastral and town survey maps are prepared using only one color and one line-thickness, it may be possible to use scanning and vectorization techniques to convert these large scale maps into the digital form. Working with a simulated cadastral map in the scale of 1 : 2000 it was found that the manual digitizing provided an average accuracy of 26 cm each in X and Y directions for coordinates of property boundaries. A scanning procedure which used 600 dpi and vectorization capability in "ArcInfo" provided 34 and 23 cm accuracy respectively in X and Y for coordinates of same points. These results show that the scanning technique may be used for converting cadastral maps into the digital form if other related difficulties in this can be overcome.

Two major difficulties were observed in scanning and vectorizing large scale maps. They are the creation of unnecessary nodes and the non-creation of some of the required nodes. Creation of unnecessary nodes (property corners) was about 65% for the used map. This additional 65% represented about 15% of nodes which are not created at the correct locations (see Appendix G). The "Cleaning" or "Building" procedure in "Arc/Info" could not be satisfactorily used to overcome these two problems. Instead, manual editing, which is not satisfactory for large projects, had to be used. However, an automated procedure must be developed in future research in order to remove unnecessary nodes and create any new nodes which are required but not created by scanning and vectorization. Details and statistics for the manual digitizing, scanning and vectorization done in this study are given in Chapter 7.

1.6 Improving digitized coordinates by a sequential adjustment

Coordinates obtained by manual digitizing can be improved by a least squares sequential adjustment (Tameem, 1994). This procedure can also be used to improve coordinates obtained from scanning and vectorization. A software was prepared to improve the coordinates obtained by scanning and vectorization, using linear, angular and area conditions. The procedure is discussed in detail in Chapter 7.

Although GIS software modules such as "CoGo" in ArcInfo can be used to input these linear. angular and area measurements in to a GIS system (ESRI. 1995). a sequential least square adjustment can be considered as a better procedure to upgrade property corners digitized from maps. This methodology will upgrade coordinates of property corners whenever and wherever additional measurements are available. The development of an automated procedure integrating GIS software with least squares sequential adjustment software facilitating the use of this methodology for large cadastral maps or large projects is recommended for future research study.

2 CONCEPTS OF A GIS AND THE SRI LANKAN GIS

The concept of GIS is not new to the world. Manual system of maps, fiscal data and census records have been in use for several centuries. The "Doomsday Book", a record of cadastral survey made under William the Conqueror in 1805 is a manual GIS (Committee on Geodesy, 1980). However, its capabilities are limited when compared to a modern computerized GIS. Only in the last two decades, with the advent of modern computer technology, has the GIS expanded its capabilities and possible applications.

A GIS is an important decision making tool for a developing country like Sri Lanka. Available resources, mainly earth-related resources, are limited; a small country must manage its resources efficiently. A GIS provides attractive long and short term benefits which more than offset the initial cost of establishing a GIS (Dale and McLaughlin, 1987).

The strength of the GIS depends on the data base. In order to provide accurate information for decision making, the data base must be well interpreted and spatially accurate.

2.1 Components of a GIS data base

The GIS data base is a combination of a number of digital maps known as data layers, which have the spatial and attribute component. A GIS is more effective and efficient if all related data layers are included in it. Therefore, it is necessary to include a large quantity of attribute data in the data base. However, the more attribute data are included, the more costly and time consuming it is. The lack of availability of trained personal also makes it necessary to limit the amount of attribute data included in a data base, at least in the initial stage.

2.2 The Sri Lankan GIS

Sri Lanka is still in its initial stage of establishing a computerized and integrated GIS. Several government and affiliated organizations such as the Ministry of Planning, Department of forests, International Irrigation Management Institute of the U.N. (IIMI) and Integrated Rural Development Projects (IRDP) have established GIS. These systems are limited to small geographical areas and none of them have the capability of handling cadastral and boundary problems. They provide solutions to problems of resource management within each agency. They are, however, incapable of solving problems at the spatial and attribute levels of the country as a whole.

A proposal for establishing an integrated GIS for the country was made and accepted by the government in 1987. The system will be coordinated by the Land Use Policy Planning Division (LUPPD) of the Ministry of Lands and Land development. Although a final decision has not been made about the data layers and the time frame, a proposal to establish the system in 3 phases (Berugoda, 1987) has to be decided during phase 1. The proposal is describe in Appendix B.

Establishing a cadastral map layer to show the property boundaries of the country has been given priority and is to be done in phase 1. This work will be done by the Survey Department of Sri Lanka. In order to expedite the work, the Department has acquired modern surveying instruments such as "total stations" and GPS receivers. Arrangements were in progress at the beginning of 1996 to purchase hardware and software for the GIS. Assessment of the geodetic network and the work required to improve it must be done before the data collection for the cadastral map layer of the GIS.

When a GIS is established, it will be the major spatial information supplier for many decision-making processes. The cadastral map layer will have to provide accurate spatial information for almost all cadastral and engineering applications. Therefore, it is important to know the spatial accuracy of the system before it is established. The spatial accuracy of the GIS is dependent on the linkage mechanism of the GIS or the geodetic reference framework. Hence, the strengths and weaknesses of the reference network must be evaluated before the data acquisition for the GIS. If the present accuracy of the geodetic network does not provide the required spatial accuracy, the problem must be solved before establishing the GIS.

3 EXISTING GEODETIC CONTROL OF SRI LANKA

3.1 History

Systematic triangulation in Sri Lanka (then Ceylon) commenced in 1857 with the measurement of the Negombo base (Jackson and Price, 1933). During that time, triangulation was the only means of establishing a geodetic control network (Kahmen and Faig, 1988). Prior to 1857, all the surveys in Sri Lanka were done in a sporadic basis, using magnetic azimuths and sometimes even without connecting them with each other. After the base measurement, trignometrical observations were made until 1887 throughout the country, except for most northern parts. The work was connected to the Indian network. This 1857 coordinate computation was found to be inconsistent and as a result, a new adjustment called a "new fixing" was made in 1890 (Jackson and Price, 1933).

The angular measurements used for both of the above adjustments were obtained using 13" vernear theodolites. The perspective of computation of coordinates was topographic mapping of the country. During the early 19^{th} century, a considerable amount of first order angular observations were also made using 8" vernear or 5" micrometer theodolites in order to fill open spaces of the principal triangulation network (Jackson and Price, 1933). Further, Jackson and Price say (1933, p2), "with the commencement of systematic large scale cadastral surveys in the country in early 1920s, the geodetic control available during that time was found to be not satisfactory and many inconsistencies and errors were observed."

In 1929, the surveyor general of Sri Lanka, Dowson, decided to investigate the condition of the network and make a possible re-computation. As a result, Price and Jackson were appointed to investigate the possibility of a re-adjustment of the network. They concluded that the observations made during 1860 and 1890 were satisfactory and recommended a re-computation (Price and Jackson, 1930).

Two base lines were re-measured and two astronomical azimuths of base lines were observed for the readjustment completed in 1932. In addition, angles of figure IX (see Appendix A), consisting of 6 stations, were re-observed. All the observations used for the 1932 adjustment were the angular observations made during 1858 to 1906 except for the two base measurements, two astronomical observations and re-observation of figure IX. The adjusted network in 1932 and its station names are also given in Appendix A.

3.2 Need for a re-adjustment

As mentioned in Chapter 1, geodetic control should provide sufficient accuracy for all the data layers of the GIS. The purpose of the 1932 coordinate adjustment of Sri Lanka was to provide a consistent set of results upon which secondary and minor triangulation could be based without serious distortions, and the triangulation was not observed for cadastral surveys (Jackson and Price, 1933).

The reason for re-adjusting the north American datum of 1927 (NAD27) is given as, "The weaknesses of NAD27 became apparent in several ways. Surveyors were buying accurate electronic distance measurement instruments and finding unexplainable discrepancies between the existing control network and distances measured by their own independent surveys (Schwartz, 1985, p8)."

This idea is valid for all the countries that have geodetic control networks established during the early part of the century. As mentioned in the previous section, Sri Lankan primary network was established in 1932 only to facilitate lower order triangulation projects, but not for cadastral surveying needs. Therefore, the accuracy of the network may be lower than cadastral surveying needs. Hence, the densified control such as traverses, established using the primary control have a very low accuracy. Therefore, the old network needs to be completely analyzed and the existing accuracy has to be determined before establishing the Sri Lankan GIS.

During the last 15 years, surveyors in Sri Lanka started using modern surveying instruments such as Electro Distance Measurements (E.D.M.) and found similar difficulties observed by the surveyors in the U.S., as described by Schwartz. These surveyors were usually forced to introduce distortions into their more accurate measurements to connect the work to the less accurate national geodetic control network.

In addition to the problems related to surveying, the low accuracy of the geodetic network was a major problem for other engineering projects such as dam movement monitoring, geological projects and seismic studies.

3.3 Means of evaluating weaknesses of the network

There may be a number of reasons for the weaknesses of the network. A research was conducted in the following four areas to see the reasons for the weaknesses.

1. Evaluation of observations used for 1932 adjustment
2. Comparison of new distance measurements with the distances obtained from the network
3. Evaluation of the methodology used for 1932 adjustment
4. Evaluation of the effect of the reference ellipsoid and geoid

3.3.1 Old observations

Observations used for 1932 adjustment can be summarized as follows (Jackson, 1932):

1. Two base line measurements which are in triangles 1 and 43.

2. 500 angle observations.

3. Two astronomical azimuth observations on two base lines.

Both base lines were about $5\frac{1}{2}$ miles long and about 127 miles apart. Both were measured in 1930, using "invar" tapes and the procedure was quite conventional for that period of time. Base lines were measured both forward and backward but the backward measurement for the base 1 has not been used for the adjustment due to the unfavorable conditions during measurements, suspecting errors in the measurement.

The average length of a side of a triangle was about 16 miles and the minimum and maximum being $5\frac{1}{2}$ and 50 miles, respectively. Angles were measured by 10 observers using 3 different types of vernier theodolites (8", 12", and 13") and a 5" micrometer theodolite. It was observed that the number of repetitions or the standard deviation of angle observations has not been used as weights in the adjustment (Jackson, 1932). According to the condition of angle observations, the present geodetic control network of Sri Lanka can be classified as third order triangulation. (For classification of networks, see Moffit and Bouchard, 9^{th} ed. pp360-362)

A least squares adjustment which is described in detail in Chapter 5, provides most probable values for unknowns (coordinates of points in a geodetic adjustment). Also, a least squares adjustment can be used to get an analysis about the quality of observations. In the 1932 adjustment, observations were measured angles, base line distances and azimuths. A new adjustment was performed using "Geolab" and old observations. Misclosures obtained for the old observations in this new adjustment are given in Table 3.1.

These values indicate that the old observations do not provide a high accuracy in a new adjustment. The absolute accuracy obtained using these observations ranges from 11.4 meters in the northern regions of the country to 0.3 meters in the central regions. The average absolute accuracy obtained is only 5.3 meters. Absolute accuracy obtained

Table 3.1 Misclosures in the observations used in 1932 adjustment

Type of Observation	Maximum	Mean	RMS
Angles	18.4 sec	-0.087 sec	1.95 sec
Baselines	0.17 m	0.0004 m	0.024 m
Azimuths	2.66 sec	0.0001 sec	0.075 sec

for all the points (using old observations) are given in Appendix C in the form of error ellipses.

3.3.2 Comparison of distances

The 1932 geodetic control adjustment was entirely a triangulation project adjusted using only 2 base lines and the method of condition equations. During the past few years, distances among triangulation stations in triangles 21, 22, 48, 49, 50, 54, 55, 56, 65, 66, 67, 108 ,109, 117 and 118 were measured by the Institute of Surveying and Mapping in Sri Lanka, using long range EDM instruments. These distances were reduced to the ellipsoid (Everest ellipsoid) and compared with the distances obtained by inverse calculation of geodetic coordinates (latitudes and longitudes) of 1932 adjustment. Results are given in Table 3.2. Station numbers and names for all stations are given in Appendix A.

Table 3.2 shows that the measured distances are not compatible with calculated distances. The differences are extremely large in many cases. In the 29 cases used for this study, differences between measured and calculated distances varied from 17 cm to 3.32 m. The average difference was 93.7 cm for 29 distances. We have to expect many more discrepancies in lower order control points which will be used for surveying property boundaries at the village level because they are established using the primary control network. Therefore, the coordinates of the 1932 adjustment are extremely weak and not suitable to use either as the linkage mechanism of the GIS or as a control for surveying with modern equipments. Existing accuracy of the network is discussed in

Table 3.2 Comparison of measured distances with calculated distances from 1932 coordinates.

Station from	Station to	Measured (meters)	Calculated (meters)	Differences (meters)
103	105	14625.318	14626.080	-0.762
103	99	22014.050	22014.560	-0.510
100	103	30386.142	30386.530	-0.388
100	99	28516.774	28516.950	-0.176
83	91	27818.916	27818.540	0.376
90	91	18855.027	18854.610	0.417
83	90	32869.964	32869.470	0.494
38	39	41238.347	41239.830	-1.483
39	47	12296.780	12296.970	-0.190
33	34	27060.375	27061.750	-1.375
37	47	28539.717	28540.260	-0.543
37	39	27121.144	27121.510	-0.366
38	40	28647.280	28647.930	-0.650
61	40	35994.657	35995.590	-0.933
61	39	21888.434	21889.710	-1.276
38	37	31312.177	31313.790	-1.613
33	38	42922.806	42925.250	-2.444
39	49	32466.006	32467.250	-1.244
40	39	45741.688	45743.250	-1.562
33	40	61106.702	61110.030	-3.328
61	49	41844.659	41846.130	-1.471
79	82	33952.457	33951.770	0.687
79	78	15364.904	15365.670	-0.766
80	79	25832.662	25832.280	0.382
80	82	19029.582	19029.090	0.492
53	56	16328.590	16328.850	-0.260
53	57	29502.465	29503.210	-0.745
56	57	14360.815	14361.440	-0.625
49	60	50179.715	50181.350	-1.635
		RMS of the differences		0.937 m
		Maximum Difference		3.328 m

section 3.4 in detail.

3.3.3 Evaluation of the old adjustment methodology

In order to evaluate the methodology used for the 1932 adjustment, a new adjustment was done using the same observations used for the old adjustment. This new adjustment was performed using a least square adjustment software "Geolab V 1.9"(Bitwise Ideas, Inc.). Resulting coordinates of this new adjustment were used to do a comparison similar to the one described in section 3.3.2. "Geolab" adjusts the network, forming an observation equation for each observation, and then performs a least squares adjustment.

The 1932 adjustment was done by dividing the total figure into 17 smaller figures (see Appendix A). This was required for the formation of condition equations. 151 polygon conditions, 55 center conditions and 78 side conditions were used, amounting to 284 conditions for the entire network (Jackson and Price, 1932). After the angles were adjusted by a least square solution, Clerk's formulas (Bomford, 1980) and seven figure log tables were used for the calculations of geodetic coordinates (latitudes and longitudes).

The comparison of distances was done by first converting the geodetic coordinates of the new adjustment to three dimensional Cartesian coordinates, which were then used for the calculation of ellipsoidal distances between stations (Fortran subroutines used for these calculations are given in Appendix J). Calculated distances were again compared with the measured and reduced distances. Results are given in Table 3.3.

Table 3.3 shows that the distances calculated using the coordinates of the new adjustment (with the same observations as the old adjustment) consistent with the measured values much better than the distances obtained using 1932 coordinates. The average deference between distances were reduced to 46.9 cm from 93.7 cm. This is an approximate improvement from the accuracy of 1 : 30,000 to 1 : 60,000. Hence, the methodology used for the calculation of 1932 adjustment can be significantly improved using a least

Table 3.3 Comparison of measured distances with calculated distances from a new adjustment using same old observations.

Station from	Station to	Measured (meters)	Calculated (meters)	Differences (meters)
103	105	14625.318	14625.52	-0.202
103	99	22014.050	22014.01	0.040
100	103	30386.142	30385.82	0.322
100	99	28516.774	28516.01	0.764
83	91	27818.916	27818.06	0.856
90	91	18855.027	18854.30	0.727
83	90	32869.964	32869.01	0.954
38	39	41238.347	41238.40	-0.053
39	47	12296.780	12296.45	0.330
33	34	27060.375	27060.72	-0.345
37	47	28539.717	28539.47	0.247
37	39	27121.144	27120.88	0.264
38	40	28647.280	28646.97	0.310
61	40	35994.657	35994.29	0.367
61	39	21888.434	21888.76	-0.326
38	37	31312.177	31312.63	-0.453
33	38	42922.806	42923.57	-0.764
39	49	32466.006	32465.56	0.446
40	39	45741.688	45741.54	0.148
33	40	61106.702	611107.56	-0.858
61	49	41844.659	41844.11	0.549
79	82	33952.457	33951.44	1.017
79	78	15364.904	15365.75	-0.846
80	79	25832.662	25831.91	0.752
80	82	19029.582	19028.93	0.652
53	56	16328.590	16328.46	0.130
53	57	29502.465	29502.45	0.015
56	57	14360.815	14361.12	-0.305
49	60	50179.715	50179.07	0.645
		RMS of differences		0.469 m
		Maximum difference		1.846 m

squares adjustment which considers the entire network as one figure.

3.3.4 Effect of the Geoid and the Ellipsoid

When measurements or calculations are made on the earth surface, we have to consider 3 surfaces (see Figure 3.1). They are:

1. The physical surface of the earth which cannot be mathematically defined.
2. The "Geoid", which is the equipotential surface of the earth.
3. The "Ellipsoid", which is the best mathematical approximation used for the calculations on the earth surface.

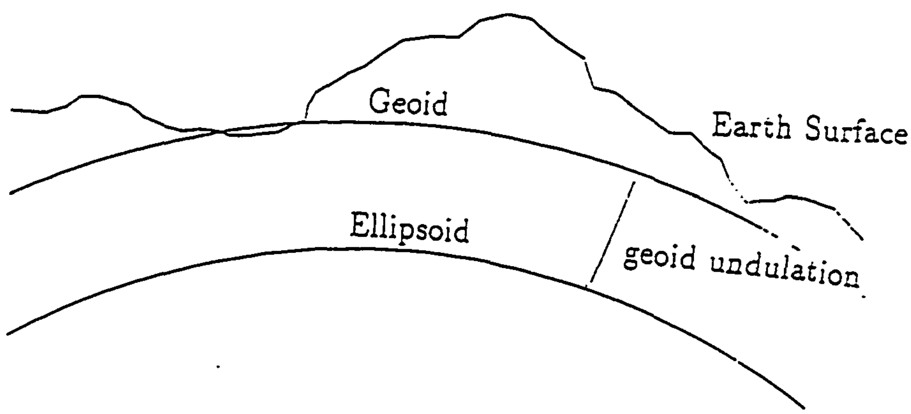


Figure 3.1 Physical surface of the earth, the geoid and the ellipsoid

When distances are measured on the earth surface, they should be reduced to the ellipsoid before the adjustment because all the calculations are done on the ellipsoid. When horizontal angles or azimuths are measured at a station, they are measured using the horizontal plane at the point of measurement. This is the plane perpendicular to the direction of gravity at the point. The direction of the gravity is perpendicular to the geoid but not to the ellipsoid. Hence, corrections have to be made to the measured angles and azimuths. This correction is known as the correction for the deviation of the vertical. Corrections required for observations in a geodetic networks are given in section 5.8.

According to Jackson (1933), two base lines measured for Sri Lankan triangulation were reduced to the mean sea level. This shows that the geoid was taken as the mean sea level and the ellipsoid was considered as coinciding with the geoid (at the mean sea level). Therefore, geoid undulations were neglected for the calculations. Geoid undulations have to be determined by sufficient gravity measurements in the country. The effect of geoid undulations is about 1 part per million for a geoid undulation of 6 meters (Bomford, 1980). Although the effect of neglecting geoid undulation is fairly small when a local ellipsoid is used, (see section 5.6 for different types of ellipsoids) this negligence is one of the factors for the low accuracy of the present geodetic network.

3.4 The present accuracy of the network

Expected accuracy of the a geodetic network can be evaluated by studying the classification of networks. Generally, classification of geodetic networks is done as first, second and third order. Sometimes these three orders are subdivided into classes, such as "second order first class" or "second order second class" (Moffit and Bouchard, 1991). According to the specifications of geodetic networks, closure in lengths for first order networks is 1 part in 100,000. For second order it is 1 part in 50,000 to 1 part in 20,000 in (class1 and class 2) and for the third order 1 part in 10,000 to 1 part in 5,000 (Moffit and Bouchard, 1991). For modern GPS geodetic networks, allowable errors in lengths given by FGCC (1988) are as follows:

1. Primary networks (Order A) 1 part in 10 million
2. Secondary networks (Order B) 1 part in 1 million

A comparison of measured distances and calculated distances from available coordinates was used to evaluate the classification of the Sri Lankan geodetic network. According to the Table 3.2, the average difference of lengths is 93 cm. As the average distance of a line of the network is 29.5 km, the available accuracy is approximately

1 : 30,000. When shorter lines such as the line of station 33 and 40 is considered, the accuracy is only in 1 : 17,000. Therefore, the accuracy of the network is between second order class 2 and third order class 1, according to the measurements taken in central and south-central regions of the country.

Since the length measurements were not observed in the northern and eastern regions of the country, the strengths of the network of those areas were found by comparing coordinates with the other parts of the network (same observations used for both adjustments). This comparison was made assuming that the reliability of angle observations is the same for the entire country and the weakness of the old adjustment is basically due to the methodology used for the calculations.

In order to obtain linear values of differences, both sets of coordinates were converted to plane coordinates using the Transverse Mercator projection. Software used for this transformation is given in Appendix J. Differences obtained in eastings and northings between old and new coordinates are shown in the map in Figure 3.2 and are also given in Appendix C in a tabular form.

These differences show that the northern and eastern parts of the present network are comparatively weaker than other areas. Therefore, when the entire network is considered, it is not possible to classify the existing Sri Lankan geodetic network even up to the third order.

Coordinate Differences in Northings

Differences in Eastings

Figure 3.2 Differences in Northings and Eastings between 1932 adjustment and a new adjustment using same observations. The largest circle indicates 14.2 meters

4 PRINCIPLES AND APPLICATIONS OF GPS

Principles and possible error sources of Global Positioning Systems (GPS) have to be identified before establishing a GPS geodetic control network. Therefore, a short description of the principles and applications of GPS is given in this chapter. It will provide basic knowledge about GPS essential to all who will be involved in establishing the new GPS geodetic control network in Sri Lanka.

In the past few decades, a number of positioning systems have been established and tested, but their success were very limited for accurate and widely-used positioning. These techniques include satellite photography, Very Long Base line Interferometry (VLBI), Lunar Laser Ranging (LLR), Satellite Laser Ranging (SLR) and Doppler Positioning Systems. All these methods were not satisfactory as a suitable method for establishing geodetic control networks (Leick, 1995).

Today, NAVISTAR (Navigation System with Time and Ranging) GPS provides accurate positioning for high order geodetic networks with convenience and a lower cost than older surveying and other positioning methods.

Older positioning methods require a considerably long field time and much more trained labour. Other than these obstacles, a real time accurate positioning with these conventional methods was nearly impossible (Leick, 1995). Thus, GPS has already proved that it is a far better positioning technology than any other previously available technique.

Accurate and current data collection for GIS can be done by using GPS, especially when the spatial data is required in relatively high accuracy such as creating the cadastral map layer. GPS has already acquired a wide popularity in applications such as land, marine and air transportation; recreational and military applications; surveying and geodesy; and space applications. In the field of geodetic surveying, GPS has already become the primary source of data collection for geodetic network adjustments. The differential GPS positioning technique, which will be discussed in detail in section 4.4.2, easily provides the required level of accuracy.

Another important area of GPS application is photogrammetry. Photogrammetry provides a reasonably accurate, current and detailed three-dimensional spatial data for GIS. The biggest concern for photogrammetry has been the need for ground control points (GCP) which are used to transform the coordinate system of aerial photographs to the ground coordinate system. Establishing suitable GCP using conventional methods is a fairly tedious and costly process. GPS has been used to get up to 30 cm accuracy for ground control points required for photogrammetric applications (Wells, 1987). Hence, it can be very effectively used in photogrammetry as a substitute for GCP needs. Main features of GPS can be described as follows:

1. High accuracy
2. Low cost to users
3. A unified world wide coordinate system
4. Suitability for many different types of applications

4.1 Components of GPS

Positioning with GPS is a result of three major components (Puterski, 1992).

1. Space segment
2. User segment
3. Control segment

The space segment consists of GPS satellites which transmit signals. These signals contain a number of information related to the satellite as well as the GPS time which is accurately determined by an atomic clock. Satellites were launched by NASA as 3 separate blocks called block 1, block 2 and block 2A. By the end of 1995, another 20 GPS satellites named block 2R were in development (Leick, 1995). Satellites were placed on 6 orbital planes, which are approximately 55 degrees inclination to the equator. Semi major axes of orbital paths of satellites are approximately 26,000 km from the center of the earth and the orbital period is slightly less than 12 hours. Because of the high altitude of satellites and due to the optimization of satellite visibility by keeping satellites unevenly in the orbital plane, more than 4 GPS satellites can be tracked from anywhere in the world, even after the mask angle of 15^0 (Leick, 1995).

User segment or the data collection segment consists of a GPS receiver and an antenna. The GPS data received by the antenna is transferred to the receiver through a cable. The GPS receiver is a combination of an amplifier, radio signal micro processor, control and display device, data recording unit and a power supply (Wells, 1987). The software and electronics in the GPS receiver decode the timing signals from 4 or more GPS satellites and first calculate the "pseudo range" (uncorrected distance) to the satellites and then compute the latitude, longitude and the elevation of the occupied station with respect to the GRS80 ellipsoid. All the data collected during the session are stored in the memory of the GPS receiver, so that they can be processed with respect to a control (known) station in order to get the geodetic level accuracy. Real time data processing can also be done with transmitted data from the control station. Details of these procedure are given in section 4.5.

Continuous monitoring and correcting the satellite orbit are required for efficient use of the GPS system. For this purpose, five monitoring stations continuously track all GPS signals for the purpose of controlling satellites and predicting their orbits (Wells, 1987). Locations of these stations are given in Table 4.1.

Table 4.1 Locations of GPS control stations

Station	Location
Colorado Springs	U.S.
Hawaii (U.S.)	Pacific Ocean
Ascencion (U.K.)	Atlantic Ocean
Diego Garcia (U.K.)	Indian Ocean
Kwajalein (U.S.)	Pacific Ocean

Positions of these monitoring stations are known to a very high accuracy. Colorado Springs station works as the master control station. Tracking data from the monitoring stations are transformed to the master control station for processing. This processing enables the accurate determination of satellite ephemerides, satellite clock corrections and other broadcasting message data which are transmitted back to satellites (Leick, 1995).

4.2 Orbital motion of a satellite

A short description of the orbital motion of a satellite is given in this section in order to understand the positioning of satellites in space. The motion of a satellite in an orbit is a result of the earth's gravitational attraction, attractions of the sun, moon and many other forces such as the pressure by solar radiation particles and the atmospheric drag (Wells, 1987). Kepler's laws describe the motion of a satellite in an ideal situation. A set of 6 parameters can be used to define the location of a satellite in the space at any given time.

These 6 parameters given according to figure 4.1 are:

1. Greenwich hour angle of the ascending node (Ω)
2. Inclination (i). The angle between the equatorial and orbital planes
3. Argument of perigee (ω). The angle between the nodal and perigee directions measured in the orbital planes

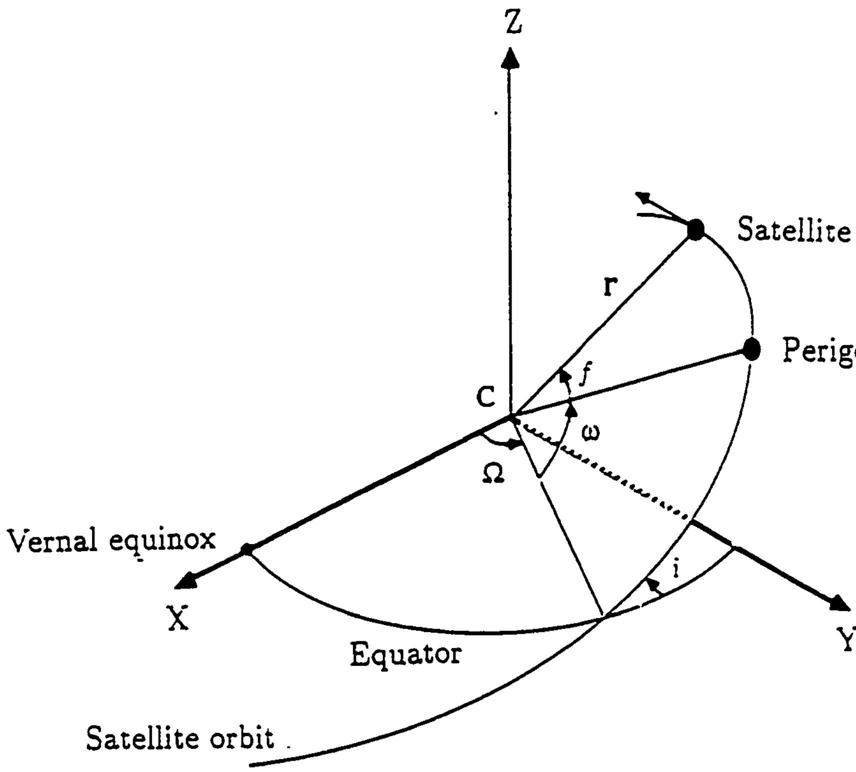


Figure 4.1 Parameters defining the position of a satellite in the space at a given time.

4. Semi major axis of the elliptical orbit (a)
5. Eccentricity of the orbit (e)
6. Time at perigee (t)

Apogee and perigee are the two points on the orbit where the distance from the earth to the satellite is maximum and minimum respectively.

The parameters Ω and i define the orientation of the orbital plane in space; ω defines the location of the perigee on the orbit and a and e define the size and shape of the orbit (Wells, 1987).

4.3 GPS satellite transmissions

GPS satellite transmissions are based on two high frequency wave bands. High frequencies are required to minimize ionospheric effects. These two wave bands are usually called $L1$ (Link 1) and $L2$ (Link2). Frequencies of $L1$ and $L2$ are 1575.42 MHz and 1227.60 MHz (Leick, 1995). These two carrier waves carry a number of modulated signals known as PRN (Pseudo Random Noise) codes. Out of these PRN codes, C/A code (Coarse/Acquisition code), and Pcode (Precise or protected code) are used for the determination of positions by GPS receivers. Although Pcode was considered as protected for military use, with new receiver designs, now there are no significant differences in position determination by Pcode or C/A code (Hunn, 1989). According to Puterski (1992), "Another PRN code, the Y code is now being transmitted on block 2 satellites. It is also for military use and its signal structure is more secure then P code."

4.4 Positioning methods

Positioning with GPS can basically be divided into two modes:

1. Stand along mode (Absolute positioning)

2. Relative mode (Differential positioning)

In both of these modes, the distance traveled by a wave between the satellites and the receiver is determined by measuring time delays or by using the technique of phase measurement. Measuring time delays to calculate the pseudo-range is usually called "pseudo-ranging" (Moffit and Bouchard, 1991).

By knowing the time of transmission and the receiving time. the pseudo-range can be written as:

$$r = V_r.\Delta t$$

where r is the pseudo range. V_r is the velocity of light and Δt is the travel time between broadcast and reception. Since the synchronization of the satellite clock and the receiver clock is not exact, the calculated range is not the true range. This is the reason for referring the calculated distance as pseudo-range (Moffit and Bouchard. 1989).

The technique of phase measurement is the way of distance measurement using the number of phase cycles over the transmission path and the phase difference at the receiver. This is done by mixing the incoming signal with a known signal generated by the receiver (Wells, 1987).

4.4.1 Stand along mode

In the stand-along mode, the process is carried out between satellites and a single GPS receiver. As the data is collected and processed by only one receiver, positioning with stand-along mode is subject to all the errors associated with GPS positioning. including clock errors in the receiver and satellites and errors due to the atmosphere and troposphere. The most common method used for stand-along mode is pseudo-ranging. This is generally known as C/A code pseudo-ranging or P code pseudo-ranging.

The accuracy of pseudo-ranging depends on the frequency (or the wave length) of the wave tracked by the receiver. The effective wave length of C/A code is 293.3 meters and the wave length of P code is one tenth of the C/A code (Moffit and Bouchard, 1991). The accuracy of pseudo-ranging in stand along mode by C/A code is known as around 20 meters and around 5 meters for the P code receivers (Leick, 1995).

4.4.2 Relative mode (differential GPS)

Relative GPS positioning must be used when accurate positioning is needed for projects such as establishing geodetic networks. Relative positioning is commonly known as differential GPS. In this technique, one GPS receiver is placed at a station that the coordinates are known to a high degree while the 2^{nd} receiver is placed at the point where the position determination is required.

Usually, data from satellites are collected at both stations and down-loaded in to a computer for processing. Software for this processing is usually provided with the GPS receivers. For "Ashtech " GPS receivers, "GPPS", "PNAV" and "Prism" are used for differential post processing. When accurate positioning is required in real time (real time differential GPS), data from the known station are transmitted to the receiver at the unknown station and processed in real time. As the data are collected at two stations and processed in differential GPS, atmospheric and ionospheric errors effecting both receivers are assumed as same, when two stations are not very far apart. So, these errors can be eliminated during the computation stage (Leick, 1995).

4.4.3 Kinematic and static modes

Differential GPS observations can be basically made using 4 different modes. They are:

1. Static mode
2. Pseudo static mode
3. Kinematic mode
4. Pseudo kinematic mode

In static mode, receivers are placed at two stations and data collection is done for a long period of time. The length of data collection depends on the type of receivers and the required accuracy in positioning. Generally the length of data collection is 45 minutes to 2 hours with 20 second epochs. For the projects which need very high positional accuracy, this length of time can be 24 hours or more (Soler and Hall. 1995).

The pseudo static mode is somewhat similar to the static mode. but the length of data collection is much shorter than the static mode. The usual period of data collection is about 15 minutes. Pseudo static mode can be used when the positioning is not required to a high precision or the data collection has to be done in a short period of time.

In the kinematic mode. the receiver at the known station (base station) is kept fixed and the other receiver or a number of receivers (rover units) move from one station to another station after the data collection for a short period of time. Usually rover units collect data from 2 to 5 minutes at one station. This technique is useful when data to be collected at a large number of points but with a high positional accuracy is not required. Generally. positional data for small scale mapping can be collected using the kinematic mode.

Pseudo kinematic mode is a modification of kinematic mode but provides much more accurate results than the kinematic mode. For this mode. a second control point is required. The rover unit is first placed at the second control point for a short period of time for initialization while one receiver is at the base station. Then the rover is moved to other points where position determination is required and finally brought back to the second control point. All this time. one receiver collects data at the base station. This mode is effectively used for photogrammetry without ground control points (Jeyapalan. 1995).

4.5 Computations in differential GPS

There are at least 4 types of GPS signal measurements that have been used for relative positioning techniques. They are "pseudo-range, carrier phase, code phase, and integrated Doppler" (FGCC, 1988). These measurements involve advanced techniques in electronics in GPS receivers. The most common way of signal measurement is pseudo ranging while the phase measurement gives the most precise measurements (FGCC, 1988). All four techniques of differential GPS involve the solution of observation equations for each measurement. By solving these equations and also using the techniques of double difference and triple difference, which are discussed later in this section, we can eliminate clock errors in receivers and satellites, ionospheric and tropospheric errors, and orbital errors of satellites. Making use of different types of measurements depend on the design of receivers for different applications, but a combination solution of pseudo-ranging and carrier phases are becoming more common (Leick, 1995).

Assume that satellites A, B, C and D as shown in Figure 4.2 are observed by a receiver at station 1. Then the pseudo-ranges from the station to each satellite can be written as (Moffit and Bouchard, 1991):

$$[(X_1 - X_A)^2 + (Y_1 - Y_A)^2 + (Z_1 - Z_A)^2]^{1/2} = r_{1A} - V_r(\Delta t_1 - \Delta t_A)$$

$$[(X_1 - X_B)^2 + (Y_1 - Y_B)^2 + (Z_1 - Z_B)^2]^{1/2} = r_{1B} - V_r(\Delta t_1 - \Delta t_B)$$

$$[(X_1 - X_C)^2 + (Y_1 - Y_C)^2 + (Z_1 - Z_C)^2]^{1/2} = r_{1C} - V_r(\Delta t_1 - \Delta t_C)$$

$$[(X_1 - X_D)^2 + (Y_1 - Y_D)^2 + (Z_1 - Z_D)^2]^{1/2} = r_{1D} - V_r(\Delta t_1 - \Delta t_D)$$

Where X, Y, Z are the earth centered cartesian coordinates of point 1 and satellites A, B, C and D; Δt is the receiver clock error at station 1; Δt_A is the satellite clock error of satellite A. Satellite clock errors are considered to be known and also can be measured by calibration (Moffit and Bouchard, 1991). When the pseudo-ranges r_{1A}, r_{1B},

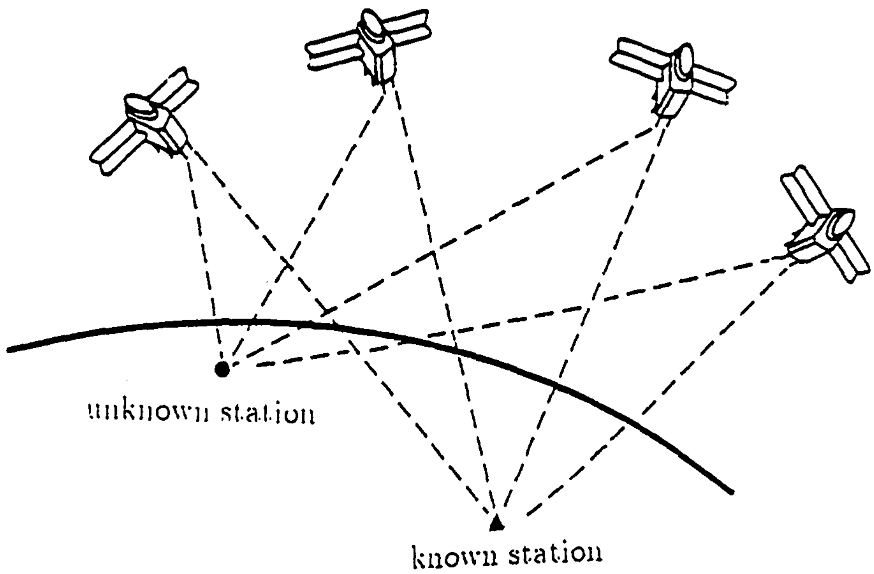


Figure 4.2 Data collection procedure for differential GPS

r_{1C} and r_{1D} are measured, only unknowns X, Y, Z and Δt_1 can be calculated by solving 4 equations to obtain a solution in the stand along mode.

These 4 equations are only for one instance of range measurement. In practice, measurements are made for a long period of time with different epochs (time intervals) depending on the receiver and required accuracy of positioning. For geodetic work, epochs ranging from a fraction of a second up to 20 seconds are used for observations. These redundant observations are then used in a least squares solution to obtain a solution for 4 parameters.

When differential GPS observations are made at a known station and at an unknown station, satellite clock errors are considered as unknowns. the unknowns Δt_1, Δt_2, Δt_A, Δt_B, Δt_C, Δt_D and also other errors due to the ionospheric and tropospheric delays can be eliminated by using the methodology of double and triple differencing, which is described later in this section.

When measurements are done using the phase difference, the difference between the

the satellite carrier phase received by the receiver and the phase of a similar wave generated in the receiver are compared and measured (Wells,1987). This measurement process cannot account for the number of whole carrier waves between the receiver and the satellites. The carrier phase observable at the receiver is a function of integer ambiguity, time taken for traveling, ionospheric and tropospheric effect, hardware delays at the satellite, and the receiver delays due to multi-path and random carrier phase measurement noises (Leick, 1995).

Taking the wave length as λ and Δe for the combined delays due to hardware at receiver and satellites, multi-path and noises (explained at the end of this section), we can write a simplified equation for a pseudo-range, using the phase measurement (Jeyapalan, 1995):

$$\phi = n\lambda + L + \Delta t_s + \Delta t_r + \Delta e$$

where n is the total number of cycles between the satellite and receiver (integer ambiguity), L is the measured phase difference, and Δt_s and Δt_r are the effects of satellite and receiver clock errors respectively. For a differential solution, if the data is collected at station 1 and 2 from satellite p, pseudo-ranges from both stations to the same satellite can be written as follows:

$$\phi_1 = n_1\lambda + L_1 + \Delta t_s + \Delta t_1 + \delta e_1 \tag{4.1}$$

$$\phi_2 = n_2\lambda + L_2 + \Delta t_s + \Delta t_2 + \delta e_2 \tag{4.2}$$

subtracting the above two equations assuming delays other than receiver clock errors are the same ($\delta e_1 = \delta e_2$) for both stations, we can write the first difference as:

$$\phi_1 - \phi_2 = (n_2 - n_1)\lambda + (L_2 - L_1) + (\Delta t_2 - \Delta t_1) \tag{4.3}$$

we can write this equation (for satellite p) in a simplified form as:

$$\Delta\phi_p = n_p\lambda + L_p + (\Delta t_2 - \Delta t_1) \tag{4.4}$$

If observations are taken from another satellite (say q):

$$\Delta\phi_q = n_q\lambda + L_q + (\Delta t_2 - \Delta t_1) \tag{4.5}$$

By subtracting equation 4.4 and 4.5, we can obtain an equation eliminating receiver clock ambiguities. This equation is known as the double difference.

$$\Delta\phi_{pq} = (n_p - n_q)\lambda + L_p - L_q \tag{4.6}$$

By measuring phase differences and writing observation equations, using parameters as coordinates of unknown station and the integer ambiguity, we can do a least square adjustment to obtain a solution for parameters.

By adding any errors due to integer ambiguities ΔN_{pq} to the equation obtained from the double difference at time t_1 and t_2,

$$\Delta\phi_{pq}^{t1} = (n_p - n_q)\lambda + (L_p - L_q) + \Delta N_{pq} \tag{4.7}$$

$$\Delta\phi_{pq}^{t2} = (n_p - n_q)'\lambda + (L_p - L_q)' + \Delta N_{pq} \tag{4.8}$$

The difference between equations 4.7 and 4.8 at 2 different epochs is known as the triple difference. Writing observation equations for the triple difference solution, again we can do a least square solution for unknowns. Both double difference and triple difference solutions give satisfactory results in geodetic accuracy, but because of the additional differencing over time, the triple differences lose some geometric strength (Leick, 1995).

4.6 Errors effecting GPS positioning

Positional accuracy of GPS can be affected by a number of factors. They are (Puterski, 1992):

1. Geometry of observing satellites
2. Variations in satellite and receiver clocks

3. Errors in satellite ephemerides
4. Errors due to the electronics of receivers
5. Effects by the ionosphere and troposphere
6. Cycle slips
7. Multi-path effect
8. Selective availability

When differential GPS is used, many of the effects of these errors can be mathematically eliminated during data processing.

Geometry of satellites is the position of satellites in space at the time of data collection, relative to each other and relative to receivers. Widely spread positions of satellites give better results. This effect is expressed in GPS as GDOP (Geometric dilution of Precision). The effect of GDOP in three dimension is expressed as PDOP (Position dilution of precision) (Wells, 1987). PDOP is measured in a scale from 0 to 10, where 0 gives the best positional accuracy.

Measurement of pseudo-ranges depends on the satellite clocks and the clock in the receiver. Satellite clocks are highly accurate atomic clocks but the receiver clocks are built less accurately than satellite clocks due to the cost factor. Cesium atomic clock loses only one second in 300,000 years(Leick, 1992). GPS time is realized by an atomic clock maintained at the master control station at Colorado Springs, Colorado. There is 1 micro second difference between the GPS time and the Universal Coordinated time (UTC) (Leick, 1995).

A temporary interruption in GPS data collection is referred to as a cycle slip. When the data is collected for a significantly long period of time, a short cycle slip can be neglected but longer cycle slips have to be compensated with additional time periods of data collection. Usually receivers are capable of informing the observer about cycle slips, using a sound signal. For differential GPS, data can be edited before post processing in order to eliminate the effect of cycle slips.

Multi-path is the effect of large reflecting surfaces close to a GPS receiver. This error is caused by a reflected GPS signal received by the receiver in addition to the direct signal. This error has to be minimized during the reconnaissance stage. So, the locations of GPS stations have to be selected away from large metal buildings and considerably large bodies of water.

The operation of GPS satellites is performed by the United States Department of Defense (DOD). DOD can intentionally degrade the clock and ephemeris signals from satellites. This is known as selective availability. The effect of selective availability of positioning with C/A code in stand along mode can be somewhere between 50 to 200 meters (Puterski, 1992). Selective availability has only a very little effect on differential GPS (FGCC, 1988).

Ionosphere is generally considered to be the region of the atmosphere from approximately 50 to 1000 km in altitude and the troposphere is known as the atmosphere close to the earth surface, generally up to 80 km. A GPS signal travels through ionosphere and the troposphere can be affected by the ionized medium of the ionosphere (Wells, 1987) and the refraction by the troposphere. The ionospheric and tropospheric effects can be eliminated in differential GPS, but it effects the position determination by stand along mode. This error can be in the range from 1 to 10 ppm (FGCC, 1988).

4.7 Specifications of differential GPS

Specifications for GPS surveying in the United States are determined and published by the Federal Geodetic Control Committee (FGCC). According to the preliminary document published by FGCC in 1988, which is the latest version of specifications available today, GPS surveys have been categorized into 4 major orders. They are called AA, A, B and D. Out of these 4 categories only AA, A and B are considered as geodetic level networks. Expected absolute accuracy of each of these networks is given in Table 4.2

Table 4.2 Expected absolute accuracy of GPS networks

Order	Expected accuracy	
AA	1 : 100,000,000	0.01 ppm
A	1 : 10,000,000	0.1 ppm
B	1 : 1,000,000	1 ppm
D-1	1 : 1,00,000	10 ppm
D-2-I	1 : 50,000	20 ppm
D-2-II	1 : 20,000	50 ppm
D-3	1 : 10,000	100 ppm

(FGCC,1988).

4.8 Getting GPS data in to a GIS

GPS gives highly accurate positional data for a GIS, but a few more steps are needed before using them with other spatial and attribute data in the GIS. Two major concerns are the reference ellipsoid and the map projection. As explained in chapter 3, reference ellipsoid is the mathematical representation of the earth, which is used for the calculation of latitudes and longitudes. Plane coordinates are calculated using these latitudes and longitudes, which are used for large scale mapping.

Many different reference ellipsoids are being used for different parts of the world. Before 1983, the U.S. used the Clerk's (1866) ellipsoid. So, NAD27 coordinates are based on the clerk's ellipsoid. NAD83 coordinates were calculated using GRS80 ellipsoid. This ellipsoid is the same ellipsoid used by GPS. Currently, Sri Lanka uses the Everest ellipsoid and hence all maps prepared by Sri Lanka are based on the Everest ellipsoid. The effect of different ellipsoids at different locations varies and has to be corrected using datum transformation techniques explained in chapter 6. With the wide use of GPS technology in many civilian applications, today's trend is to use a single ellipsoid for the entire world. Achieving this objective is far from over and especially developing countries have to do much more work in this area of the subject. It will take many more years to establish a common ellipsoid for the whole world and prepare maps using this common ellipsoid. Until then, GPS data will have to be adjusted for the relevant ellipsoid which other maps of the GIS data are based on.

Maps prepared in the U.S. using NAD83 coordinates have the same reference ellipsoid as GPS. Therefore, the GPS data are compatible with the coordinates given in new maps. Older maps of the U.S. as well as maps of Sri Lanka do not have the same reference ellipsoid as GPS and have to be brought to the same ellipsoid before using them for data analysis in the GIS.

A satisfactory GIS must have a variety of spatial and attribute information. Today, the primary way of collecting spatial data for a GIS is from hard copy maps, which are prepared using a map projection. Positional data collected by GPS are directly on the reference ellipsoid and are not related to any map projection. They are usually given as latitudes and longitudes (geodetic coordinates) or as earth centered 3 dimensional cartesian coordinates. Therefore, these geodetic or cartesian coordinates have to be transformed to the same map projection used in the GIS.

Many popular GIS packages have the capability of transforming ellipsoidal or cartesian coordinates to a given map projection. For example, Arc Info Rev 6.0 is equipped with the NADCON ellipsoidal transformation module which is developed by NGS (ESRI, 1995). Thus, maps prepared on NAD27 datum can be transformed to the GRS80 datum. In addition, almost all popular GIS software has the capability of transforming coordinates from one map projection to another map projection.

5 A NEW GEODETIC CONTROL SYSTEM

According to the analysis done in Chapter 3, it appears that the existing geodetic control of Sri Lanka does not satisfy the needs of GIS. The network is not suitable as the linkage mechanism. Also, it is not suitable for GPS observations and long range EDM observations, which will be major techniques for spatial data collection for the establishment and maintenance of future GIS. Therefore, the geodetic network of Sri Lanka has to be strengthened and a new set of coordinates adopted before any significant work can be started for data collection for GIS. This chapter discusses the procedure for the establishment of a new geodetic control network for Sri Lanka.

5.1 Reconnaissance

Establishment of a Geodetic control network starts with reconnaissance. In this stage, points with coordinates to be determined are physically selected on the ground. The success of the geodetic control network is greatly dependent on reconnaissance because the collection of data, cost of data collection, accuracy of the network and future use of the network depend on the locations of points of the control network. In early days, when triangulation was the only means of establishing geodetic networks, reconnaissance was basically concentrated on 2 factors. They were:

1. The strength of the figure
2. Inter visibility among stations

A considerable attention was paid to the geometric strength of figure because a network with equilateral triangles was considered the strongest. Generally triangles with small angles such as 10^0 or 15^0 degrees were considered weak. Consideration of the strength of the figure was important because the influence of small angles on trigonometric formulas used for the calculation of distances was proportionately higher than larger angles. The errors of measurements of small angles made a disproportionately large contribution to the errors of calculation of lengths. Today, with the use of EDM instruments and GPS, paying attention to the strength of a figure has become unnecessary.

Inter-visibility between stations is important for angle measurements as well as distance measurements using EDM instruments, but is not required for GPS. Inter visibility depends on 3 factors:

1. Earth curvature
2. Obstructions due to terrain conditions
3. Secondary obstructions such as trees

Maximum possible sighting distance between two stations due to the earth curvature can be mathematically calculated. But the geographical area of Sri Lanka is not very large and satisfactory control points have already been selected for previous adjustments. Thus, the effect of the earth curvature is not a major concern for the new geodetic control adjustment in the country.

Obstructions due to local conditions such as trees are important even for GPS observations, because the mask angle for GPS observations has to be used as 10^0 to 15^0 degrees for geodetic measurements (FGCC, 1988). Hence, all the stations of the old network has to be checked for local obstructions, as a part of the reconnaissance of the new network.

Stations used in geodetic networks established by triangulation are generally located on hill tops or remote areas. This was due to the easy sighting between stations. Also, this way of selecting points provided a better protection for monuments used in early geodetic networks. As the long term protection of control points (monuments) is one of the major concerns in the reconnaissance stage, it is important to establish them in protected areas. Another important consideration is the easy access of monuments for day to day work. These two factors usually contradict each other. For the long term safety of monuments, it is better to establish them away from populated areas but for easier access, they have to be close to population centers. With the usage of GPS for geodetic observations we can select locations for control points in any convenient place.

Although the strength of the figure is not important for GPS networks, the number of vectors observed for one station play a major role when the network is adjusted. For example, if the location of a station is determined using only two vectors of GPS observations, the positional accuracy of that station can be weak. GPS observational requirements for Sri Lanka, which are discussed in section 5.3.4 are important when dealing with this problem. Therefore, the planning of the observational procedure has to be done with great care, in order to achieve good positional accuracy for the network.

5.1.1 Field measurements used for old networks

Older geodetic networks established in the 1960s or before were established using triangulation (Kahmen and Faig, 1988). Triangulation is based on the measurement of two or more base lines, a few astronomical azimuth observations and the angular observations in all the triangles. The measurement procedure for base lines, which usually used "Invar" tapes, were very tedious, time consuming and costly. Angle observations were done in the night to avoid the effect of refraction, using suitable light sources as targets. For all these reasons, the workload for observations of geodetic surveys was very large. The average time for angle observations at one station was 6 weeks for the observation of the German first order network (Kahmen and Faig, 1988).

The existing Sri Lankan primary geodetic network has 110 stations. Although some of them cannot be used for angle observations due to obstructions such as buildings, only a very few stations have been destroyed during the last 64 years (since the last adjustment). Further, almost all stations of the Sri Lankan network are established on permanent natural features such as huge rocks. Hence, the possibility of disturbances to monuments is low. Therefore, it is economical and fast to use all available old stations for the new network. Also, it will facilitate the calculation of accurate transformation parameters between two systems.

5.2 Geodetic control and its users

Good geodetic control is an essential part of a GIS. Hence. All the GIS users are direct beneficiaries of a good geodetic control system. Even without a GIS, geodetic control plays an important role in many disciplines. Users of a geodetic network can be basically grouped in to three categories (Dewhurst, 1990):

1. Primary users: those who employ the coordinate information directly. Geodesists, Geographers and Land Surveyors are in this category.
2. Secondary users: those who employ the work of the primary users. They usually add information and convert the work of primary users into a more general and usable form. Cartographers and digital map makers fall into this category.
3. Tertiary users: those who use the work of secondary users for learning, planing and many engineering works. Engineers, planning specialists, attorneys, students, social scientists, economists and legislators can be put in to this category.

5.3 GPS observations for new adjustment

Frequent changes of coordinates of a geodetic network are not possible due to many different levels of users in the country. Hence, when a coordinate system is established, it must be good for present as well as future applications. Therefore, coordinates of a geodetic network have to be calculated with the highest possible accuracy. GPS has been proven the best and most accurate data collection method available today for establishing geodetic networks (Leick 1995, Puteski 1992, Wells 1987), if the correct procedure is adopted for GPS observations.

In the U.S., the new geodetic control network adjusted in 1983 (NAD83) is now considered obsolete, mainly due to the obtainable accuracy of GPS. NAD83 was established using a very little GPS observation (Snay, 1989), due to the availability of this new technology during that time. Therefore, Sri Lanka must use GPS for establishing the new geodetic network if the country is to avoid another geodetic project within a short period of time.

5.3.1 Expected and predicted accuracy of the network

Regarding the new geodetic control in the U.S., NAD83, Schwartz (1983, p9) says,"as the number of types of coordinate users increased with the rapid population growth, so did their accuracy needs. The number of coordinate users have rapidly increased due to the need of growing population centers. These population centers need accurate maps for tax assessment and land use planning and the construction and maintenance of sewer and water supply lines, highways, bridges, tunnels, telephone lines, pipe lines, power transmission lines and many other related services."

Also, accuracy needs were increased due to the availability of modern technology for surveying, mapping and other engineering areas. The increase of land values due to the population growth is another contributing factor to the demand of more accurate geodetic control.

In the early part of this century, even up to the 1960s, geodetic control was established using purely "Triangulation" techniques. Those networks satisfied the needs of mapping, boundary determination and many other construction work such as highways, dams and irrigation projects. Accuracy expectations of those networks were not as high as today, due to the non availability of convenient and accurate long range distance measurement techniques. Accuracy expected by those geodetic networks was around 1 part in 50,000 but by 1983 this demand increased up to the accuracy of 1 part in 100,000 (Schwarz, 1983).

Today, with the wide usage of GPS technology, this demand has increased to at least 1 part in 1 million. The HARN (High Accuracy Reference Network) project, which is being established in the state of Iowa and throughout the U.S., is aiming at accuracy of 1 part in 10 million, hoping that it will at least satisfy the needs of geodetic control in the early part of 21^{st} century (A order accuracy by FGCC, 1988).

Geodetic control of a country is determined by calculating latitudes and longitudes of a number of permanently marked points. As these points were established using triangulation, they are usually known as triangulation points or some times as primary control points. In the triangulation, a few lengths (at least two) between network points, which are known as "base lines", were measured to high accuracy. The method used for these length measurements was very tedious and time consuming. Then almost all the angles in the network were observed. By using these measurements and fixing one point usually called the "datum point", geodetic coordinates of other points were calculated using the principal of "least square". Geodetic coordinates of the datum point were usually assumed as equal to the values obtained using astronomical methods.

Once the geodetic coordinates for a country are finalized, they are usually published by the government of the country. In the U.S., the National Geodetic Survey (NGS) is the government agency which performs this work and in Sri Lanka, it is the Survey Department. Once the coordinates are published, they become permanent coordinates of the triangulation points until a re-calculation is done and a new set of coordinates are published. The general understanding in the past was that the coordinates published can be used without a change for about 50 years, but the rapid development of technology has made it impossible to keep up this time frame. In the U.S., NAD27 (adjusted in 1927) was replaced by NAD83 after 56 years but now the HARN coordinates are going to replace them again after only 16 years.

Sri Lanka uses the coordinates calculated and published in 1932. Thus, for the last 64 years, geodetic needs of the country were fulfilled by these coordinates, although they could not cope with the present needs of surveying and engineering technologies. The Survey Department of Sri Lanka has proposed to established a new geodetic network consisting of 240 points, including about 50 points used for the 1932 adjustment. Observations for the new adjustment using long range EDM (Electro Distance Measurement) instruments and GPS were expected to be commenced in the early part of 1996.

As mentioned in Chapter 1, an absolute accuracy of 1 : 1,000,000 in primary control points will provide sufficient geodetic control for GIS applications. As the average distance between the primary control points is about 30 km, 2 cm absolute accuracy of points will provide the accuracy of 1 ppm for the Sri Lankan network.

It is extremely useful if the final accuracy of the network can be predicted before the adjustment, according to the quality of GPS observations. Then, the observation procedure for data collection can be adjusted to get the desired accuracy. In order to do this prediction, simulated sets of GPS observations were calculated and a least squares adjustment was performed (using "Geolab"). Using this method the final accuracy of the Sri Lankan network could be predicted for a number of GPS observation methods. The complete procedure for obtaining simulated data and the predicted accuracies of Sri Lankan network are discussed in next two subsections.

5.3.2 Simulated data

Positional data obtained by differential GPS (see Chapter 4) are as follows:

1. Earth centered X,Y,Z coordinate differences for vectors between stations ($\Delta X, \Delta Y, \Delta Z$). These coordinates are based on the GPS ellipsoid (GRS80).

2. Mark to mark distances between stations.
3. Azimuth between two stations according to the GPS ellipsoid
4. Ellipsoidal height difference according to the GPS ellipsoid

Out of all these data, $\Delta X, \Delta Y$ and ΔZ between stations are calculated by GPS first, and then other data are derived using geodetic formulas and the GPS ellipsoid (Jeyapalan, 1995).

Simulated mark to mark distances and $\Delta X, \Delta Y, \Delta Z$ were obtained as follows:

1. A least squares adjustment was performed using all currently available data to obtain latitudes and longitudes of stations
2. Mark to mark distance and $\Delta X, \Delta Y, \Delta Z$ were calculate using latitudes and longitudes.

The software "Con_cord" mentioned in Chapter 1 was used for the calculations. Also, "Geolab", the least squares adjustment software, provides $\Delta X, \Delta Y, \Delta Z$ and mark to mark distances for adjoining stations.

These distances and $\Delta X, \Delta Y, \Delta Z$ were simulated (introduced random errors according to a normal curve) using the following mathematical model:

$$S_s = S_A \pm \sigma \pm n$$

where, S_s = Simulated values

S_A = Values before the simulation

σ = Random error introduced according to a selected std. deviation

n = Error introduced as a ppm correction

Values of σ were obtained using the random values generation command "rnorm" in the statistical package "Splus"(Statistical Science Inc. 1993), using the mean as zero and the standard deviation as 1 mm for "A" order control points and 5 mm for "B" order points. n, the ppm error were used as 1 ppm or .01 ppm in different cases as shown in Table 5.1.

Table 5.1 Positional accuracy obtained for "A" order points, using different simulations

Number of points	Point numbers	Random error introduced mm	ppm error introduced mm	Average positional accuracy cm	Range of positional accuracy cm
4	34, 77, 88, 104	1	10^{-8}	.50	.42 - 0.59
4	1, 77, 88, 104	1	10^{-8}	1.48	1.41 - 1.58
5	34, 77, 80, 88, 104	1	10^{-8}	.38	.28 - 0.53
5	34, 77, 80, 88, 104	1	10^{-6}	6.05	5.38 - 7.56
6	1, 34, 77, 80, 88, 104	1	10^{-8}	1.25	0.82 - 1.92

5.3.3 Fixed stations for the adjustment

When a geodetic adjustment is performed, one or more points have to be fixed and the coordinates have to be taken as correct. These points are called fixed points or "datum" points. Only one datum point was used for the adjustment performed in earlier days. For the 1932 adjustment in Sri Lanka, "Kandawala" was used as the datum point and for the NAD27 in the U.S. the station "Meades Ranch" in Kansas was the datum point.

When only one station is fixed, coordinates of all other stations are calculated relative to the fixed station. Theoretically, fixing more stations should give more accurate results, but before the technology of GPS, this theory was not practical. These fixed points are the 100 km apart "A" order points in the recommended Sri Lankan geodetic network.

We can fix more than one point, if only the coordinates of those points can be calculated to a very high accuracy. At least 1 cm absolute accuracy is required for fixing points in the primary network adjustment. Working with simulated data with 1 mm random error and different ppm errors, absolute accuracies were obtained for "A" order points as shown in Table 5.1.

Figure 5.1 shows the location of these points. It can be seen from the Table 5.1 that a 1/2 cm positional accuracy can be obtained for a few control points, if random errors can be limited to 1 mm and GPS observations can be taken to the accuracy of .01 ppm. This indicates that the Sri Lankan geodetic network can be adjusted using 4 or 5 fixed points taking GPS observations at those locations to the accuracy of 0.01 ppm and limiting random errors to 1 mm.

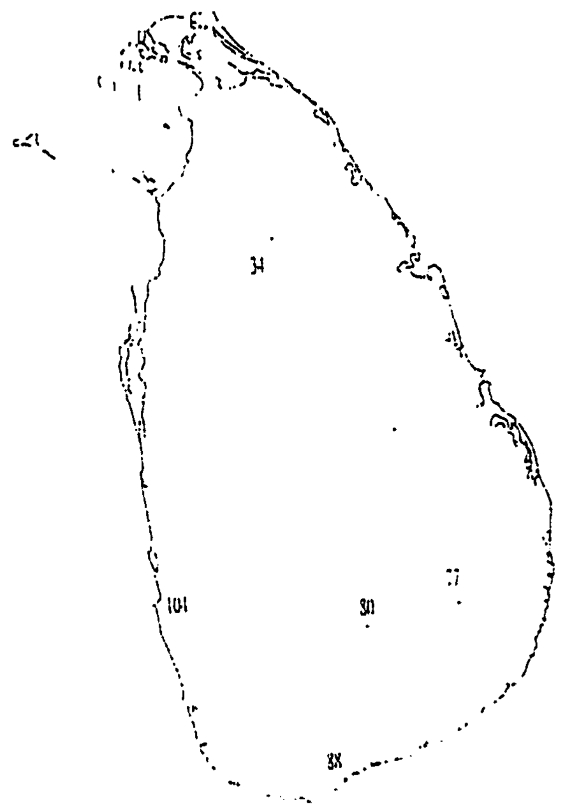


Figure 5.1 Locations of points chosen for fixing

5.3.4 Procedure for GPS observation to achieve the required positional accuracy of the network

The goal of the proposed geodetic control project is to achieve 2 cm absolute accuracy for all primary points. Hence, the GPS observation procedure has to be designed to achieve this objective.

A number of distance measurement methodologies were tested using different types of simulated data to see the final positional accuracy of the network. Standard deviation of 5 mm (for random errors) was used for data simulation. Different methodologies used, ppm errors and the positional accuracies obtained in each case are given in Table 5.2.

Table 5.2 shows that the GPS observation taken using triangulation lines (see Appendix A), with 5 mm random errors and 10^{-10} ppm error will provide approximately 6 cm absolute accuracy for primary control points. The same procedure with 10^{-8} ppm provides only 24.9 cm accuracy for primary control points. Also, simulated data shows that a better positional accuracy can be obtained with only 10^{-8} ppm errors, if observations are not limited to triangulation lines. It was possible to obtain an average of 2.68 cm positional accuracy for the network, when all adjoining lines shorter than 75 km were included in the adjustment. In this adjustment, the lowest positional accuracy obtained was only 5.60 cm.

Table 5.2 Positional accuracy obtained for "B" order points using different GPS observation methods.

Methodology used	Number of lines	Number of fixed stations	Random error introduced (mm)	ppm error introduced (mm)	Average positional accuracy (cm)	Range of positional accuracy (cm)
Triangulation lines	110	4	5	10^{-8}	24.94	14.39 - 40.04
		4	5	10^{-10}	6.40	1.25 - 21.74
Including additional lines(¡ 75km)	127	4	5	10^{-6}	14.75	5.8 - 27.49
		4	5	10^{-8}	2.64	1.03 - 5.69
		4	5	10^{-10}	2.68	1.00 - 5.60
All Possible lines	2500	4	5	10^{-8}	8.92	3.28 - 23.68

The accuracy of 2.68 cm at stations easily provides the accuracy of 1 : 1,000,000 for the network, when compared with the average length of lines in the network. Therefore, the following procedure can be recommended in order to achieve a 1 : 1,000,000 absolute accuracy for the primary network.

1. Fix 4 points in 4 corners, which were calculated to the positional accuracy of 1/2 cm as shown in Figure 5.1
2. Limit random errors to 5 mm
3. Observe all possible adjoining distances which are shorter than 75 km using a GPS procedure which gives 0.01 ppm accuracy

5.3.5 Pre-adjustment using simulated data

For the prediction of co-ordinates for the Sri Lankan primary network, a least square adjustment was performed using simulated values. These simulated values were obtained using the procedure explained in section 5.3.2.

Using the Reference ellipsoid as "Everest", these simulated data for 50 stations were used to calculate the co-ordinates of fifty stations in the Sri Lankan primary network. Station numbers 34, 77, 80, 88 and 104 which were shown in figure 5.1 were treated as fixed. The data were processed using "Geolab". At the stage of simulating the GPS data, the elevations of the stations were not known to any reasonable degree of correctness. So, they were excluded from the purview of this evaluation and only the planimetric location have been considered. The results using these simulated values were compared with adjusted values obtained using actual data and the results are discussed in section 5.3.6.

5.3.6 Observation and processing of actual GPS data

GPS observations were performed at 445 stations mentioned in the previous section. These data were collected as a readjustment project to the primary control network of Sri Lanka launched by the Survey Department. Five Wild GPS receivers (System 300 - Model SR399E) instruments were used for observations (see figure 5.2).

Except at the station 111, Samadi ISMD, the data collection time at each station was 3 hours. Seven days continuous data collection were done at the Samadi ISMD and

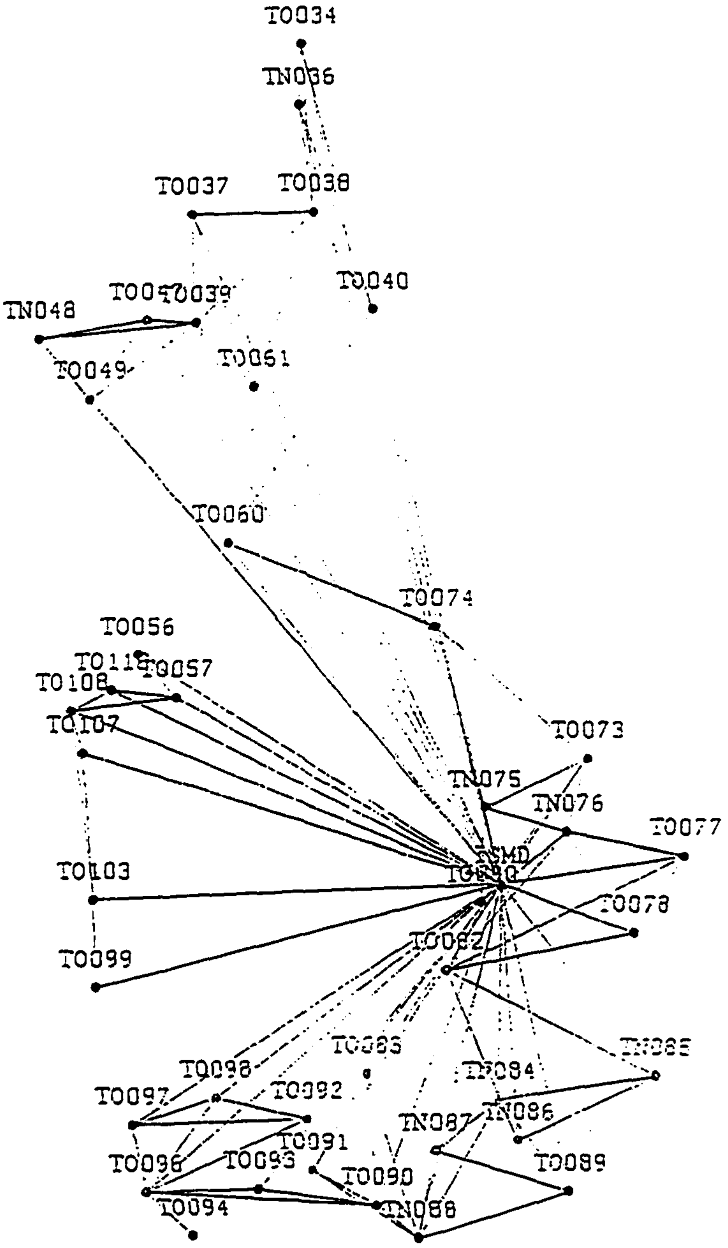


Figure 5.2 Stations used for new GPS observations. TO are old trignometrical stations and TN are newly established points.

data were processed in the single point mode. It is expected that the absolute accuracy obtained for this point by 7 days data collection is one meter. GPS data collected at the above mentioned trigonometrical stations were processed using two different methods.

In method I, the station Samadi ISMD, coded 111, was considered as fixed. Then the data of 45 stations were processed using "Geolab" to solve GPS vectors and to determine the co-ordinates in terms of latitudes and longitudes of each station.

Two steps were used to perform method II. In the first step, the vectors that were observed more than once, from the base station were isolated and and corresponding stations were identified using GPS observed vectors shown in figure 5.2. There were a total of 18 such stations. The observation data for these 18 stations were processed treating station 111 , Samadi ISMD as fixed and all other seventeen stations as free and co-ordinates of seventeen stations were determined in terms of latitude, longitude and elevation of each station. The input file and the output file for this adjustment are given in Appendix E and Appendix F respectively. The results obtained by the above step for 18 stations are free from the effects of possible errors in measurement vectors that were measured only once. Subsequently they are treated as fixed to adjust all the observed stations.

A comparison of results obtained through the above mentioned two methods is given in table 5.3. The mean difference between the two methods shows only .0005579 seconds in latitude and 0.000513 seconds in longitude. The standard error is 0.00339seconds in latitude and 0.00462 in longitude. The values are considerably small. So the results obtained by the two methods do not show a significant difference.

A comparison of results obtained by adjusting simulated data with the results obtained by processing observed GPS data (fixing only the base station), given in table 5.4, indicates a mean difference of 1.19845 seconds in latitude and 7.744308 seconds in longitude. The standard error is 0.13117 seconds in latitude and 0.21002 seconds in longitude. The major reason for these differences can be due to the datum shift between

Table 5.3 Adjustment by fixing one point and eighteen points.

Stn ID	Station Name	1_point fixed Adjusted Values Latitude			Longitude			Adj Elev	18-point fixed adjusted value Latitude			Longitude			Adj Elev	Lat Diff sec	Long Diff sec	Elev Diff
111	SAMADI_ISMD																	
34	IRATTAPEKULA	8	42	24.757215	80	29	4.604413	34	8	42	24.759336	80	29	4.613210	34	-0.002121	-0.008797	-0
36	ISSENBESSAWAGAL	8	34	6.635614	80	28	52.714121	63	8	34	6.637746	80	28	52.722915	63	-0.002132	-0.008794	-0
37	BOGAHAWEWA	8	19	17.532417	80	14	3.425888	2	8	19	17.532289	80	14	3.429650	2	0.000128	-0.003762	-0
38	KATUPOTAKANDA	8	19	43.897771	80	31	6.353465	302	8	19	43.897530	80	31	6.362270	302	0.000241	-0.008805	-0
39	TAMBUTTKANDA	8	4	35.293906	80	14	34.658723	180	8	4	35.293760	80	14	34.661190	180	0.000146	-0.002467	-0
40	RITIGALA	8	6	34.135726	80	39	23.753609	666	8	6	34.142440	80	39	23.771130	667	-0.006714	-0.017521	-0
47	MADAMOLA	8	5	4.537904	80	7	54.107387	160	8	5	4.537831	80	7	54.108621	160	0.000073	-0.001234	-0
48	CROWS_NEST	8	2	34.513212	79	53	8.594639	-29	8	2	34.513141	79	53	8.595875	-29	0.000071	-0.001236	-0
49	PARAMAKANDA	7	54	23.286765	80	0	10.384913	60	7	54	23.286694	80	0	10.386147	60	0.000071	-0.001234	-0
51	MEDAGAMA	7	34	27.297564	80	7	38.386485	216	7	34	27.294462	80	7	38.385883	215	0.003102	0.000602	0
56	NARANGAMA	7	19	51.902324	80	6	53.870618	97	7	19	51.893230	80	6	53.861550	97	0.009094	0.009068	0
57	ENGODA	7	14	7.508936	80	12	10.427022	209	7	14	7.484730	80	12	10.400460	208	0.024206	0.026562	1
59	AMBOKKA	7	36	17.796710	80	34	39.632167	1132	7	36	17.797824	80	34	39.633671	1132	-0.001114	-0.001504	-0
60	YAKDESSAGALA	7	34	59.112490	80	19	18.974153	422	7	34	59.123250	80	19	18.989210	422	-0.010760	-0.015057	-0
61	GALGIRIYA	7	56	4.343467	80	22	52.682986	471	7	56	4.343344	80	22	52.686741	471	0.000123	-0.003755	-0
73	DORAPATAGALA	7	5	59.503455	81	9	42.098141	1415	7	5	59.504070	81	9	42.098890	1415	-0.000615	-0.000749	-0
74	KNUCKLES	7	23	46.937584	80	48	34.850771	1762	7	23	46.939820	80	48	34.853770	1762	-0.002236	-0.002999	-0
75	GOMMOLIYA	6	59	16.637164	80	55	14.777842	1581	6	59	16.637850	80	55	14.778510	1581	-0.000686	-0.000668	-0
76	NAMUNUKULA	6	55	58.843032	81	6	52.715544	1934	6	55	58.843190	81	6	52.715810	1934	-0.000158	-0.000266	-0
78	UGALA	6	42	26.214325	81	16	17.911791	475	6	42	26.214378	81	16	17.911887	475	-0.000053	-0.000096	-0
80	BERAGALA_NORTH	6	46	26.387026	80	54	51.378719	1678	6	46	26.387393	80	54	51.379140	1678	-0.000367	-0.000421	-0
82	KIRIOLUHENA	6	37	17.947623	80	50	5.540478	632	6	37	17.947630	80	50	5.540510	632	-0.000007	-0.000032	0
83	GONGALA	6	23	9.044697	80	39	14.478505	1258	6	23	9.044654	80	39	14.478108	1258	0.000043	0.000397	-0
84	HABURUGALA	6	19	49.093556	80	57	34.439380	3	6	19	49.093490	80	57	34.439360	8	0.000066	0.000020	-5
86	KARAMBAGALA	6	14	17.281771	81	0	16.604945	19	6	14	17.281846	81	0	16.604902	20	-0.000075	0.000043	-2
88	KADURUPOKUNA	6	0	54.571854	80	46	40.749550	-39	6	0	54.571720	80	46	40.749180	-39	0.000134	0.000370	-0
89	HAMBANTOTA_T	6	7	19.402884	81	7	36.966446	-70	6	7	19.403033	81	7	36.966143	-68	-0.000149	0.000303	-2
90	AMANGALAKANDA	6	5	23.631969	80	40	45.406161	124	6	5	23.631980	80	40	45.405340	124	-0.000011	0.000821	-0
91	URUMUTTA	6	10	6.039856	80	31	40.985065	329	6	10	6.039814	80	31	40.984669	330	0.000042	0.000396	-0
92	MORAWAKA	6	17	4.413762	80	30	50.876483	623	6	17	4.413811	80	30	50.875534	623	-0.000049	0.000949	-0
93	HINDEKNATU	6	7	31.850067	80	24	5.125776	307	6	7	31.850103	80	24	5.124870	307	-0.000036	0.000906	-0
94	GALLE_TOWER	6	1	5.791578	80	14	43.427665	-31	6	1	5.791600	80	14	43.426960	-31	-0.000022	0.000705	-0
95	KURUNDAKANDA	6	4	46.323352	80	14	4.250551	-13	6	4	46.323388	80	14	4.249645	-13	-0.000036	0.000906	-0
96	AMUNERIYAGODA	6	7	2.702443	80	8	13.839409	-67	6	7	2.702540	80	8	13.837510	-67	-0.000097	0.001899	-0
97	PANILKANDA	6	16	12.515713	80	6	19.824587	-49	6	16	12.515761	80	6	19.823637	-49	-0.000048	0.000950	-0
98	HAYCOCK	6	19	55.590030	80	17	58.768263	558	6	19	55.590078	80	17	58.767313	558	-0.000048	0.000950	-0
99	BOMBUWALA	6	34	49.429162	80	1	13.067383	55	6	34	49.428112	80	1	13.066260	55	0.001050	0.001123	0
103	OLEBODA	6	46	45.808193	80	0	53.589338	11	6	46	45.806100	80	0	53.587090	11	0.002093	0.002248	0
107	ASGIRIYA	7	6	32.738389	79	59	24.399076	-34	7	6	32.735574	79	59	24.396100	-34	0.002815	0.002976	0
108	ALUTAIPOLA	7	12	18.963643	79	57	48.201704	-47	7	12	18.957290	79	57	48.195020	-47	0.006353	0.006684	0
														Mean		0.0005579	-0.000513	-0.2
														rms of difference=		0.0047956	0.0065330	0.96
														rms of single value=		0.0033910	0.0046195	0.68

Table 5.4 Adjustment of simulated and actual GPS observations.

		Old Adjusted Values							New Adjusted Values							Lat Diff	Long Diff	Elev Diff
Stn ID	Station Name	Latitude			Longitude			Altitude	Latitude			Longitude			Adj Elev	sec	sec	
111	SAMADI_ISMD																	
34	IRATTAPEKULA	8	42	23.859900	80	28	57.164788	134	8	42	24.757215	80	29	4.604413	34	-0.897315	-7.439625	100
36	ISSENBESSAWAGAL	8	34	5.203913	80	28	45.782570	100	8	34	6.635614	80	28	52.714121	63	-1.431701	-6.931551	37
37	BOGAHAWEWA	8	19	16.580340	80	13	55.872460	105	8	19	17.532417	80	14	3.425888	2	-0.952077	-7.553428	103
38	KATUPOTAKANDA	8	19	42.937000	80	30	58.909000	404	8	19	43.897771	80	31	6.353465	302	-0.960771	-7.444465	101
39	TAMBUTTKANDA	8	4	34.294178	80	14	27.110663	281	8	4	35.293906	80	14	34.658723	180	-0.999728	-7.548060	101
40	RITIGALA	8	6	33.153685	80	39	16.374435	768	8	6	34.135726	80	39	23.753609	666	-0.982041	-7.379174	101
47	MADAMOLA	8	5	3.539557	80	7	46.517521	264	8	5	4.537904	80	7	54.107387	160	-0.998347	-7.589866	104
48	CROWS_NEST	8	2	33.513049	79	53	0.912860	100	8	2	34.513212	79	53	8.594639	-29	-1.000163	-7.681779	129
49	PARAMAKANDA	7	54	22.258701	80	0	2.746281	164	7	54	23.286765	80	0	10.384913	60	-1.028064	-7.638632	104
51	MEDAGAMA	7	34	26.250752	80	7	30.870805	100	7	34	27.297564	80	7	38.386485	216	-1.046812	-7.515680	-116
56	NARANGAMA	7	19	50.737116	80	6	46.260501	201	7	19	51.902324	80	6	53.870618	97	-1.165208	-7.610117	103
57	ENGODA	7	14	6.338846	80	12	2.862389	309	7	14	7.508936	80	12	10.427022	209	-1.170090	-7.564633	101
59	AMBOKKA	7	36	16.733058	80	34	32.201407	100	7	36	17.796710	80	34	39.632167	1132	-1.063652	-7.430760	-1032
60	YAKDESSAGALA	7	34	58.048355	80	19	11.477374	526	7	34	59.112490	80	19	18.974153	422	-1.064135	-7.496779	104
61	GALGIRIYA	7	56	3.320560	80	22	45.190946	572	7	56	4.343467	80	22	52.682986	471	-1.022907	-7.492040	101
73	DORAPATAGALA	7	5	58.365809	81	9	34.930412	1517	7	5	59.503455	81	9	42.098141	1415	-1.137646	-7.167729	102
74	KNUCKLES	7	23	45.847294	80	48	27.537084	100	7	23	46.937584	80	48	34.850771	1762	-1.090290	-7.313687	-1662
75	GOMMOLIYA	6	59	15.472410	80	55	7.529257	100	6	59	16.637164	80	55	14.777842	1581	-1.164754	-7.248585	-1481
76	NAMUNUKULA	6	55	57.711201	81	6	45.567178	100	6	55	58.843032	81	6	52.715544	1934	-1.131831	-7.148366	-1834
78	UGALA	6	42	25.009359	81	16	10.810282	576	6	42	26.214325	81	16	17.911791	475	-1.204966	-7.101509	101
80	BERAGALA_NORTH	6	46	25.176000	80	54	44.130000	1778	6	46	26.387026	80	54	51.378719	1678	-1.211026	-7.248719	100
82	KIRIOLUHENA	6	37	16.692197	80	49	58.281216	734	6	37	17.947623	80	50	5.540478	632	-1.255426	-7.259262	102
83	GONGALA	6	23	7.728787	80	39	7.170754	1361	6	23	9.044697	80	39	14.478505	1258	-1.315910	-7.307751	102
84	HABURUGALA	6	19	47.791246	80	57	27.252158	100	6	19	49.093556	80	57	34.439380	3	-1.302310	-7.187222	97
86	KARAMBAGALA	6	14	15.474057	81	0	9.614992	100	6	14	17.281771	81	0	16.604945	19	-1.807714	-6.989953	81
88	KADURUPOKUNA	6	0	53.596000	80	46	32.876000	100	6	0	54.571854	80	46	40.749550	-39	-0.975854	-7.873550	139
89	HAMBANTOTA_T	6	7	18.088765	81	7	29.862648	32	6	7	19.402884	81	7	36.966446	-70	-1.314119	-7.103798	102
90	AMANGALAKANDA	6	5	22.266423	80	40	38.149575	227	6	5	23.631969	80	40	45.406161	124	-1.365546	-7.256586	103
91	URUMUTTA	6	10	4.664991	80	31	33.659379	434	6	10	6.039856	80	31	40.985065	329	-1.374865	-7.325686	105
92	MORAWAKA	6	17	3.070246	80	30	43.535120	100	6	17	4.413762	80	30	50.876483	623	-1.343516	-7.341363	-523
93	HINDEKNATU	6	7	30.454696	80	23	57.748909	100	6	7	31.850067	80	24	5.125776	307	-1.395371	-7.376867	-207
94	GALLE_TOWER	6	1	4.411401	80	14	34.641509	100	6	1	5.791578	80	14	43.427665	-31	-1.380177	-8.786156	131
95	KURUNDAKANDA	6	4	44.899628	80	13	56.780688	100	6	4	46.323352	80	14	4.250551	-13	-1.423724	-7.469863	113
96	AMUNERIYAGODA	6	7	1.292483	80	8	6.340285	100	6	7	2.702443	80	8	13.839409	-67	-1.409960	-7.499124	167
97	PANILKANDA	6	16	11.145494	80	6	12.295493	100	6	16	12.515713	80	6	19.824587	-49	-1.370219	-7.529094	149
98	HAYCOCK	6	19	54.239095	80	17	51.323227	100	6	19	55.590030	80	17	58.768263	558	-1.350935	-7.445036	-458
99	BOMBUWALA	6	34	48.161660	80	1	5.478606	100	6	34	49.429162	80	1	13.067383	55	-1.267502	-7.588777	45
103	OLEBODA	6	46	44.564128	80	0	45.992191	100	6	46	45.808193	80	0	53.589338	11	-1.244065	-7.597147	89
107	ASGIRIYA	7	6	31.575331	79	59	16.785001	100	7	6	32.738389	79	59	24.399076	-34	-1.163058	-7.614075	134
108	ALUTAIPOLA	7	12	17.809067	79	57	40.574854	100	7	12	18.963643	79	57	48.201704	-47	-1.154576	-7.626850	147
														Mean		-1.198459	-7.443084	
														rms of difference=		0.1855146	0.2970142	
														rms of single value=		0.1311786	0.2100208	

the co-ordinates used for obtaining simulated vectors.

So it can be concluded that an adjustment of a network using simulated GPS vectors give a reasonable estimate for the co-ordinates of a geodetic network.

5.4 Gravity measurements

Gravity measurements are used to calculate gravity anomalies at points. Then these gravity anomalies are used to obtain the Geoid Undulation at station. Geoid undulation is used to calculate the correction of the Deflection of vertical. Figure 3.1 shows the geoid, ellipsoid and geoid undulation. Gravity anomaly is defined as the difference between the measured gravity and normal gravity, which is defined by the following equation (Ewing and Mitchell, 1979).

$$g_{\phi} = g_e(1 + \beta Sin^2\phi - \beta_1 Sin^2 2\phi)$$

where β and β_1 are constants, g_e is the gravity at the equator and g_{ϕ} is the gravity at a location of latitude ϕ. For GRS80, the normal gravity is defined by the International Union of Geodesy and Geophysics (I.U.G & G.) as follows (Ewing and Mitchell, 1979). Unit of gravity given in gals.

$$g_{\phi} = 978.0490(1 + 0.0052884 Sin^2\phi - 0.0000059 Sin^2 2\phi)$$

Applications of the deflection of vertical are discussed in section 3.3.4. Determination of accurate geoid undulations, or (in other words) the determination of a good geoid model for a country is important not only for the adjustment of the new network, but also for many other applications such as resource exploration, seismic studies and other scientific studies in geology. This task can be performed by taking gravity measurements at about 1/2 degree intervals throughout the country. Selecting locations at all the old primary control points and new proposed points, gravity measurements can be obtained at approximately 15 mile intervals. This is a satisfactory interval for gravity measurements of the country. When the NAD83 was adjusted in the U.S., gravity measurements at approximately 5 arc minutes (less than 10 miles) were used for the calculation of the geoid model and hence the calculation of the correction for deflection of verticals (Makey, 1989).

5.5 Azimuth observations

Azimuth observations provide the orientation to the geodetic network. The traditional method was to do astronomical azimuth observations and consider them as geodetic azimuths for the calculations. When the Sri Lankan adjustment was done in 1932, only two astronomical observations were used for the adjustment (Jackson, 1933). These two observations were at "Vavunative" and "Kandawala" which are given as station numbers 67 and 54. Four sets of astronomical observations at Vavunative (to Tavelamunai) and six sets at Kandawala (to Halgastota) were observed. Computed azimuths of each set for these observations are given below:

Vavunative : $160^0\ 54'$ and seconds 24.94, 33.70, 27.42, 30.24,

Kandawala : $176^0\ 41'$ and seconds 32.83, 34.87, 36.43, 35.80, 27.65, 33.07

It shows that the standard errors for these two sets are 3.76 and 3.18 seconds respectively for both stations.

5000 astronomical azimuth observations were used for the NAD83 adjustment of the U. S. These astronomical azimuths, which were observed by the National Geodetic Survey (NGS) prior to 1970, are accurate up to 1.1 arc seconds (Gergen, 1989).

A considerable amount of azimuth observations are required to do a good orientation of the ellipsoid, which is the mathematical surface of the geodetic network. Astronomical azimuth observations used for the 1932 adjustment in Sri Lanka and also the observations used for NAD83 show that the accuracy of astronomical azimuth determination is usually low.

GPS provides accurate azimuths based on the GRS80 ellipsoid (Wells, 1987). Usage of GPS azimuths for the new adjustment depends on the selection of the mathematical surface for the calculations. This is one of the criteria for the selection of a suitable mathematical surface (ellipsoid) for Sri Lanka, which is discussed in detail in the next section.

5.6 Mathematical surface for the geodetic network

When we work with coordinate systems of the earth, or when taking measurements on the earth, we have to consider three surfaces. They are the actual physical surface of the earth, the geoid and the reference ellipsoid (see Figure 3.1).

The actual physical surface of the earth is so rugged that it is not suitable for mathematical calculations. An equipotential surface, usually taken as the mean sea level is defined as the "geoid". The reference ellipsoid is the mathematical surface used for calculations on the earth surface. This is the best approximation to the earth (Bomford, 1980). Therefore, the ellipsoid (or some times known as spheroid) was used as the mathematical approximation to the earth surface and for all geodetic calculations for centuries. The ellipsoid is defined as a three dimensional surface which can be obtained by rotating an ellipse around its minor axis.

When defining an ellipsoid for geodetic calculations, the following independent constants have to be defined (Bomford, 1980):

1. Length of the semi major axis.
2. Length of the semi minor axis (or flatness).
3. Direction of the minor axis (usually taken as parallel to the earth's mean polar axis).
4. The definition of the center of the ellipsoid (the earth's center of Gravity is used for GRS 80 ellipsoid, which is the accepted ellipsoid by the

International Association of Geodesy).

5. An initial point of zero longitude. Usually Greenwich is used by all nations.
6. The definition of one to one correspondence between ground points and the points on the ellipsoid.

The definition of the earth's center of gravity was possible only recently, when satellite geodesy became a possibility. As a result, three constants had to be defined in place of the center of the ellipsoid before the era of artificial satellites.

1. A locally defined origin (datum point). This is a station defining the connection between the earth surface and the ellipsoid. Also, this point defines the height of the earth surface above the ellipsoid.
2. Astronomic azimuth at the origin is defined as equal to the geodetic azimuth
3. Astronomic latitude at the origin is defined as equal to the geodetic latitude.

Table 5.5 Sizes of different ellipsoids and the year they were introduced. (Source: Bomford, 1980 and Moffit and Bouchard, 1992)

Ellipsoid	year	Semi major axis	semi minor axis
Everest	1830	6377310	6356109
Bessel	1841	6377397	6356079
Clarke	1866	6378206	6356584
Clarke	1880	6378301	6356566
Hayford	1909	6378388	6356912
Krassovsky	1948	6378245	6356863
Fischer	1960	6378155	6356773
WGS72	1974	7378135	6356750
GRS80	1977	6378137.0	6356752.314

Different reference ellipsoids were used by different countries. Table 5.3 shows their semi major and semi minor axis. Reference ellipsoids defined after the reliable determination of gravitational center of the earth use the center of the earth as the center of the ellipsoid. Older ellipsoids use the datum point to relate the ellipsoid to the earth surface. Ellipsoids which use datum points are called local reference ellipsoids. For the geodetic control adjustment of Sri Lanka in 1932, the "Everest ellipsoid was used as the mathematical surface. For the NAD27 in the U.S., Clark's 1866 ellipsoid was used. When the NAD83 was computed, the earth centered ellipsoid GRS80 was adopted as the mathematical figure of the earth by all North American Countries (Schwartz, 1985).

Although a well-defined earth-centered ellipsoid is available today, and accepted by the International Association of Geodesy (IAG), many countries still continue to use local ellipsoids. According to Bomford, "International values have been agreed on, but in most countries, past history continues to dictate the adoption of others." The reluctance for a change is due to the lack of understanding about the importance of a common, earth-centered ellipsoid for the world (Bomford, 1980). With the wide use of GPS in civilian applications, the concept of a common ellipsoid will be more popular in future.

5.7 Everest vs. GRS80 for Sri Lanka

It is very controversial when a suitable reference ellipsoid is recommended for the adjustment of a geodetic network. Sri Lanka has two choices when adopting a reference ellipsoid. The first is to use the Everest ellipsoid, which was used for the 1932 adjustment. The second is to use the earth centered reference ellipsoid adopted by the International Association of Geodesy, the GRS80. Characteristics of both reference ellipsoids are discussed below in detail in order to evaluate the advantages and disadvantages of both ellipsoids before using them in a new adjustment.

5.7.1 Everest ellipsoid

(1) Locally fit to the earth surface. Therefore, the geoid undulation is much smaller than an earth-centered ellipsoid. Geoid undulation is used to apply the correction for deviation of vertical (see Stoke's and Vening Meinesz formulas in Heiskanen and Moritz, 1967). When gravity observations are not available for the calculation of the correction

for the deflection of vertical, smaller geoid undulations are important because this correction can then be neglected. If sufficient gravity measurements are available for the calculation of the deflection of vertical there is no significant importance of having a smaller geoid undulation.

(2) Today, the size of the earth is known to the nearest meter (Bomford, 1980). New values (GRS80) for the size of the earth to the nearest meter are:

Semi major axis: 6378137

Semi minor axis: 6356752

The Everest ellipsoid used in 1932 for the adjustment of the Sri Lankan network uses the following values as semi major and semi minor axis of the earth (Jackson and Price, 1933):

Semi major axis: 6377310

Semi minor axis: 6356109

This shows that the Everest ellipsoid is approximately 827 meters shorter in its major axis and 643 meters shorter in the minor axis than the current values. Therefore, the Everest ellipsoid is not a good approximation of the earth. There is no reason to use a distorted ellipsoid as the mathematical figure of the earth surface.

5.7.2 GRS80 ellipsoid

(1) This is an earth-centered ellipsoid. The Geoid undulation for Sri Lanka for GRS80 is about 100 meters, according to the world geoid model (Rapp, 1988). If GRS80 is used as the mathematical surface for Sri Lanka, this large geoid undulation has to be calculated and used for the correction of deflection of the verticals. For this, sufficient gravity measurements have to be taken.

(2) GRS80 is the currently available best defined approximation to the earth and accepted by the international Association of Geodesy.

(3) This ellipsoid is used as the reference ellipsoid for GPS. Therefore, the azimuths and ellipsoidal heights given by GPS can be directly used for the adjustment.

(4) Normal gravity of GRS80 is defined by the International Union of Geodesy and Geophysics. Hence, the gravity anomalies can be calculated after gravity measurements, which are used for the calculation of deviation of vertical.

(5) There are some concerns about the national security of a country if a common earth centered ellipsoid is used for the calculation of coordinates because then the precise locations become wide open information. This concern has not prevented the use of GRS80 reference ellipsoid for NAD83 by the U.S. and all other North American nations. This concern cannot be taken as a major issue when selecting a mathematical figure for the coordinate calculation of Sri Lanka.

When the above characteristics of Everest and GRS80 ellipsoids are considered, it can be recommended that the GRS80 is a better choice for the new geodetic control network for Sri Lanka. Also, it should be recommended that enough gravity measurements be taken and a good geoid model be defined for the country in order to calculate the correction for deviation of vertical. GRS80 ellipsoid will provide all the conveniences needed for the wide use of GPS in the country.

5.8 Reduction of angle and distance observations

Measured distances and directions have to be corrected for the ellipsoid before they are used in the adjustment. There are corrections due to the shape of the earth and the variations of gravity. These corrections are explained in detail in almost all Geodesy books (eg. Bomford or Heiskanen and Moritz). A list of corrections required for the geodetic calculations are given below(Jeyapalan, 1994):

1. Reduction of measured mark to mark distances to the ellipsoidal arc.
2. Correction for directions due to deviation of vertical

3. Correction for directions due to skew normals
4. Correction for directions due to convergence of meridians

5.9 The least squares adjustment

Since its introduction by Gauss in 1801, (published by Legendre in 1820) least squares adjustment plays a major role in many calculations (Uotila, 1986). When more measurements are made than the minimum requirement for the calculation of unknowns (redundant observations), the least squares principle provides the best accepted procedure for getting the most probable value for unknowns.

The basic idea of the least squares principle is very simple but it provides excellent results for calculations with redundant observations. The principle assumes that the sum of squares of residuals of observations be minimum for the best solution for unknowns and gives the most probable values for unknown parameters. Residuals are defined as the difference between the most probable values and the measured values of observations. The objectives of a least squares solution can be given as follows (Bomford, 1980):

1. To produce a unique value for unknowns
2. To obtain a solution which has the maximum probability
3. To get an indication about the precision with which the unknowns have been determined.

When adjusting a geodetic control network, the least squares solution can be performed in two ways. They are:

1. The method of observation equations
2. The method of condition equations

Observed directions, lengths, coordinates etc, are written as a function of unknowns in the method of observation equations. In the method of condition equations, the errors of all observations are taken as unknowns. Then, these errors are determined and the observations are corrected. Once the observations are corrected, they can be used to determine unknowns.

A number of least squares solutions were performed for the Sri Lankan network, using different combinations of available observations. First, using only the observations used for the 1932 adjustment and then using all the available data including 29 new distance observations and 21 height observations. The first adjustment provided the extent of errors in the existing system due to improper adjustment procedures. The second adjustment provided better values for coordinates, but the absolute accuracies obtained are only within 10 meters. Therefore, Sri Lanka has to perform a new geodetic control adjustment using new reliable observations in order to provide the required accuracy for GIS. Procedure and recommendations for taking GPS observations for this purpose is discussed in sections 5.3.4.

5.10 Approximate values of parameters for the new adjustment

In the method of observation equations, observations are written as a function of parameters (Uotila, 1986). This function is the mathematical model for the adjustment. If there are s number of observations with t unknowns, observation equations can be written as follows:

$$
\begin{aligned}
a_1x_1 + b_1x_2 + \cdots + t_1x_t &= k_1 + v_1 \\
a_2x_2 + b_2x_2 + \cdots + t_2x_t &= k_2 + v_2 \\
\cdots\cdots\cdots\cdots &= \cdots \\
\cdots\cdots\cdots\cdots &= \cdots \\
a_sx_1 + b_sx_2 + \cdots + t_sx_t &= k_s + v_s
\end{aligned}
$$

As there are redundant observations, $(s - t)$ is defined as the degree of freedom. Parameters, (x values) are $\delta\phi$ s and $\delta\lambda$ s, where

$$\delta\phi = \phi_0 - \phi$$

$$\delta\lambda = \lambda_0 - \lambda$$

ϕ_0 and λ_0 are approximate values for parameters and ϕ and λ are the most probable values of parameters. v's are residuals of observations and k's are the differences between actual observations and the values calculated for observations using approximate values of parameters. In a real world situation, it can be seen that many of a's and b's will be zero because observations do not depend on all the parameters.

Hence, it is required to calculate approximate values of parameters before the adjustment. As many of the network points for the new adjustment for Sri Lanka will be from the old adjustment done in 1932, coordinates obtained by the 1932 adjustment can be used as approximate values for the new adjustment. For other new GPS points, approximate values can be obtained by processing GPS vectors with respect to any other old station. Old coordinates, which can be used as approximate values, are given in Appendix A.

5.11 Weights and priori variance

When the reliability of observations are not the same, it is appropriate to use different weights for different observations in the adjustment. Assume that there are $l_1, l_2, \cdots, l_n$ observations and the variance-covariance matrix for these observations is $\sum_{Lb}$. The variance-covariance matrix is formed by the variances of each observation in its diagonal elements and co-variances in non-diagonal elements. When the observations are uncorrelated, the variance-covariance matrix becomes a diagonal matrix with only variances in the diagonal elements. Weights for observations can be written in a matrix form as follows:

$$P_x = \sigma_0^2 \sum_{Lb}^{-1}$$

where, P_x is called the weight matrix and σ_0^2 is called the priori variance of observations. σ_0^2 can be any pre selected number and it is unit less (Uotila, 1988). When observations are un-corelated, we can select σ_0^2 as one and define the weight matrix as $\sum_x^{-1}$, which will be formed only by the variances.

For the adjustment of Sri Lankan geodetic network, variances for each observations have to be calculated in order to form the variance-covariance matrix, assuming that there are no co-relations among different types observations.

5.12 Observation equations

Directions and distances are the usual observations for many least squares adjustments in surveying and geodesy. Observation equations for these two types of observations can be written as follows (Oliver, 1977):

5.12.1 Directions

Assume that:

α = Observed direction (according to an approximate azimuth)

z = Orientation correction

v = Residual

α_0 = Approximate value of observation calculated using approximate values of parameters

$d\alpha$ = Correction for the approximate value

Then,

$$\alpha + z + v = \alpha_0 + d\alpha$$

This observation equation for the azimuth from station 1 to station 2 can be written in terms of parameters, which are the corrections for approximate values of latitudes and longitudes.

$$d\alpha = \frac{\rho_1 Sin\alpha_{12}^0}{s} d\phi_1 + \frac{\rho_2 Sin\alpha_{21}^0}{s} d\phi_2 + \frac{\nu_2 Cos\phi_2 Cos\alpha_{21}}{s}(d\lambda_1 - d\lambda_2)$$

where ρ and ν are the radii of curvatures of prime vertical and meridian. s is the distance between two points, which can be calculated using approximate values of parameters. α_{12} is the azimuth from station 1 to station 2 and ϕ_1, ϕ_2, λ_1, λ_2 are latitudes and longitudes of stations 1 and 2.

5.12.2 Distances

If s is the observed distance and s_0 is the approximate value for the distance obtained using the approximate values of parameters,

$$s + v = s_0 + ds$$

This observation equation can be written in terms of parameters as (Olliver, 1977):

$$ds = -\rho_1 Cos\alpha_{12}^0 Sin1'' d\phi_1 - \rho_2 Cos\alpha_{21}^0 Sin1'' d\phi_2 + \nu_2 Cos\phi_2 Sin\alpha_{21}^0 Sin1''(d\lambda_1 - d\lambda_2)$$

Those observation equations provide the accuracy better than ±0.05, ±0.15 and ±0.5 seconds for lines of 20 km, 50 km and 200 km respectively (Olliver,1977). Since the iteration procedure is used to calculate the most probable values of parameters, the accuracy of those observation equations are satisfactory for a least squares adjustment with an iterative procedure.

5.13 Solution for parameters

When a least squares solution is obtained using observation equations, the procedure is to form normal equations first and then perform a solution for parameters. A short description of the procedure for obtaining this solution and the method of evaluating results are given below (Uotila, 1986):

Assume that $l_1^b, l_2^b, \cdots, \cdots, l_n^b$ are observations and $v_1, v_2, \cdots, \cdots, v_n$ are residuals associated with observations. Parameters are $x_1, x_2, \cdots, \cdots, x_m$. If the mathematical model is linear,

$$l_1^b - v_1 = a_{11}x_1 + a_{12}x_2 + \cdots + \cdots + a_{1m}x_m$$

$$l_2^b - v_2 = a_{21}x_1 + a_{22}x_2 + \cdots + \cdots + a_{2m}x_m$$

$$\cdots\cdots = \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots$$

$$l_n^b - v_n = a_{n1}x_1 + a_{n2}x_2 + \cdots + \cdots + a_{nm}x_m$$

These equations can be written in matrix form as follows:

$$L_b - \epsilon = A.X$$

When the expected values are considered, expected values of residuals become zero. So, the equation can be written as:

$$\hat{L}_a = A.\hat{X}_a$$

where $\hat{L}_a$ is the estimate for true values of observations and $\hat{X}$ is the expected values of parameters.

Residuals V, is given by:

$$V = \hat{L}_a - L_b$$

Then the observation equation can be written as:

$$V = A.\hat{X}_a - L_b$$

For a non linear mathematical model, observation equation can be obtained using the Taylor's series (Uotila, 1986):

$$L_b - \epsilon = F(X^a) = F(X^0) + \left(\frac{\partial F}{\partial X_a}\right)\Big|_{X_a = X_0} .(X_a - X_0) + \cdots + \cdots$$

where X_0 are approximate values of parameters.

Using approximate values of parameters, we can calculate a theoretical value for observations. These are known as approximate values for observations and denoted as L_0. Assuming,

$$X = \hat{X}_a - X_0 \qquad \text{and} \qquad L = L_0 - L_b$$

the observation equation for a non-linear mathematical model can be written as:

$$V = A\hat{X}_a + L$$

where, $A = \left.\frac{\partial F}{\partial X_a}\right|_{X_a = X_0}$

In other words, A is the matrix of partial derivatives of observations with respect to parameters. When there are n observation equations and m parameters, order of the A matrix become $n \times m$.

For a least squares solution, we have to minimize $\sum v^2$ or in matrix form $V^T V$, assuming the weight matrix as the identity matrix. Including weights, we can minimize $V^T \sum^{-1} V$. where $\sum$ is the variance-covariance matrix for observations. If observations are uncorrelated, $\sum$ becomes a diagonal matrix.

Considering a constant, σ_0^2, the weight matrix P can be defined as:

$$P = \sigma_0^2 * \sum^{-1}$$

The constant σ_0^2 is known as the priori variance of unit weight and usually taken as one.

$V^T PV$ can be minimized by taking partial derivatives of functions with respect to each variable and making them equal to zero (Uotila, 1986). The solution for parameters can be written in matrix form as:

$$\hat{X}_a = -(A^T PA)^{-1}.(A^T PL)$$

The results of a new least squares adjustment for the primary geodetic control of Sri Lanka obtained using observation equations (Using "Geolab") are given in Appendix C. The Everest ellipsoid was used as the mathematical figure for this adjustment. Observations used are the same observations used for the 1932 adjustment.

5.14 Evaluation of the adjustment

The evaluation of the adjustment of a geodetic network is necessary to see acceptability of the results of the adjustment. In early days this evaluation was done using statistical methods. That is by calculating two variance-covariance matrices for adjusted observations and parameters. Also, a constant called a posteriori variance of unit weight, ($\hat{\sigma}_0^2$) is calculated after the adjustment, and compared with the priori variance of unit weight. Calculation of the posteriori variance of unit weight is discussed in section 5.14.2.

In addition to these error statistics, today, additional GPS observations can be used for the evaluation of the geodetic network, as GPS provides centimeter level positional accuracy for relative positioning. In order to get this centimeter level accuracy, a correct procedure has to be used as described in the next section.

5.14.1 Using GPS observations for evaluation

GPS provides positional accuracy higher than 1 ppm and hence can be used to do an accuracy analysis of a geodetic network (Snay, 1989). Also, GPS can be used to evaluate the accuracy of the densified control (with "C" order points), which is the next step after establishing the primary control.

For the evaluation of the primary network ("A" and "B" order points), new GPS observations can be taken at primary stations. Then the vectors obtained by new observations can be compared with the vectors obtained by reverse calculations of coordinates of the new adjustment.

When the NAD83 coordinate system was established for North America, an accuracy analysis was made using additional GPS observations. For this analysis, GPS data between stations were taken and processed. Components parallel to two stations (collinear component) and the component perpendicular to two stations (transverse component) were compared. This comparison and evaluation was based on the following mathematical model (Snay, 1989).

$$e = a.K^b$$

where e is the RMS (Root Mean Square) error of the vector component in meters and K is the inter-station distance in kilometers. Values for a and b for different levels of network were used as given in Table 5.6.

Table 5.6 Parameters used for equation 5.1 for the evaluation of different levels of networks (Snay, 1989)

Order	Collinear component	Transverse component
First	$a = 0.008$, $b = 0.7$	$a = 0.02$, $b = 0.5$
Second	$a = 0.01$, $b = 0.7$	$a = 0.025$, $b = 0.5$
Third	$a = 0.01$, $b = 0.7$	$a = 0.03$, $b = 0.5$

This methodology can also be used for the evaluation of the new geodetic control network of Sri Lanka, in addition to the statistical evaluations discussed in the next section.

5.14.2 Variance-covariance matrices for parameters and adjusted observations

Results of the new adjustment can be evaluated using the variance covariance matrices of adjusted parameters, adjusted observations and the posteriori variance of unit weight (Uotila, 1986).

Variance-covariance matrix of adjusted parameters gives a measure about the absolute accuracy of adjusted parameters and can be calculated as follows (Uotila, 1986): As given in section 5.13

$$\hat{X}_a = X_0 + X$$

$$L = L_0 - L_b$$

Then

$$\hat{X}_a = X_0 - (A^T PA)^{-1}(A^T PL)$$
$$= X_0 - (A^T PA)^{-1}[A^T P(L_0 - L_b)]$$
$$= X_0 - (A^T PA)^{-1}A^T PL_0 + (A^T PA)^{-1}A^T PL_b$$

But, for a mathematical model $Y = GX + C$

The variance-covariance matrix of Y is given as a function of the variance-covariance matrix of X by (Uotila, 1986):

$$\sum_Y = G \sum_X G^T$$

Using this relationship, we can write the variance covariance matrix for adjusted parameters as:

$$\sum_{\hat{X}_a} = (A^T PA)^{-1} A^T P \sum_{L_B} PA(A^T PA)^{-1}$$

Using the relationship of $P_x = \sigma_0^2 \sum_{L^b}^{-1}$

this relationship can be simplified as:

$$\sum_{\hat{X}_a} = \sigma_0^2 .(A^T PA)^{-1}$$

Variance covariance matrix of adjusted observations can be obtained using the following equation (Uotila, 1986), and be used to evaluate the quality of each observation.

$$\sum_{\hat{L}_a} = \hat{\sigma}_0^2 .(A^T PA)^{-1}.A^T$$

where $\hat{\sigma}_0^2$ is the posteriori variance of unit weight.

The posteriori variance of unit weight is given by:

$$\hat{\sigma}_0^2 = \frac{V^T PV}{n - u}$$

where n is the number of observations, u is the number of parameters and $(n - u)$ is the degree of freedom. The posteriori variance of unit weight has to be compared to the priori variance of unit weight, which is chosen before the adjustment. A good adjustment gives equal or closer values for both of these constants.

6 TRANSFORMATION PARAMETERS FOR THE NEW ADJUSTMENT

When a new geodetic network adjustment is performed, we get a new set of latitudes and longitudes of stations, which are different from the old set of latitudes and longitudes. Maps are prepared using a two dimensional coordinate system designed using a suitable map projection for the country. Therefore, after the new adjustment there are three sets of different coordinates in the country. They are:

1. Old geodetic coordinates (Latitudes and Longitudes)
2. New geodetic coordinates
3. Old two dimensional coordinates

As mentioned in Chapter 5, there are three categories of coordinate users in a country. Primary users (geodesists) establish the coordinate system. All the other users who usually have only a little knowledge about coordinate adjustments, make use of those coordinates. When one set of coordinates are replaced by another, secondary and tertiary users have to be well-informed about the change and sufficiently supported to overcome any difficulty due to the new introduction of coordinates. Therefore, primary users (Geodesists) have the responsibility of providing an easy and accurate method to transform coordinates between old and new systems. Also, latitudes and longitudes have to be converted to a plane coordinate system using a suitable map projection for the country.

Transformation of coordinates must be unique and uniform. Also, the extent of possible errors in transformation of coordinates must be known. Availability of these information to secondary and tertiary users will prevent any possible confusion. Therefore, the responsibility of primary users are not only readjusting and publishing new coordinates, but also devising a good system of coordinate transformation of the country.

There can be two major differences between different rectangular coordinate systems. They are:

1. Shift of the origin
2. Rotation of axes

Transformation of coordinates of a country from one system to another can be done in several ways. Some popular options are:

1. Direct transformation of latitudes and longitudes
2. Convert latitudes and longitudes to a rectangular earth centered coordinate system before the transformation
3. Convert to a two dimensional coordinate system using a map projection and then do the transformation

In addition to these methods of coordinate transformations, we can do the transformation and calculate transformation parameters using different mathematical models. Examples are:

1. Affine transformations
2. Transformations using a rotation and a transformation matrix

Affine transformations can be done as linear or a higher order transformations. Some conditions such as perpendicularity of axis, a common scale factor etc, can also be applied to these mathematical models. Application of conditions eliminates some of the parameters and hence reduces the need of points common to both new and old systems. Usually a satisfactorily accurate system of coordinate transformation and a mathematical model for a country has to be accepted after studies of achievable accuracy in coordinate transformation. For the coordinate transformation in the U.S. from NAD27 to NAD83, many transformation methodologies have been introduced by the DMA (Defense Mapping Agency) and NGS (National Geodetic Surveys). "NADCON" is one of the softwares introduced by the NGS for this purpose. NADCON provides 0.15-0.50 accuracy of coordinate transformations for the U.S. (Dewhurst, 1990). For the generation of a data base for GIS applications. which include transformation parameters to convert NAD27 to NAD83, it has been proposed to calculate transformation parameters for each 7.5 by 7.5 minute rectangle blocks in the continental United States. It is expected that these transformation parameters provide the accuracy of coordinate transformation within centimeter level (Shrestha and Dicks, 1990). Coordinate transformation accuracies obtained for Sri Lanka is given in section 6.3.

6.1 Mathematical models for coordinate transformations

For coordinate transformations, assuming a shift of the origin of the coordinate system (translation) and a rotation of axes, parameters for transformation of geodetic coordinates can be calculated using number of methods:

1. Convert latitudes and longitudes to global X, Y, Z coordinates and then do a rotation and a translation
2. Convert latitudes and longitudes to global X, Y, Z coordinates and then use a linear affine transformation.
3. Convert old latitudes and longitudes directly to new latitudes and longitudes using a second or higher order polynomial

In the first method, the mathematical model can be written as:

$$\begin{pmatrix} X' \\ Y' \\ Z' \end{pmatrix} = R. \begin{pmatrix} X \\ Y \\ Z \end{pmatrix} + \begin{pmatrix} \Delta X \\ \Delta Y \\ \Delta Z \end{pmatrix}$$

where, X', Y', Z' are new coordinates, X, Y, Z are old coordinates, $\Delta X, \Delta Y, \Delta Z$ are translations and R is the rotation matrix. Assuming that the rotation around Z axis is κ, rotation around Y axis is ϕ and the rotation around X axis is ω, the rotation matrix R can be written as:

$$R = \begin{pmatrix} \cos\kappa & -\sin\kappa & 0 \\ \sin\kappa & \cos\kappa & 0 \\ 0 & 0 & 1 \end{pmatrix} \begin{pmatrix} \cos\phi & 0 & \sin\phi \\ 0 & 1 & 0 \\ -\sin\phi & 0 & \cos\phi \end{pmatrix} \begin{pmatrix} 1 & 0 & 0 \\ 0 & \cos\omega & -\sin\omega \\ 0 & \sin\omega & \cos\omega \end{pmatrix}$$

In this mathematical model, unknowns are κ, ϕ, ω and $\Delta X, \Delta Y, \Delta Z$. A least squares solution for parameters in this mathematical model can be obtained using at least 3 points common to old and new systems.

In the second case, the mathematical model for the affine transformation is:

$$X' = a_1 X + b_1 Y + c_1 Z + \Delta X_0$$

$$Y' = a_2 X + b_2 Y + c_2 Z + \Delta Y_0$$

$$Z' = a_3 X + b_3 Y + c_3 Z + \Delta Z_0$$

There are 12 unknowns in this mathematical model. So, we need at least 4 common points, in order to calculate transformation parameters. More than 4 points will provide a least squares solution.

In the third case, old latitudes and longitudes are directly transformed to new latitudes and longitudes. For this case, a second order mathematical model can be written as:

$$\phi' = \phi + a_1\phi + a_2\lambda + a_3\phi^2 + a_4\lambda^2 + a_5\phi\lambda + \Delta\phi$$

$$\lambda' = \lambda + b_1\phi + b_2\lambda + b_3\phi^2 + b_4\lambda^2 + b_5\phi\lambda + \Delta\lambda$$

where, ϕ', λ' are new latitudes and longitudes , and ϕ, λ are old values. In this mathematical model, there are 12 unknowns, and geodetic coordinates are not transformed into global X, Y, Z coordinates. Hence, at least 6 common points are required for the transformation by this mathematical model.

Out of these three mathematical models, the second case, the direct transformation of global X, Y and Z coordinates has an advantage because GPS provides global X, Y and Z coordinates with respect to GRS80 ellipsoid (Jeyapalan, 1995). The software (*Con$_c$ord*) written for coordinate transformations in Sri Lanka uses this mathematical model.

6.2 Methods used for the coordinate transformation from NAD27 to NAD83

Several procedures of coordinate transformation were used in the U.S., when coordinates from NAD27 were transformed to NAD83. These different procedures used different mathematical models and are capable of producing different results. A summary of their characteristics and accuracies are given in Table 6.1. As the common control points for NAD27 and NAD83 are not densely spreaded, it has become necessary to use a number of different interpolation and extrapolation techniques for many of these methods in order to get a reasonable accuracy in coordinate transformations. As the average distance between primary control points in Sri Lanka is 30 km, and also, all possible old primary points are to be included into the new adjustment, we can expect that the interpolation and extrapolation techniques need not to be used to achieve a sufficient accuracy in coordinate transformation for Sri Lanka.

In the regression analysis method, coordinate difference between NAD27 and NAD83 (datum shift) was defined using a two dimensional polynomial. This procedure was assumed as a solution for thinly spreaded common points for both systems. "LEFT1" is

Table 6.1 Comparison of various transformation methodologies used in the U.S. for the transformation of coordinates from NAD27 to NAD83. Source: (Dewhurst, 1990)

Methods	Originator	Advantages	Disadvantages	Approximate accuracy (m)	Field use (Yes/No)
Molodensky Abridged Molodensky	DMA	Defined worldwide	General Doppler-derived	~5 - 10	Yes
Regression analysis	DMA	Defined worldwide	Inaccurate Cumbersome Local Dependency	~3 - 5	Yes
LEFTI	NGS	Documented	External data required Geometry dependent Awkward Expert required	1 - 5	No
NADCON	NGS	Fast Accurate Continuous Standardized Single source Consistent	Interpolation and extrapolation	0.15 - 0.5	Yes
Independently derived	varies	Tailored for user	Not standardized Expert may be required Discontinuous	Varies	Perhaps

a computer program which uses actual NAD27 and NAD83 coordinates with an affine transformation to calculate parameters and then do the conversion. "Nadcon" uses a similar technique but uses interpolated gridded points as common points for the calculation of parameters (Dewhurst, 1990).

6.3 Accuracy of coordinate transformation of Sri Lanka

Accuracy of a coordinate transformation can be correctly evaluated only after the finalization of the new set of coordinates. A final set of transformation parameters can also be calculated only when the new coordinates are finalized. When an adjustment is performed using a methodology similar to the proposed methodology for the new adjustment but using only a limited number of new observations, a fairly accurate study can be done about the deviation pattern between the old and new coordinates. This pattern of coordinate differences can be expected to be consistent for Sri Lanka, because all the observations used for the old adjustment have approximately the same precision and also it seems that the differences are mainly due to the poor methodology used in the adjustment procedure of the old adjustment.

Using a new least square adjustment performed using old observations and all currently available new observations, a new set of coordinates were calculated. These new coordinates were used to calculate transformation parameters. Transformation parameters were calculated for number of different sizes of geographical areas in the country. These transformation parameters were used to transform old coordinates to new coordinates and compared with the actual values of new coordinates. Results obtained for each method of transformations are given in Tables 6.1 and 6.2.

Results in Table 6.3 show that a set of transformation parameters with an average accuracy of 14 cm can be published for the district level. This accuracy will satisfy the needs of almost all secondary and tertiary level coordinate users in the country. If

Table 6.2 Accuracies of different coordinate transformations for Sri Lanka, up to provincial level

Methodology used	Number of points used	Zone or province	Max & min errors (m) in X, Y or Z	Average of RMS (meters)
One set of parameters for entire country	110		0.002 - 3.528	1.059
Two equal zones	55	Northern	0.001 - 1.344	0.801
	55	Southern	0.006 - 2.924	0.553
According to provinces	17	Western	0.000 - 0.355	0.166
	13	Central	0.020 - 0.360	0.253
	35	Northern	0.006 - 0.525	0.250
	16	Sabara	0.002 - 0.816	0.408
	18	Uva	0.003 - 0.556	0.359
	16	Southern	0.001 - 0.541	0.245
	18	NCP	0.009 - 0.588	0.366
	13	NWP	0.008 - 0.624	0.345
	18	Eastern	0.004 - 1.079	0.562

more accuracy in coordinate transformation is required, it has to be done using actual values of old and new coordinates of common points. The software "Con_cord" can be used with 4 common points for this purpose. "Con_cord" was written to transform coordinates using a 12 parameter affine transformation described in section 7.1, using the exact values of common points. No interpolation or extrapolation technique was used for transformations. The main menu of "Con_cord" is given in Appendix D.

6.4 Two dimensional coordinates

When latitudes and longitudes of geodetic control stations are calculated, next step is to convert them to a plane coordinates using a suitable map projection for the country. Plane coordinates are used for small and large scale mapping. Although GPS and other

Table 6.3 Accuracies of district level parameters for coordinate transformations

District	Number of points used	Max & min errors (m) in X, Y or Z	Average of RMS (meters)
Colombo	5	.002 - .144	0.107
Gampaha	9	.003 - .141	0.068
Kalutara	5	.005 - .099	0.083
Galle	7	.000 - .081	0.064
Matara	5	.001 - .088	0.065
Hambantota	9	.004 - .150	0.126
Ampara	5	.000 - .030	0.020
Batticalo	8	.003 - .085	0.078
Trincomalee	6	.002 - .264	0.167
Mullatiw	12	.001 - .284	0.129
Killinochchi	10	.002 - .256	0.135
Jaffna	5	.000 - .010	0.007
Mannar	11	.004 - .191	0.152
Vaunia	8	.002 - .162	0.093
Putlam	10	.007 - .391	0.311
Kurunegala	12	.005 - .325	0.288
Anuradhapura	15	.003 - .593	0.308
Polonnaruwa	10	.000 - .212	0.152
Matale	5	.009 - .151	0.118
Kandy	6	.005 - .019	0.040
N' eliya	6	.003 - .129	0.074
Badulla	7	.010 - .360	0.202
Monaragala	11	.011 - .608	0.383
Ratnapura	9	.002 - .516	0.281
Kegalle	5	.007 - .096	0.084

spatial data collection methods such as surveying provide accurate spatial data, hard copy maps are still the major source of spatial data for a GIS. In this process, hard copy maps have to be transformed into the digital form using a digitizer or a scanner. Digitizers provide the spatial data in "vector" format and scanners provide data in "raster" format.

When hard copy maps are used as the primary source of spatial data in a GIS, it is important to understand basic concepts of producing maps and the errors which can be associated in the process of map making. In order to achieve this objective, a short description about map projections are given in the following sections.

Maps are produced using the measurements taken on the earth surface, which is a curved surface. As the actual earth surface is very irregular and cannot be easily defined by mathematical relationships, an ellipsoid is accepted as the curved surface of the earth (see Figure 3.1). Therefore, the spatial data collected on the earth surface have to be reduced to the ellipsoid and then transformed in to the two dimensional surface of the map. This transformation procedure from the three dimensional earth surface to the two dimensional map surface is known as a map projection.

There are many map projections, but only a few have a significant importance for wide use of mapping. The "Transverse Mercator" and "Lambert Conformal" projections, both of which were developed by Lambert in 1772, can be described as the most useful and widely used map projections for large scale mapping in the world (Snyder, 1987). These two map projections are used for State Plane Coordinate system (STPL) of the United States. The Transverse Mercator projection facilitates the Universal Transverse Mercator (UTM) projection which is used as a common map projection for mapping the entire world. Sri Lanka uses the Transverse Mercator projection (National Atlas, 1988). As the country has a more North-South expansion, Transverse Mercator projection provides satisfactory results as a conformal projection for mapping (Snyder, 1987).

The transformation of data from a 3 dimensional earth surface to a 2 dimensional map cannot be done without introducing errors, which are generally known as distortions. When a relatively small area of the earth surface is to be mapped, these distortions can be assumed as small and neglected but for large areas they have to be considered and corrected. Distortions in map projections can be categorized into 3 groups:

1. Distortions in distances (Linear Distortions)
2. Distortions in angles (Angular Distortions)
3. Distortions in areas of the map (Area Distortions)

6.4.1 Projection Surfaces and their classifications

In map projections, three dimensional earth surface is transformed into the two dimensional map using an intermediate surface. These intermediate surfaces are called projection surfaces. Today, all the projections used for map projections are based on 3 projection surfaces (Pearson, 1990). They are a plane, a cone or a cylinder. Projections based on these three surfaces are known as Azimuthal, Conical or Cylindrical projections, respectively. Figure 6.1 shows these three basic map projections.

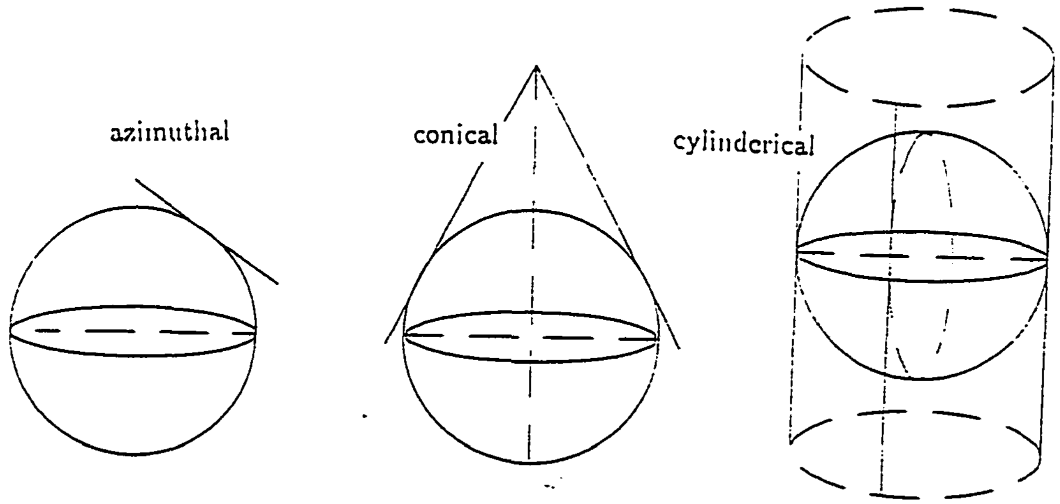


Figure 6.1 Projection surfaces used for azimuthal, cylindrical and conical projections.

The position of the projection surface to the earth gives another way of classifying map projections. They are known as "Regular", "Transverse" or "Oblique". In the cases of conical and cylindrical map projections, when the polar axis of the earth is parallel to the axis of the cone or the cylinder, they are known as regular map projections. Transverse projections are generated when these two axes are perpendicular to each other. All other cases which do not fall into categories of regular or transverse fall into the category of "Oblique".

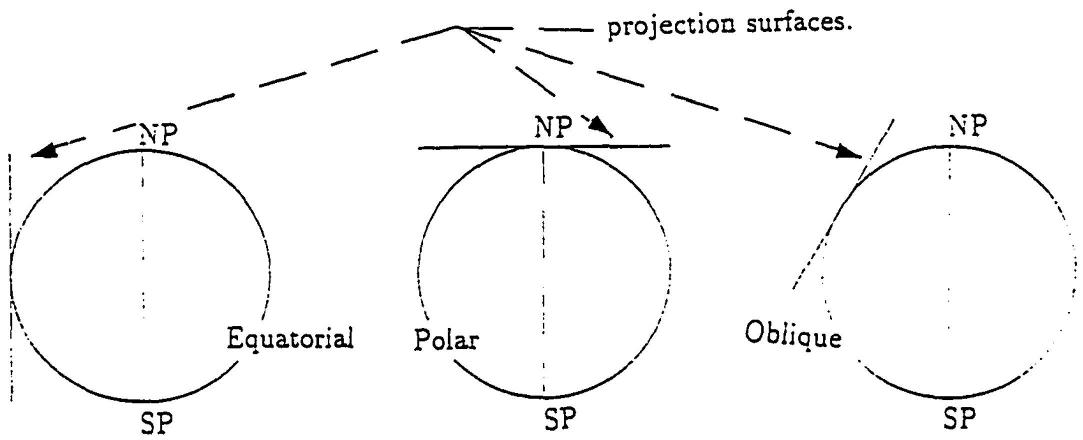


Figure 6.2 Polar equatorial and oblique map projections.

Azimuthal projections are classified as "Polar", "Equatorial" or "Oblique", depending on the common point of both surfaces to the earth and the projection surface. If the common point is the pole or a point on the equator, they are known as "Polar" and "Equatorial" respectively. All other cases are categorized as "Oblique". Figure 6.2 shows this categorization.

In some cases, the projection surface can be tangent or secant to the earth surface, as shown in Figure 6.3. This can be described as the third way of classifying map projections. When a cone or a cylinder is used as a tangent, there is only one line common to the earth surface and this line is known as the standard parallel. In secant position there are two standard parallels.

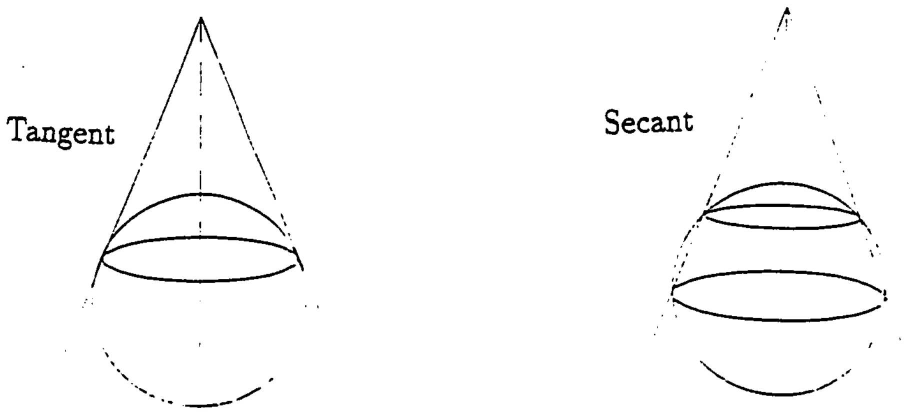


Figure 6.3 Tangent and secant map projections.

All the map projections can be classified using another characteristic of map projections. considering whether they can be obtained graphically or mathematically. In graphical projections, the surface of the earth is projected to the mapping plane through a point using simple geometric techniques. Mathematical projections can be derived only by mathematical calculations. The Transverse Mercator projection. which is used for Sri Lanka. can be described as a mathematical projection.

6.4.2 Map projection for Sri Lanka

As mentioned in the previous section, out of hundreds of available map projections. only a few can be used for large scale mapping, due to their nature of distortions. Lambert Conformal and Transverse Mercator projections, which are conical and cylindrical. respectively, are the two most popular and widely-used map projections. Transverse Mercator projection has been used for all mapping in the country (National Atlas, 1988). This projection is appropriate as a conformal projection for mapping in the country. Transverse Mercator projection is a conformal and transverse map projection. The transformation equations for this projection are obtained by imposing conditions on normal cylindrical projections.

6.4.3 Normal Cylindrical Projections

Normal cylindrical projections are tangent and regular, because the projection surface (the cylinder) touches the earth surface at the equator as shown in Figure 6.4.

In normal cylindrical projections, which are sometimes known as "Casini" projections, meridians of the datum surface are projected as equally spaced straight lines on the projection surface. Projection equations for Casini projection can be obtained as follows (Muller, 1988):

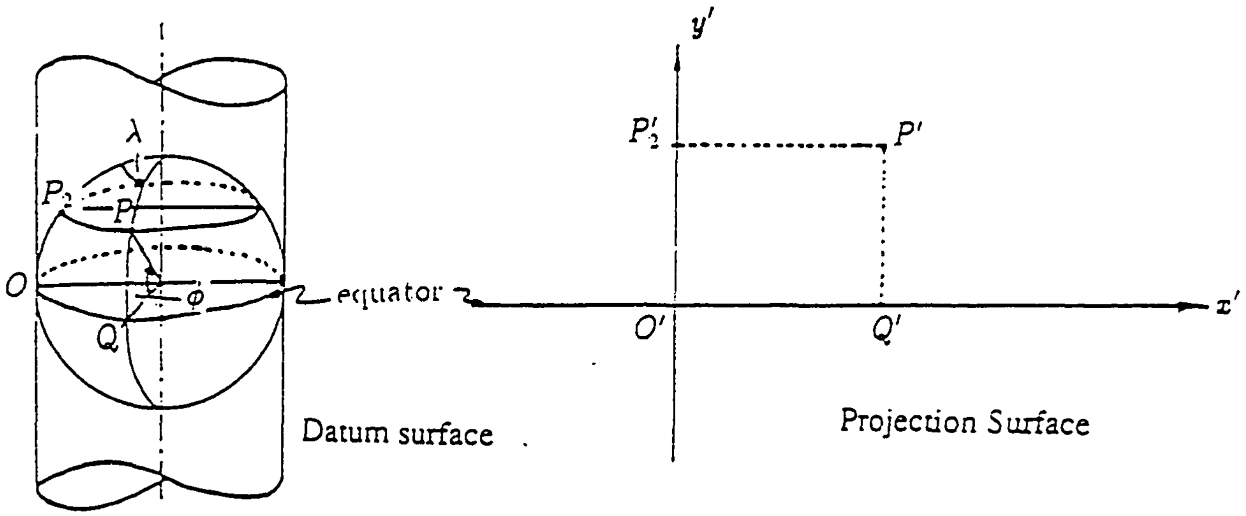


Figure 6.4 Datum and projection surface for cylindrical projections. (Source: Muller, 1988)

In Figure 6.5, NAB is a rectangular spherical triangle. Using the "Naphire's" rule for spherical triangles, we can get the transformation equation for X.

Napier's rules are:

(1) Sine of any section of the circle (Figure 6.6) is equal to the product of cosines of two opposite sections.

(2) Sine of any section of the circle is equal to the product of tangents of two adjacent

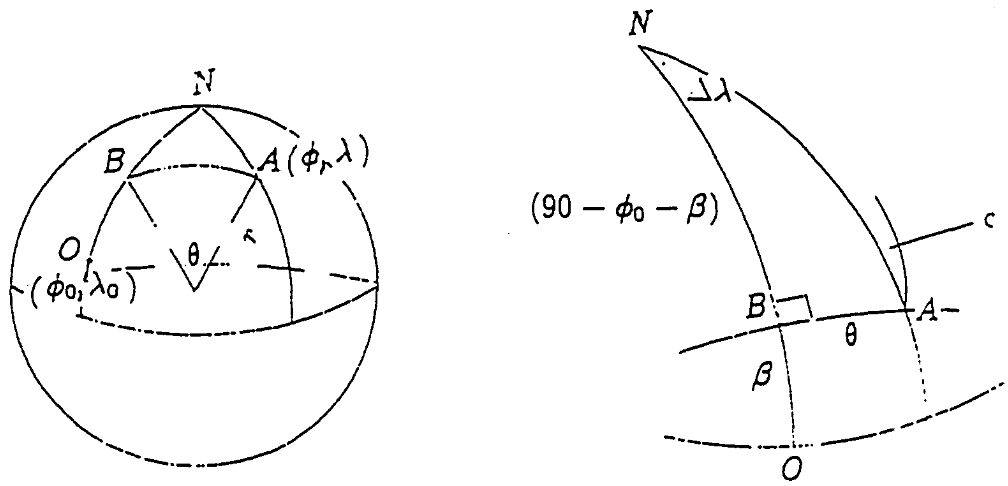


Figure 6.5 Spherical triangle in the datum surface used for the calculation of "Casini" projection

sections.

According to Figure 6.6, using the relationship for opposite sections,

$$Sin\theta = Cos(90 - \Delta\lambda).Cos\phi Sin\frac{x}{r} = Sin\Delta\lambda.Cos\phi \tag{6.1}$$

Using the relationship for adjacent sections,

$$Sin(90 - \Delta\lambda) = tan(90 - \phi_0 - \beta).tan\phi \tag{6.2}$$

$$Cos\Delta\lambda = Cot(\phi_0 + \frac{Y}{r})tan\phi \quad Cot(\phi_0 + \frac{Y}{r}) = Cos\Delta\lambda.Cot\phi \tag{6.3}$$

Equations 6.11 and 6.14 can be used to calculate the X and Y coordinates of point A.

In order to calculate the convergence, we can again use the naphier's rule,

$$Sin[90 - (90 - \phi)] = tan(90 - \Delta\lambda).tan[90 - (90 - c0] \tag{6.4}$$

$$Sin\phi = Cot\Delta\lambda.tanc \tag{6.5}$$

$$tanc = Sin\phi.tan\Delta\lambda \tag{6.6}$$

Since the angles c and $\Delta\lambda$ are small, the convergence can be written as:

$$c = \Delta\lambda.Sin\phi$$

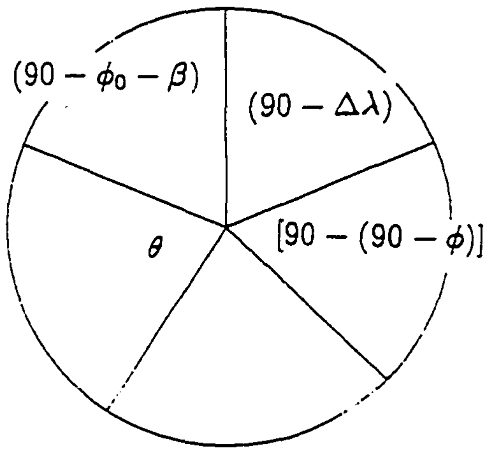


Figure 6.6 Circle used for Naphire's rule for rectangular spherical triangles

6.4.4 Transverse Mercator Projection

Transverse Mercator projection is obtained by imposing the condition of conformality and placing the cylinder in the transverse position in "Casini" projection as shown in Figure 6.7. This projection gives good results for geographical areas, which have a greater expansion in north-south direction. Characteristics of the Transverse Mercator Projection can be summarized as follows (Snyder (1987):

1. Cylindrical (Transverse)
2. Conformal
3. Central meridian and parallels are straight lines
4. Other meridians and parallels are complex curves
5. Scale is true along the central meridian

Transformation equations of Transverse Mercator projection can be obtained by first, imposing the condition of conformality to the Casini projection and then using the transverse position of the cylinder.

Projection equations for Transverse Mercator projection for the case of a globe can

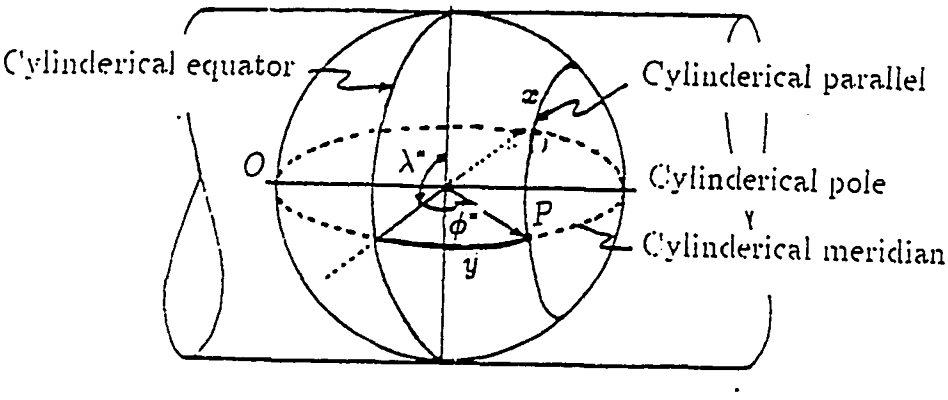


Figure 6.7 Position of the cylinder in the transverse position in "Transverse Mercator" projection (source: Muller, 1988)

be taken as follows (Snyder, 1987):

$$x = R.K_0.arctanh(B) \qquad or \qquad x = \frac{1}{2}.R.k_0.ln[(1+B)(1-B)] \qquad (6.7)$$

$$y = R.k_0.(arctan[tan\phi/Cos(\lambda - \lambda_0)] - \phi_0) \qquad (6.8)$$

$$k = \frac{k_0}{(1-B^2)^{1/2}} \qquad (6.9)$$

where $B = Cos\phi.Sin(\lambda - \lambda_0)$

k_0 = Scale factor along the central meridian λ_0

ϕ_0. λ_0 coordinates of the origin

y axis along the central meridian

Reveres formulas and the formulas in a case of an ellipsoid for Transverse Mercator projection are given in Snyder, 1987, along with transformation equations for many other map projections. For the conversion of latitudes and longitudes to Transverse Mercator projection in Sri Lanka, the central meridian and the standard parallel are chosen through the station number 65 (Pidurutalagala) of the geodetic network. Plane coordinates used in the country today use an origin located 200 km west and 200 km south of "Pidurutalagala" in order to avoid positive and negative values in coordinates (National Atlas, 1988). The software "Con_cord", mentioned earlier in this chapter can be used to convert latitudes and longitudes of the country to Transverse Mercator projection coordinates.

7 GETTING SPATIAL DATA INTO THE GIS

Creating a GIS with a cadastral map layer involves activities such as:

1. Establishing suitable geodetic control
2. Data capture
3. Getting data into the GIS

When the geodetic control is established as discussed in Chapter 5. the next step is the capturing of spatial data for the GIS. This has to be done considering two different utilizations of the GIS data base:

1. Data required for cadastral and engineering applications
2. Data required only for analysis purposes. and can be considered as information for decision making.

The cost for capturing spatial data becomes extremely high when the required spatial accuracy is increased (Henson, 1984). Due to this, it is important to consider the accuracy requirements of each data layer of the GIS. Spatial accuracy required for analysis and decision making depends on many factors such as type of the project, location, time etc, but not up to the level of accuracy required for cadastral and engineering needs. Hence, the possibilities of data capturing techniques for analysis and also for cadastral and engineering needs are discussed in the next few sections of this Chapter.

7.1 Data layers for cadastral and engineering applications

High spatial accuracy is the important part of the cadastral map layer which provides accurate coordinates for cadastral and engineering applications. The most accurate way of getting these high accuracy data into the cadastral map layer of the GIS is the direct input of digital values into the system (Byrene, 1991).

It is obvious that the capturing of highly accurate spatial data is very costly. According to Hensen (1984), the cost of data acquisition is extremely high when the requirement of the spatial accuracy becomes higher than 1 foot. Due to this reason, all the available and suitable resources of spatial data have to be used in the process of creating the cadastral map layer of the country.

There is a considerable amount of cadastral and town survey maps available in Sri Lanka. It can be estimated that these maps cover about half of the entire geographical area of the country. Almost all of these large scale maps have been prepared connecting the coordinates to the national geodetic network. They are excellent sources of getting spatial data into the cadastral map layer of the GIS. There are two ways of converting these hard copy maps into the digital form:

1. Manual digitizing.
2. Scanning and then converting raster data into the vector form.

Manual digitizing has been the popular and most widely used way of converting hard copy maps into the digital form. This technique provides highly accurate digital data, but the technique has the following disadvantages which are not found in scanning;

1. Very time consuming and tedious
2. Costlier than scanning
3. Open for human errors

Due to these reasons, it is extremely useful if a suitable scanning and vectorization procedure for large scale maps can be developed.

7.1.1 Scanning large scale cadastral maps

Scanning small scale maps and converting them into a usable form of vector data is not easy due to the availability of many different types of information in many different forms. If scanned, much more effort is required to edit them for the purpose of preparing them in a usable form.

Large scale cadastral and town survey maps are usually prepared using black lines which have a single line-thickness (see Appendix H). Details of these maps are restricted only to boundary lines and reference numbers. Due to these reasons, large scale maps are much more suitable for scanning and vectorization than small scale topographic or thematic maps.

A simulated cadastral map, which is given in Appendix G was scanned and converted into the vector form using "Arc/Info", in order to find the obtainable accuracy of coordinates. The same map was manually digitized. Coordinates obtained from both cases were compared with actual coordinates. Results of this comparison are given in Table 7.1.

Table 7.1 Comparison of coordinates by manual digitizing and scanning and vectorization

	Number of points	Mean difference in X (m)	Mean difference in Y (m)	Std. dev. in X (m)	Std. dev. in Y (m)
Manual Digitizing	38	0.26	0.26	0.34	0.23
Scanning and Vectorization	32	0.34	0.23	0.23	0.18

Although the accuracy of manual digitizing can be varied according to the operator, results in Table 7.1 show that the coordinates obtained by scanning and vectorization process using "Arc/Info" are comparable with the coordinates obtained by manual digitizing if the correct nodes can be identified. Coordinates obtained by these two cases are given in Appendix G.

7.1.2 Major problems faced in scanning and vectorization of cadastral maps

The procedures adopted for scanning and vectorizing the cadastral map were as follows:

1. Scanning the map using a "HP Scanjet IIc" scanner. Different dpi (dots per inch) were tested but the comparison is based on 600 dpi.
2. Getting the scanned image to "Grid" in ArcInfo.
3. Convert the grid into a line coverage in ArcInfo.
4. Transforming the line coverage to the ground coordinate system.

The original map and the map obtained by scanning and vectorization are given in Appendix G. It can be seen that many unnecessary nodes have been created by scanning and vectorization process. Although only 87 nodes were expected, scanning and vectorization produced 130 nodes. In this case, about 65% extra nodes have been created. But a map which has mostly rectangular property boundaries will produce much smaller amount of extra nodes. Maps without traverse lines will produce even lesser extra nodes.

Also, it was observed that a few number of necessary nodes have not been created in the process. Instead of the correct nodes, a number of wrong nodes were observed around the correct location. The percentage of these missing nodes was about 15%. Both of these problems were solved using manual editing on the screen. An automated procedure has to be developed to deal with these two problems, in order to use scanning for the conversion of large cadastral maps into the digital form.

7.1.3 Improving digitized coordinates using a least squares sequential adjustment

When coordinates of points are obtained by converting a hard copy map into digital form using manual digitizing or scanning and vectorization, some of the errors which can be contained in those coordinates are:

1. Plotting errors, which are introduced in the mapping stage. These errors can be due to human errors or due to the scale of the map.
2. Errors due to shrinkage and expansions of the map sheet.
3. Errors due to the digitization process.

These 3 types of errors can be partly eliminated and the coordinates can be improved by using a sequential least squares adjustment. This methodology has been proposed to update digital cadastral data using area, linear, angular and tangency conditions (Tamim and Schaffrin, 1985). According to Tamim and Schaffrin, this methodology has reduced errors of manually digitized coordinates by 75%. The same methodology can be used to improve coordinates obtained by scanning and vectorization.

Mathematics behind the sequential least squares adjustment can be explained as follows (Uotila, 1986):

As shown in Chapter 5, the solution for parameters can be written according to equation in section 5.11.

$$\hat{X}_a = -(A^T PA)^{-1}.(A^T PL)$$

In this case, parameters are the corrected coordinates of nodes. Digitized coordinates can be considered as the first set of observations and the conditions such as linear, angular and area can be considered as the second set of observations. When there are two sets of observations for the same set of parameters, they can be written in a mathematical model as follows:

$$L_1^a = F_1(X_a)$$

$$L_2^a = F_2(X_a)$$

Using the equation in section 4.11, the solution for the common set of parameters can be written as (Uotila, 1986):

$$\hat{X}_a = -(A_1^T P_1 A_1 + A_2^T P_2 A_2)^{-1}.(A_1^T P_1 L_1 + A_2^T P_2 L_2)$$

and the variance-covariance matrix for adjusted parameters, as shown in equation 5.13 becomes:

$$\Sigma_{\hat{X}_a} = \sigma_0^2.(A_1^T P_1 A_1 + A_2^T P_2 A_2)^{-1}$$

The equation for the solution of parameters can be written in another form to show the effect of the second set of observations (Uotila, 1986). If the solution by only the first set of parameters is $\hat{X}_a^*$ then,

$$\hat{X}_a = \hat{X}_a^* + \Delta X$$

where, ΔX is the effect of the second set of observations which are linear, angular and area conditions in this case.

It can be shown that ΔX is given by:

$$\Delta X = -N_1^{-1}.A_2^T(A_2 N_1^{-1} A_2^T + P_2^{-1})^{-1}.(A_2\hat{X}^* + L_2)$$

where, $N_1 = (A_1^T P_1 A_1)^{-1}$

Using this mathematical approach, a software was written to do the sequential least squares adjustment. This software allows manual entry of linear, angular and area conditions. It is required to extract the node coordinates of property boundaries before this adjustment. A manual approach was used to perform this step. It is required to integrate GIS software with the sequential least squares adjustment (SLS) software in order to use this methodology for larger projects. The new procedure should be capable of first getting node coordinates from the map and secondly, to perform the SLS adjustment. Thirdly, it should be able to upgrade the map using adjusted coordinates. The subroutine written for SLS is given in Appendix J.

7.2 Spatial data sources for data analysis in the Sri Lankan GIS

For analysis and decision making purposes the most widely used method is acquiring data from available hard copy maps. Usually, these small scale hard copy maps contain many errors due to two reasons. First is the inaccurate data used for mapping and the second is the errors introduced in the process of map making. Errors in mapping process can be due to following characteristics of maps (Robinson and Howard, 1984).

1. Scale of the map.
2. The map projection.
3. Generalizations involved in the map.
4. Symbolizations used in the map.

Generalizations and symbolizations are required in mapping in order to store different types of data according to the scale of the map and for convenient reading of the map. Amount of spatial errors introduced in generalization and symbolization is directly proportional to the scale of the map (Robinson and Howard, 1984). Errors introduced by the map projection depend on the type of map projection used for mapping and the location of the mapping area. Details of map projections are discussed in chapter 6.

In Sri Lanka, maps which can be used for the creation of the GIS are as follows:

1. A road map in the scale of 1 : 500,000.
2. A map covering the entire country in the scale of 1 : 250,000.
3. One inch to one mile series map (1 : 63360).
4. One hundred thousand scale land use map series (1 : 100,000).
5. Fifty thousand topographic map series (1 : 50,000).

6. Ten thousand topographic map series (1 : 10,000).
7. Block survey maps which separate state and private properties, usually in 4 chain scale (1 : 3168).
8. Large scale cadastral survey maps (1 : 4000, 1 : 2000 or larger).
9. Large scale town survey maps (1 chain, 1 : 1000 or 1 : 500).

Block survey, town survey and cadastral survey maps are prepared in large scales, without generalizations and symbolizations. They are good sources of obtaining spatial data with higher accuracy. Procedures of getting data to the GIS from these large scale maps were discussed in the section 7.1.

The road map and the map in 1 : 250,000 have been derived using the one inch map series of the country (National Atlas, 1988), and 1 inch to one mile maps have been replaced by 1 : 50,000 map series. All these small scale map series are prepared by the Survey Department, which is the government agency responsible for surveying and mapping activities in the country.

According to the map accuracy standard of the U.S., which are usually met by the department, the available spatial accuracy of small scale map series are given in Table 7.2. (Manual of Photogrammetry, 1987)

Publishing of 1 : 100,000 land use map series and 1 : 50,000 topographic map series was started in 1982 and 1977, respectively (National Atlas, 1988). Both of these map series were completed in 1990 (informal departmental sources). It is required to update

Table 7.2 Spatial accuracy of small scale maps

Map series	liner accuracy
1 :500,000	250 meters
1 :250,000	125 meters
1 :100,000	50 meters
1 : 50,000	25 meters
1 :10,000	8 meters

these maps frequently, but much of the information available in those maps is older than five years. Production of 1 : 10,000 map series also started in early 1980s and it is not yet completed.

When completed, 1 : 10,000 map series will provide information for many data layers of the national comprehensive GIS. But smaller scale maps have to be used for the areas for which suitable larger scale maps are not available.

Converting small scale maps into digital form has to be done using manual digitizing. Because of many different types of information, different colors, line types and symbols, it is extremely difficult to use scanning for the conversion of these maps into the digital form. When the digitization is done, it is required to convert the data in the digitizer coordinates to the real world coordinates (geo-referencing). Details of converting each map series into digital form and procedures for geo-referencing is discussed below:

7.2.1 1 : 500,000 scale road map

The road map is the most widely used map among the general public. According to Table 7.2, the positional accuracy which can be obtained from this map is only about 250 meters. When digitizing errors are added, this positional accuracy will even be lower than 250 meters. Hence, this map cannot be recommended for creating the national GIS. Instead, it can be used as a quick data capturing source for other purposes such as training.

Data available in 1 : 500,000 map are mainly the road network, water features and the shore line of the country. When digitized, geo-referencing of this map can be done using coordinates of grid intersections. Geodetic network points are not shown in this map. Instead, some of the mountain peaks where those geodetic control points are located are shown in this map. In order to determine the possibility of using peaks as geodetic control points, a geo-referencing transformation was performed for a digitized coverage of this map. Thirteen mountain peaks were used for the transformation.

Errors obtained in transforming digitizer coordinates to the real world coordinates are given in Appendix I and the map created is given in Appendix A. Although the RMS (Root Mean Squares) error expected according to map accuracy standards was 250 meters, an error of 508 meters were observed for 11 tic points (assuming there are blunders in point numbers 81 and 101). This indicates that the locations of geodetic control network points cannot be accurately identified in 1 : 500,000 road map of the country. Although the large RMS error can be partly due to digitization and mapping errors, the main two reasons are the wrong location identification and the unsuitability of the symbol used for peaks, as control points. It can be recommended that the geodetic control points should be shown on new versions of this map, so that they can be used mainly for geo-referencing. Also, it will create an awareness among general public about the geodetic control network, which will ultimately help the protection of geodetic network monuments.

7.2.2 1 : 100,000 scale Land use map

This map series has been prepared on a district basis. Information shown in the map is basically restricted to vegetation types, but major roads and some of the administrative boundaries are also shown in the map. When digitized, like in the 1 : 500,000 map, digitizer coordinates can be transformed to the real world coordinates using coordinates of grid intersections. Points of the geodetic control network are shown in this map as mountains. The solid triangular shape symbol used is not suitable for geodetic control points. It can be recommended that the mountains and geodetic control points are shown using two different symbols in future versions of this map.

The 1 : 100,000 map for "Matale" district was digitized and transformed to real world coordinates using geodetic control points identified on the map. This map contained only one primary control point (station 59 - "Ambokka") in the old network. However, eleven secondary control points were identified (total 14 in the district) on the map and used as tic points for geo-referencing. Names and coordinates of those 11 secondary control points are shown in Table 7.3. Similar to the 1 : 500,000 map, the RMS error observed was larger than expected. This can also be attributed to the same reasons given in the previous section.

Table 7.3 Secondary geodetic control points in Matale district: coordinates are shown as Pidurutalagala as the origin

Name of the station	X-coordinate (m)	Y-coordinate (m)
Etapola	+54095.96	-21221.72
Karanampota	+81527.74	-26167.65
Beliyakanda	+89573.53	-20527.84
Nalanda Rock	+77057.48	-14885.35
Kirigalpotta	+52631.44	-4816.28
Patanagala	+54592.85	+24394.29
Wahigala	+85539.59	-6978.84
Erawalagala	+95598.03	-4362.63
Sigiriya Rock	+112310.50	+15967.85
Kudapatana	+67975.93	+2077.10
Mariyakanda	+60601.62	+11661.35

7.2.3 1 : 50,000 Topographic map

This is the most recent and complete map series available in the country. These maps will provide following data for the national GIS:

1. Road network
2. Water features
3. Vegetation
4. Elevation
5. Administrative boundaries

In this map series, geodetic control points are shown using a well-defined point in a triangular shape symbol. Hence, they can be effectively used as tic points for digitizing and geo-referencing.

Some of the information available in sheet number 48, "Matale", were digitized and transformed to the real world coordinates using geodetic control points. Due to the large amount of information available in the map, it was observed that this map series is not appropriate for digitizing contours. However, other information can be effectively picked up using this map series.

7.2.4 1 : 10,000 Topographic map

Once completed, this map series will provide the best spatial information for many data layers of the national GIS. As there are 1834 map sheets in this series (National Atlas, 1988), it takes a considerable time to complete. Publishing of this series was started by the Survey Department of Sri Lanka in early 1980s. The series can be estimated to be completed in the year 2000 (informal departmental sources). By its completion, published map sheets in early stage will be about 20 years old. Therefore, the department has to accelerate revision of this map series in order to use them effectively in creating the national GIS.

Due to the large scale, one sheet of this map series cover only 40 square km (15.6 sq. miles). Hence, it cannot be expected that at least 4 primary or secondary geodetic control points are available in one map sheet. Thus, additional points are required with known ground coordinates for the purpose of geo-referencing. Although surveying and photogrammetry can be used, GPS can be recommended as the best procedure for obtaining ground coordinates of points identified on the map.

8 CONCLUSION

An appropriate GIS is a good tool for decision making and hence facilitating the proper use of resources in a country. Geodetic control network, which is the linkage mechanism for data layers, is an important component of a GIS (Dale and Mclaughlin, 1988). When property boundary level information is to be handled by the GIS of a country, it is critical to have an accurate geodetic control network because property boundaries have to be determined within a tolerance of few centimeters.

The present Sri Lankan geodetic control network does not provide adequate spatial accuracy for the GIS. The present absolute accuracy of the network was found to be less than 1 : 30,000 (see chapter 3). A preferable absolute spatial accuracy for a geodetic network for the use of GIS and GPS is 1 : 1,000,000 or higher (FGCC, 1988) in order to provide centimeter-level relative accuracy for property boundaries.

It was also found (using "Geolab" software), that the angle and base line measurements used for the old adjustment can provide only up to an accuracy of 1 : 60,000, if the network is newly adjusted using old observations. The absolute accuracy of individual points obtained in this adjustment varied from 5 mm to 11.43 meters in the 95% confidence interval. This indicated that a set of new observations are necessary for a new geodetic adjustment.

8.1 New geodetic network

In addition to performing as the linkage mechanism of the GIS, a geodetic control network provides coordinate control for surveying and mapping. For cadastral mapping, we need to obtain centimeter-level accuracy. Thus, it can be recommended that a geodetic network that has "C" order points (see chapter 5) with 5 cm absolute accuracy (10 km apart) should be established for Sri Lanka. The control network with "C" order points can be established using a primary control network. This primary control network should have "A" and "B" order points (see chapter 5) with better relative and absolute accuracy.

8.1.1 GPS data for the geodetic network

GPS has been realized as the best available technology for data collection for a new adjustment of a geodetic network (Leick, 1995, Wells, 1986, Puterski, 1992). When GPS is used as observations for the adjustment, the accuracy of the adjustment is mainly dependent upon the procedure used for GPS data collection. Federal geodetic control committee (FGCC) of the U.S. has published a common specifications for GPS data collection. However, these specifications can provide varying results for different networks, according to the densification of points and their locations. Hence, it is extremely useful to develop a method to predict the accuracy of the adjustment according to the quality of GPS observations. Then the procedure for GPS observations can be designed to achieve the desired accuracy of the network.

A set of simulated data was used for the prediction of the accuracy of the geodetic network of Sri Lanka. Simulated data were obtained by introducing a random error and a systematic error (as a part per million correction) to the distances obtained using the coordinates of a new adjustment. This new adjustment was performed using old available data and the "Geolab", as explained in section 5.1.2.

Before the 1960's, geodetic network adjustments were usually done by the method of "Triangulation" (Kahmen and Faig, 1988). These adjustments used only one point, called the "datum point" as a fixed point. Coordinates of these fixed points obtained by astronomical methods were assumed as correct. Today, GPS allows us to fix more than one point, if the positional accuracy of those points can be determined to a very high accuracy. Fixing more points provides a greater control for the coordinate adjustment. Simulated data has shown (see Chapter 5) that an average accuracy better than 1 cm can be obtained for a few control points which are approximately 100 km apart. This has been achieved by limiting random errors to 1 mm and using a GPS procedure which provides a 10^{-8} ppm measuring accuracy, as explained in section 5.3.2. This procedure can be used to find the coordinates of points which can be used as fixed stations in the adjustment.

Survey Department of Sri Lanka has proposed to take GPS observations in Sri Lanka according to trignometrical lines, as shown in Appendix A. Simulated data has shown that this procedure can provide only an average positional accuracy of 24.9 cm, if GPS data is good up to 10^{-8} with 5 mm random errors. Also, it was found that a better positional accuracy (average of 2.64 cm) can be achieved by increasing the number of observed lines as explained in chapter 5. This accuracy was obtained by introducing the same 10^{-8} ppm error to simulated values. Therefore, it can be recommended that the GPS observations should not be restricted to triangulation lines. As there are no inter-visibility requirements for GPS, any line can be used for GPS observations.

8.1.2 Reference ellipsoid for Sri Lanka

When the data is collected for the adjustment, the next important decision to make is the selection of a reference ellipsoid for calculations. The Everest ellipsoid has been used for the old Sri Lankan adjustment (Jackson and Price, 1933). As the Everest ellipsoid is locally fit to the country, it has the advantage of providing smaller geoid undulations

(Bomford, 1980). But the international ellipsoid of GRS80 is recommended for the new adjustment of Sri Lanka due to the reasons given in section 5.7.2. Use of GRS80 ellipsoid will greatly facilitate the wide use of GPS technology, which is becoming inevitable in many civilian applications.

8.2 Coordinate transformation

When a new set of coordinates are calculated for a country, it is required to calculate transformation parameters in order to integrate future and present work. Also, transformation parameters facilitate the use of maps prepared using the old system of coordinates with the newly adjusted linkage mechanism of the GIS. These parameters should provide sufficient accuracy for conversion of maps. It was found that a single set of parameters calculated for the entire country provides only 1 meter accuracy whereas a set of transformation parameters calculated for district level provides an average accuracy of 14 cm. Also, it was found that provincial level parameters provide about 30 cm accuracy for coordinate transformations in the country. Accuracy obtained for each district is given in Table 6.3. Parameters were calculated without using any interpolation or extrapolation technique, but using control points in and around the district boundaries. The software "*Con_cord'*" was developed for all types of coordinate transformation of the country. The main menu of "*Con_cord'*" is given in Appendix D.

8.3 Spatial data for cadastral and engineering applications

The most accurate way of creating the cadastral map layer is the direct entry of GPS and surveying data into the GIS (Byrne, 1991). This will provide highly accurate spatial data for all cadastral and engineering applications. It is extremely difficult to survey all land parcels in the country as a new project due to time and cost factors. Hence, the conversion of available cadastral maps into digital form is required. Scanning and vectorization capabilities available today provide comparable results as in manual digitization as shown in section 7.1.1. It was found that the scanning and vectorization provide about 85% of correct nodes. 15% of missing nodes were created using manual methods. An automated method for this purpose is required for large cadastral maps or large projects.

8.3.1 Updating coordinates by a sequential adjustment

A sequential least squares adjustment (Tamim and Schffrin, 1995) is a useful mathematical procedure for the elimination of digitization and mapping errors from digitized coordinates. As shown in section 7.1.1, digitized coordinates provide approximately 36 cm spatial accuracy (26 cm each in X and Y) for a cadastral map in the scale of 1 : 2000. In order to improve this accuracy of digitized coordinates, measurements in better accuracy are required for the adjustment as the second set of observations in a sequential least squares adjustment. Surveying and GPS provide this type of spatially accurate measurements.

A software was written to upgrade digitized coordinates using linear, angular and area observations as a second set of measurements and the mathematical concept explained in section 7.1.3. This software can upgrade the coordinates of a particular point where linear, angular or area measurements are available. This methodology can be mentioned as important when coordinates of a few points have to be upgraded with respect to other points. The subroutine written for this purpose is given in Appendix H.

8.4 Spatial data from small scale maps

A GIS data layer can be created using small scale maps, only if accurate measurements are not required (see Table 7.2 for spatial accuracy of small scale maps). In Sri Lanka 1 : 500,000, 1 : 100,000 and 1 : 50,000 maps are available for this purpose.

Production of a 1 : 10,000 map series is in progress.

According to the map accuracy standards, the spatial accuracy of the above scaled maps varies from 250 meters to 8 meters, respectively. Thus, when digitized, conversion of digitizer coordinates to real world coordinates (geo-referencing) can be done using the coordinates of intersections of grid lines. Using points of the geodetic control network is a better method of geo-referencing because those points are actual points available on ground that can be checked. Geodetic control points are not shown in 1 : 500,000 and 1 : 100,000 maps in the country. Instead, mountain peaks where geodetic control points are located are shown. In some cases, locations of these two types may not be the same. Although these mountain peaks can be used as geodetic control points for geo-referencing, they provided lesser accurate results. Possible reasons for this low accuracy are discussed in sections 7.2.1 and 7.2.2. It can be recommended that geodetic control points are shown in revised versions of 1 : 500,000 and 1 : 100,000 maps in Sri Lanka. Geodetic control points mapped in 1 : 50,000 maps can be effectively used for geo-referencing. Due to the small area, sufficient "A", "B" or "C" order geodetic control points can not be shown in 1 : 10,000 maps, which can be used for geo-referencing. Surveying, GPS or photogrammetric methods have to be used to obtain ground coordinates when more control points (tic points) are required. The software developed ("*Con_cord'*") can be used to convert these coordinates to any appropriate form in order to use them in the GIS.

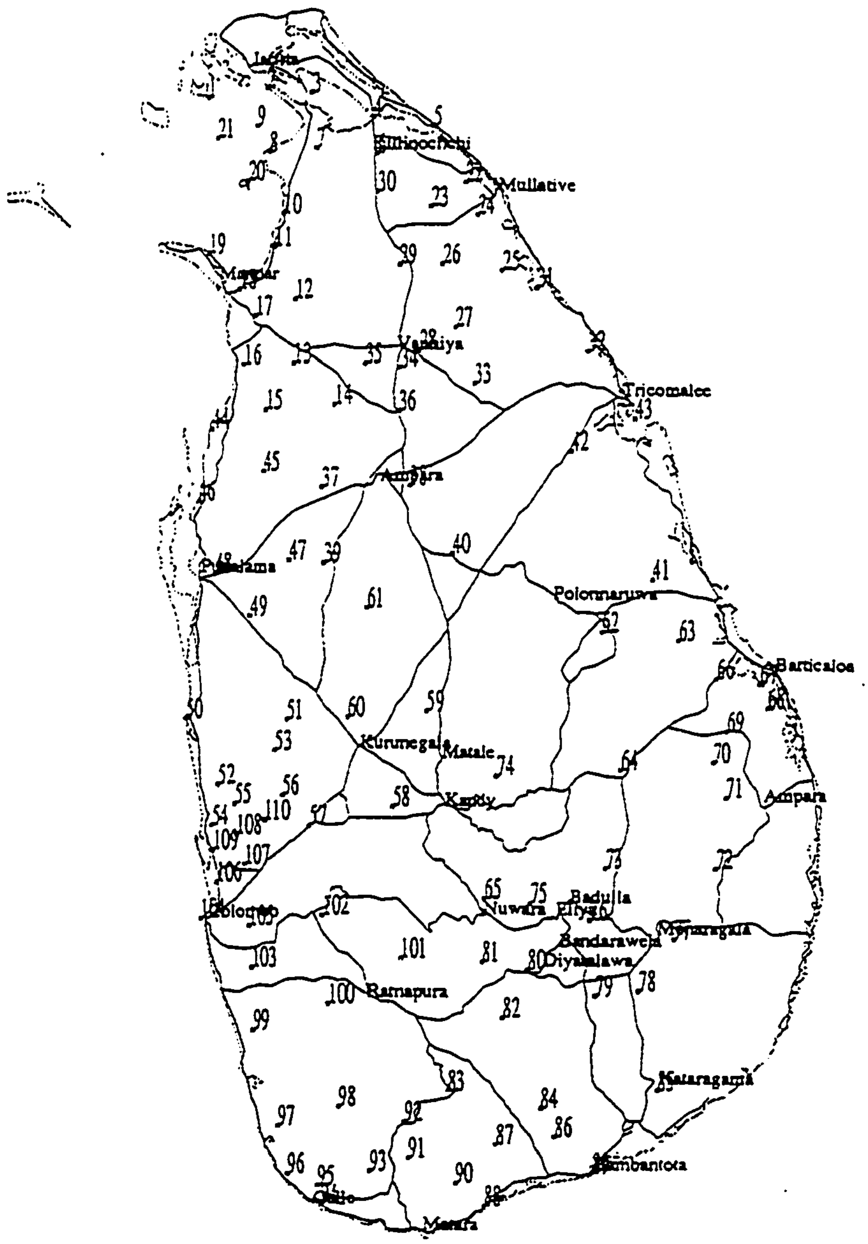
8.5 Summary of recommendations

In a brief summary, it can be recommended that a new geodetic network for Sri Lanka, which provides 1 : 1,000,000 absolute accuracy for "C" order points (10 km apart) be established before establishing the GIS. This accuracy will also satisfy the needs of GPS applications. A few points that are about 100 km apart can be used as fixed points for the adjustment. Coordinates of these points have to be determined up to 1 cm accuracy. A GPS observation procedure which provides 10^{-8} ppm measuring accuracy should be used to obtain 1 cm absolute accuracy for these "A" order points. "B" order points which have 2 cm absolute accuracy (25 km apart) should be established in the primary network. The GRS80 reference ellipsoid should be used for the new adjustment.

All large scale cadastral maps have to be digitized and incorporated into the cadastral map layer. New surveying and GPS measurements should be entered to the GIS as digital values. Digitized data can be maintained as a separate layer in order to separate digitized data from more accurate data acquired by surveying and GPS and directly entered to the system. An automated system has to be developed to obtain all the required nodes in a scanned and vectorized cadastral map. Until then, manual digitizers have to be used for converting hard copy maps to digital form. When the 1 : 10,000 map series is completed, it can be used as the base map for creating transportation, vegetation, water features and elevation layers of the national GIS. In the areas where 1 : 10,000 maps are not available, the 1 : 50,000 map series has to be used. These data can later be upgraded when 1 : 10,000 maps are produced.

APPENDIX A EXISTING GEODETIC CONTROL

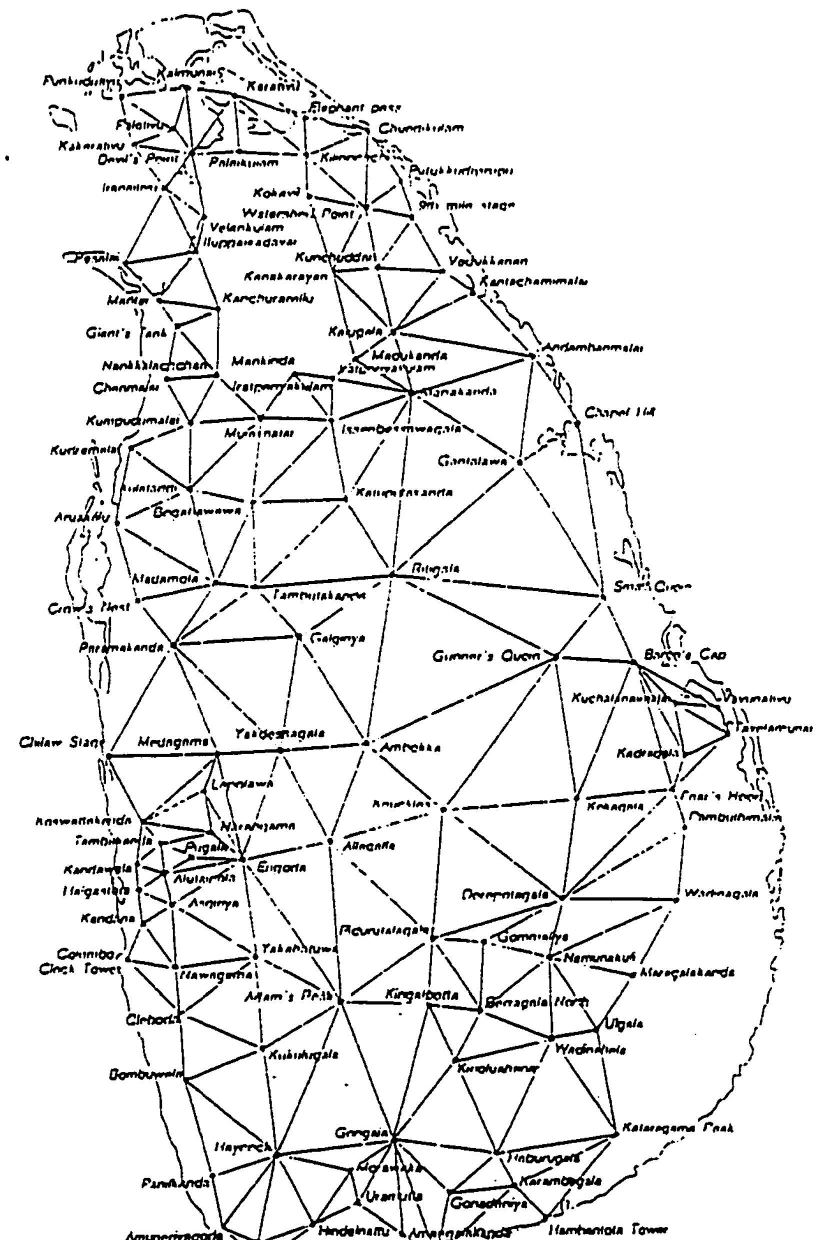
Locations of geodetic control points and major roads



0 20 40 MILES
0 20 40 KILOMETERS

Observed lines of the network

(Source: Jackson, 1933)



Station numbers and names of the primary network

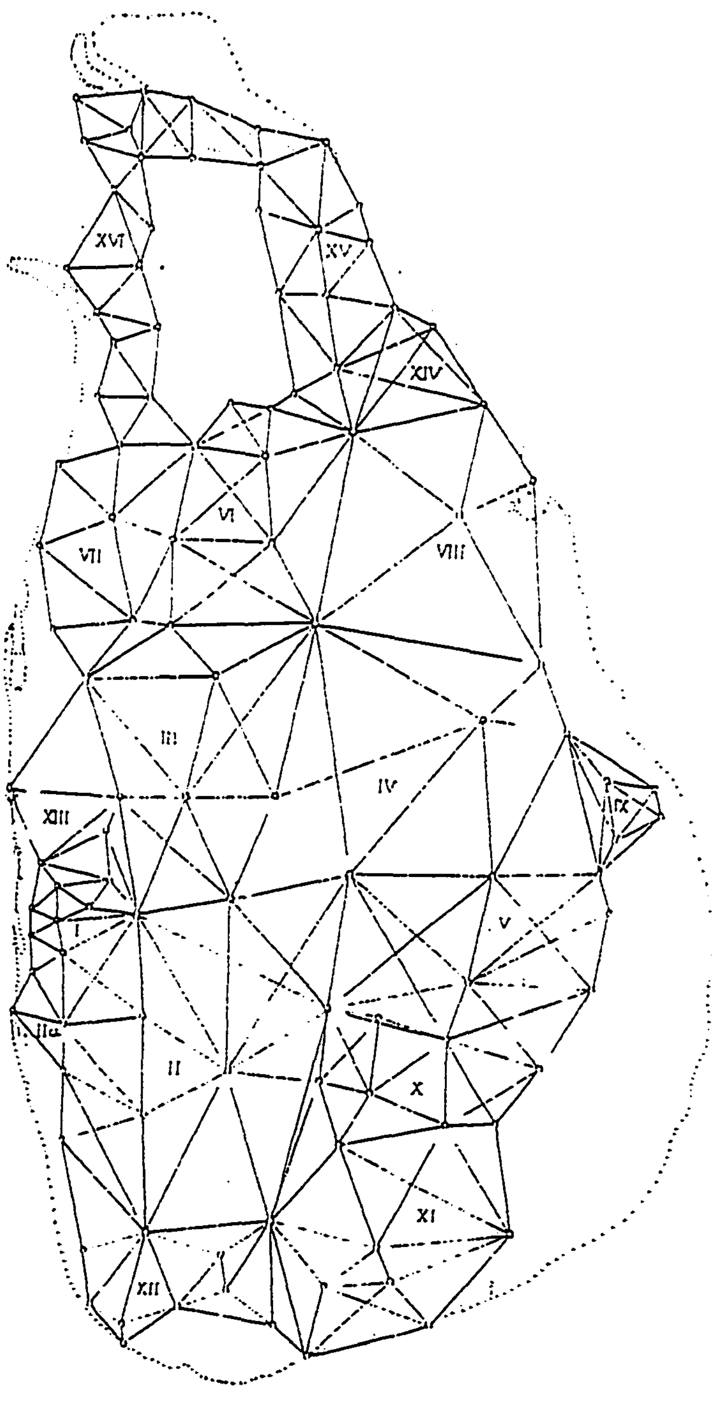
NUMBER	NAME
001	Punkudutivu
002	Kalmunai
003	Karativu
004	Elephant Pass
005	Chundikulam
006	Kilinochchi
007	Palayilkulam
008	Devil's Point
009	Paletivu
010	Velankulam
011	Iluppaikadavai
012	Kanchuravillu
013	Narikkalachchan
014	Mulamalai
015	Kumpudumalai
016	Chenmalai
017	Giant's Tank
018	Mantai
019	Pesalai
020	Iranaitivu
021	Kakerativu
022	Putukudiyirupu
023	Watershed Point
024	9th Mile Stage
025	Vedukkanari
026	Kurichuddan
027	Kalugala
028	Madukanda
029	Kanakarayan
030	Kokavil
031	Kantachamimalai
032	Andambanmalai
033	Manakanda
034	Irataperiyakulam
035	Mankinda
036	Issenbessawagala
037	Bogahawewa
038	Katupothakanda
039	Tambutakanda
040	Ritigala

041	Smali Quoin
042	Gantalawa
043	Chapel Hill
044	Kudremalai
045	Kulataddi
046	Aruakaiu
047	Madamola
048	Crow's Nest
049	Paramakanda
050	Chillaw Stage
051	Medagama
052	Koswattekanda
053	Lepolawa
054	Kandawala
055	Tambakanda
056	Haranagama
057	Engoda
058	Allagalla
059	Ambokka
060	Yakdessagala
061	Galgiriya
062	Gunner's Quoin
063	Baron's Cap
064	Kokagala
065	Pidurutalagala
066	Kuchalanaimalai
067	Vavunativu
068	Tavalamunai
069	Kadragala
070	Friar's Hood
071	Pambuthimalai
072	Wadinagala
073	Dorepotagala
074	Knuckles
075	Gommaliya
076	Namunakuli
077	Maragalakanda
078	Ulgala
079	Wadinahela
080	Berragala North
081	Kirigalpota
082	Kirioluahena
083	Gongala
084	Haburugala
085	Kataragama Peak
086	Karambagala
087	Gonadeniya
088	Kadurupokuna
089	Hambantota Tower
090	Amangalakanda
091	Uramutta

092	Morawaka
093	Hidelnattu
094	Galle Tower
095	Kurundakanda
096	Amuneriyagoda
097	Panilkanda
098	Haycock
099	Bombuwela
100	Kukulugala
101	Adam's Peak
102	Yakahatuwa
103	Oleboda
104	Colombo Tower
105	Nawagama
106	Kandana
107	Asgiriya
108	Alutaipola
109	Halgastota
110	Pugala

Breakdown of 17 smaller figures for the 1932 adjustment

(Source: Jackson, 1933)



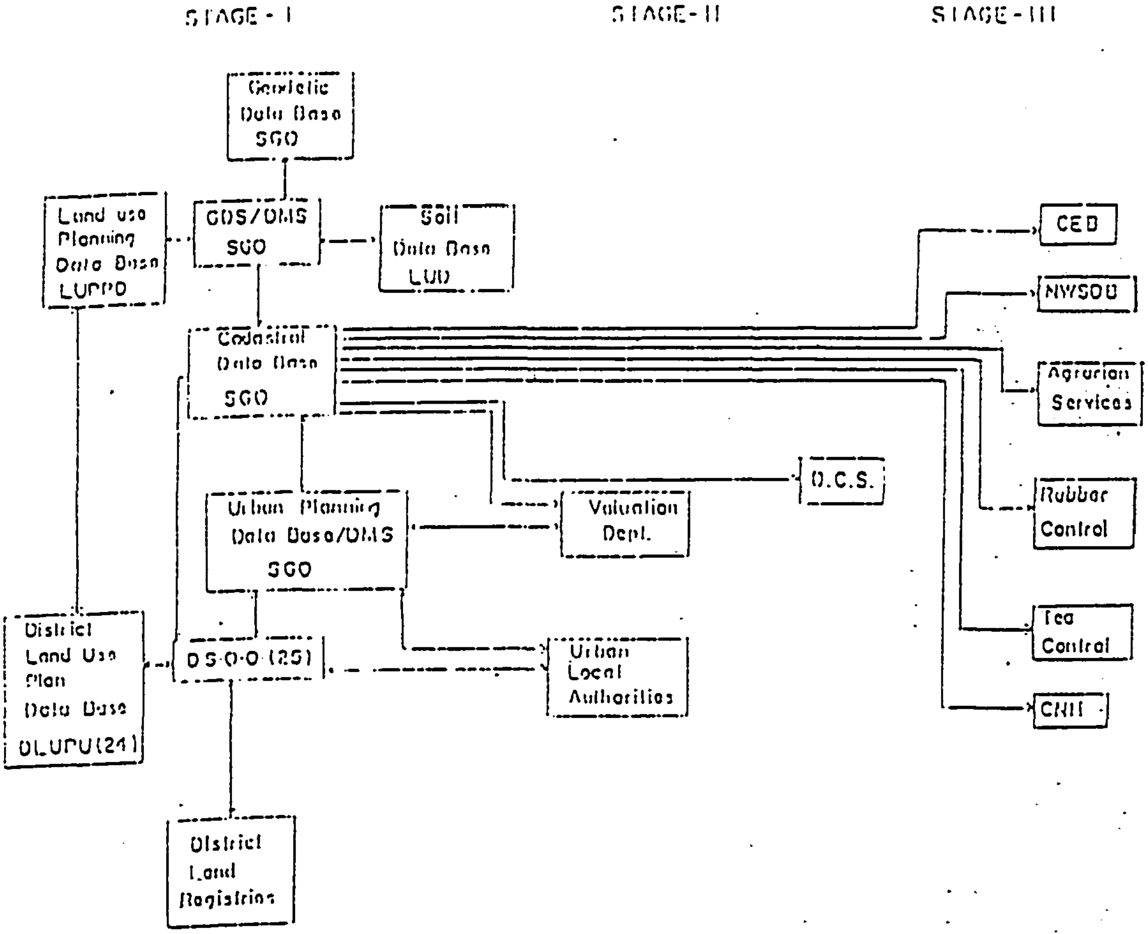
Latitudes, longitudes and plane coordinates of 1932 adjustment

Stn. #	Latitude D	M	Sec.	Longitude D	M	Sec.	Northing (meters)	Easting (meters)
1	9	34	03.530	79	51	06.800	14118.259	-5018.813
2	9	35	55.455	80	02	59.364	14286.612	-3938.405
3	9	35	04.435	80	11	43.087	14207.200	-3144.819
4	9	30	37.810	80	23	28.900	13798.561	-2075.573
5	9	28	54.107	80	35	05.755	13639.339	-1019.336
6	9	23	36.844	80	24	18.972	13155.643	-2000.347
7	9	24	34.453	80	12	32.566	13245.033	-3071.389
8	9	23	46.521	80	03	31.570	13173.329	-3891.881
9	9	28	49.599	80	00	54.535	13636.671	-4129.013
10	9	11	46.274	80	06	15.742	12072.947	-3644.995
11	9	05	35.259	80	04	37.714	11506.651	-3794.824
12	8	55	20.736	80	08	42.502	10567.555	-3424.911
13	8	43	03.156	80	08	26.935	9441.269	-3450.451
14	8	35	16.837	80	16	16.082	8728.130	-2738.637
15	8	34	12.969	80	03	03.144	8632.517	-3943.908
16	8	42	52.064	79	58	34.280	9426.030	-4350.898
17	8	52	04.960	80	00	48.591	10269.934	-4145.130
18	8	57	14.788	79	57	36.908	10743.694	-4435.199
19	9	04	08.555	79	51	26.378	11376.905	-4996.221
20	9	18	10.284	79	59	06.783	12660.703	-4294.562
21	9	26	19.016	79	53	04.104	13408.387	-4842.822
22	9	18	06.045	80	41	22.101	12649.517	-449.043
23	9	13	25.239	80	34	44.455	12220.957	-1052.400
24	9	11	51.576	80	43	52.532	12077.662	-223.944
25	9	01	06.341	80	48	41.384	11092.394	217.405
26	9	02	20.355	80	37	00.032	11205.578	-847.156
27	8	50	03.567	80	39	33.817	10080.435	-614.075
28	8	45	35.457	80	32	39.183	9671.323	-1244.030
29	9	02	11.387	80	28	46.268	11192.345	-1596.633
30	9	16	13.515	80	24	20.735	12478.660	-1998.374
31	8	57	52.442	80	55	26.931	10796.475	833.123
32	8	45	34.719	81	05	22.399	9670.555	1738.109
33	8	39	26.933	80	43	24.464	9108.245	-263.916
34	8	42	24.064	80	28	57.227	9379.302	-1581.409
35	8	43	07.737	80	22	07.227	9446.560	-2204.229
36	8	34	05.385	80	28	45.831	8617.847	-1599.305
37	8	19	16.710	80	13	55.876	7262.301	-2953.745
38	8	19	43.072	80	30	58.961	7300.992	-1397.839
39	8	04	34.414	80	14	27.112	5914.986	-2903.022
40	8	06	33.253	80	39	16.424	6094.643	-641.691
41	8	01	42.262	81	18	12.180	5652.121	2912.885
42	8	26	27.851	81	02	18.904	7919.110	1460.597
43	8	32	51.523	81	14	32.287	8506.028	2574.851
44	8	30	28.521	79	52	12.747	8291.842	-4933.254
45	8	22	22.213	80	02	36.142	7547.228	-3985.980
46	8	16	29.531	79	49	40.456	7011.145	-5167.846
47	8	05	03.660	80	07	46.512	5960.522	-3517.562
48	8	02	33.625	79	53	00.871	5733.939	-4865.803

49	7	54	22.353	80	00	02.720	4982.445	-4225.171
50	7	34	35.817	79	47	08.163	3172.969	-5408.614
51	7	34	26.315	80	07	30.836	3154.959	-3545.730
52	7	21	55.982	79	53	45.559	2011.373	-4805.417
53	7	28	33.398	80	05	09.066	2616.394	-3762.574
54	7	14	06.838	79	52	36.670	1295.180	-4911.853
55	7	18	13.607	79	57	10.778	1671.211	-4493.206
56	7	19	50.761	80	06	46.224	1818.121	-3615.695
57	7	14	06.356	80	12	02.843	1291.580	-3133.699
58	7	17	46.976	80	28	06.304	1627.027	-1664.485
59	7	36	16.779	80	34	32.227	3321.231	-1075.408
60	7	34	58.108	80	19	11.479	3202.157	-2478.207
61	7	56	03.406	80	22	45.215	5133.850	-2150.795
62	7	51	37.027	81	08	33.946	4727.001	2033.703
63	7	50	04.728	81	23	27.247	4587.669	3393.981
64	7	24	58.213	81	12	24.320	2286.076	2386.912
65	7	00	01.729	80	46	18.160	0.0	0.0
66	7	42	37.320	81	30	50.741	3905.565	4070.471
67	7	41	42.684	81	39	12.635	3823.587	4835.105
68	7	37	02.735	81	40	49.752	3396.396	4983.942
69	7	32	30.361	81	32	56.524	2979.073	4263.724
70	7	26	31.577	81	30	20.888	2430.801	4027.492
71	7	19	42.330	81	32	51.563	1806.275	4258.221
72	7	05	56.355	81	30	59.941	544.744	4090.115
73	7	05	58.339	81	09	34.959	545.378	2130.281
74	7	23	45.860	80	48	27.553	2174.441	197.209
75	6	59	15.466	80	55	07.540	-70.509	807.555
76	6	55	57.677	81	06	45.581	-371.953	1872.624
77	6	52	18.451	81	23	03.712	-705.193	3365.390
78	6	42	24.956	81	16	10.805	-1612.119	2736.260
79	6	41	35.517	81	07	52.937	-1688.269	1976.368
80	6	46	25.149	80	54	44.131	-1246.665	772.191
81	6	47	55.642	80	45	52.054	-1108.609	-39.840
82	6	37	16.694	80	49	58.272	-2084.146	336.029
83	6	23	07.719	80	39	07.179	-3380.310	-658.256
84	6	19	47.796	80	57	27.250	-3685.445	1022.039
85	6	23	11.560	81	20	02.605	-3372.833	3092.057
86	6	14	15.486	81	00	09.610	-4192.717	1270.270
87	6	12	44.277	80	48	46.658	-4332.244	226.883
88	6	00	53.638	80	46	32.895	-5417.243	22.521
89	6	07	18.113	81	07	29.852	-4829.596	1943.291
90	6	05	22.278	80	40	38.175	-5007.045	-519.564
91	6	10	04.655	80	31	33.692	-4575.649	-1351.448
92	6	17	03.060	80	30	43.554	-3936.791	-1427.744
93	6	07	30.427	80	23	57.782	-4810.724	-2048.239
94	5	01	04.356	80	14	34.674	-5399.472	-2909.324
95	6	04	44.848	80	13	56.805	-5062.758	-2966.871
96	6	07	01.235	80	08	06.356	-4853.931	-3502.219
97	6	16	11.098	80	06	12.295	-4014.139	-3675.472
98	6	19	54.209	80	17	51.327	-3674.647	-2607.213
99	6	34	48.126	80	01	05.461	-2307.881	-4141.741
100	6	39	48.826	80	15	43.886	-1850.445	-2800.051
101	6	48	32.544	80	29	50.403	-1051.839	-1507.365
102	6	56	42.472	80	14	35.381	-302.610	-2902.936
103	6	46	44.543	80	00	45.955	-1213.905	-4169.839
104	6	56	03.800	79	50	26.480	-358.261	-5113.687
105	6	54	39.659	80	00	14.664	-488.347	-4216.438
106	7	03	01.670	79	53	41.461	279.264	-4814.990
107	7	06	31.582	79	59	16.745	598.885	-4303.000
108	7	12	17.820	79	57	40.537	1127.841	-4448.807
109	7	09	20.658	79	52	53.229	858.126	-4887.453
110	7	15	04.529	80	03	11.824	1381.555	-3943.230

APPENDIX B PROPOSED LAND INFORMATION NETWORK FOR SRI LANKA

(Source: Berugoda, 1987)



SGO - Surveyor General's Office
LUPPD - Land Use Policy Planing Division of the ministry of Land
DLUPU - District Land Use Policy Units
DSOO - District Survey Offices
LUD - Land Use Division of the Irrigation Department
GDS/DMS- Geogeraphical Data and Digital Mapping Data
D.C.S. - District Kachcheries
CEB - Ceylon Electricity Board
NWSDB - National Water Supply and Drainage Board
CNH - Commisioner of National Housing

APPENDIX C RESULTS OBTAINED FOR A NEW ADJUSTMENT USING OLD OBSERVATIONS

Absolute accuracy which can be obtained by old observations

2-D AND 1-D STATION CONFIDENCE REGIONS (95.000 %):

IDENT.	MAJOR SEMI-AXIS	MINOR SEMI-AXIS	AZ(MAJ)
01	11.4343	9.7183	179.36
02	11.3311	9.4292	4.69
03	11.1273	9.1500	8.50
04	10.6479	8.6689	13.15
05	10.6014	8.5026	16.94
06	9.9966	8.0235	14.14
07	10.0918	8.1935	9.78
08	10.1314	8.3261	6.43
09	10.6572	8.8509	4.83
10	8.9914	7.2176	9.11
11	8.4501	6.7107	8.73
12	7.5309	5.7912	11.16
13	6.5351	4.8174	11.77
14	5.9166	4.1960	15.84
15	5.9446	4.2675	9.30
16	6.6899	5.0271	7.80
17	7.3701	5.6853	8.47
18	7.8642	6.2233	6.69
19	8.6290	7.0304	3.66
20	9.6894	7.9724	5.69
21	10.6071	8.9471	1.80
22	9.7166	7.5965	20.74
23	9.1768	7.1245	19.46
24	9.2262	7.0950	22.74
25	8.4672	6.3043	26.77
26	8.2693	6.2245	22.33
27	7.3413	5.3079	25.97
28	6.8722	4.9220	23.91
29	8.1501	6.1864	19.31
30	9.3291	7.3815	15.34
31	8.4497	6.2333	30.05
32	8.0326	5.7115	37.34
33	6.6556	4.6727	30.00
34	6.5704	4.6769	22.40

2-D AND 1-D STATION CONFIDENCE REGIONS (95.000 %):

IDENT.	MAJOR SEMI-AXIS	MINOR SEMI-AXIS	AZ(MAJ)
35	6.5491	4.6943	18.60
36	5.9500	4.1578	23.62
37	4.7334	3.2652	16.29
38	5.0001	3.3792	28.23
39	3.7038	2.5198	19.70
40	4.4473	2.8656	38.81
41	5.9689	3.8453	58.81
42	6.3957	4.5559	43.06
43	7.4268	5.4378	48.57
44	5.8672	4.2166	0.63
45	5.0092	3.5350	7.12
46	4.8802	3.4996	174.98
47	3.7103	2.5668	12.58
48	3.8072	2.7298	173.36
49	3.0168	2.2105	1.77
50	2.6766	1.1876	139.21
51	1.7023	1.0511	24.31
52	0.5073	0.3762	168.09
53	1.0733	0.7558	37.57
55	0.3304	0.2075	47.05
56	0.8113	0.5013	64.90
57	1.0183	0.5754	88.84
58	2.0129	1.1858	83.30
59	2.8253	1.6989	59.57
60	2.0628	1.2821	47.25
61	3.3098	2.1718	32.16
62	5.0697	3.1836	63.21
63	5.8465	3.5920	67.51
64	4.7213	2.8156	80.80
65	3.1338	1.8499	103.75
66	6.1017	3.4691	75.19
67	6.3799	3.6041	76.42
68	6.3836	3.6053	76.98
69	6.0907	3.4315	78.15
70	5.8632	3.4815	82.13
71	5.9813	3.6786	87.11
72	5.8771	3.6476	95.47
73	4.4930	2.7265	94.99
74	3.3104	1.9432	77.57
75	3.6700	2.1770	102.45
76	4.4254	2.6730	102.93
77	5.5250	3.4692	103.05
78	5.3030	3.3504	109.88
79	4.8199	2.9878	112.53
80	3.9036	2.3620	113.32
81	3.3780	2.0204	115.47
82	3.9406	2.4355	122.77

2-D AND 1-D STATION CONFIDENCE REGIONS (95.000 %):

IDENT.	MAJOR SEMI-AXIS	MINOR SEMI-AXIS	AZ(MAJ)
83	4.0931	2.5929	139.21
84	5.0811	3.2446	129.87
85	6.2425	4.0033	119.42
86	5.5012	3.5525	131.47
87	5.0685	3.2603	138.04
88	5.7551	3.7908	144.65
89	6.2652	4.1276	131.43
90	5.2342	3.4484	145.04
91	4.6180	3.0153	149.23
92	4.1379	2.6690	147.32
93	4.6210	3.0461	156.60
94	4.9764	3.4190	166.55
95	4.6586	3.1917	165.99
96	4.4364	3.1345	170.64
97	3.7373	2.5893	167.34
98	3.6466	2.3397	157.38
99	2.3923	1.5161	168.92
100	2.3204	1.4338	144.50
101	2.5003	1.4725	123.68
102	1.4604	0.8455	128.44
103	1.5687	0.9466	166.22
104	1.0059	0.6129	12.22
105	1.0736	0.6193	159.91
106	0.5435	0.3020	174.47
107	0.4867	0.2624	139.54
108	0.2650	0.1500	109.79
109	0.2162	0.0955	176.77
110	0.5478	0.3118	85.23

Differences between old and new adjustments

(same observations)

Station Number	Difference in eastings (m)	Difference in northings (m)	Station Number	Differnce in eastings (m)	Difference in northings (m)
1	-0.619	14.114	2	1.208	14.208
3	2.435	13.902	4	3.842	12.378
5	5.059	11.515	6	3.498	11.564
7	2.069	112.168	8	0.910	12.150
9	0.698	13.147	10	1.196	10.611
11	0.831	9.631	12	1.022	8.010
13	0.522	6.601	14	0.826	5.803
15	-0.108	5.945	16	-0.418	6.785
17	-0.009	7.783	18	-0.424	8.612
19	-1.051	9.750	20	0.216	11.7158
21	-0.472	12.914	22	5.106	9.833
23	4.067	9.774	24	4.727	9.127
25	5.323	8.037	26	3.836	8.437
27	3.771	6.998	28	2.957	6.616
29	2.809	8.705	30	3.095	10.653
31	5.920	7.430	32	6.594	5.583
33	3.850	5.936	34	2.464	6.283
35	1.725	6.524	36	2.184	5.411
37	0.277	4.166	38	1.806	3.909
39	0.118	3.776	40	1.781	3.035
41	3.852	1.445	42	4.690	3.553
43	6.834	3.499	44	-1.344	5.580
45	-0.284	4.586	46	-1.473	4.272
47	-0.007	3.830	48	-1.215	3.610
49	-0.779	2.986	50	-3.061	2.080
51	-0.958	2.009	52	-1.288	0.475
53	-1.201	0.951	54	-1.174	0.329
55	-1.143	0.459	56	-0.975	0.716
57	-0.421	0.516	58	0.115	0.573
59	0.977	1.431	60	0.201	1.889
61	20.814	2.698	62	2.840	1.217

Station Number	Difference in eastings (m)	Difference in northings (m)	Station Number	Differnce in eastings (m)	Difference in northings (m)
71	2.718	-1.410	72	1.923	-1.907
73	1.065	-0.907	74	0.715	0.392
75	0.433	-0.195	76	0.496	-1.130
77	0.659	-1.976	78	0.006	-1.762
79	-0.521	-1.318	80	0.001	-0.721
81	0.147	-0.502	82	-0.107	-0.158
83	0.538	-0.700	84	0.232	-0.173
85	-1.276	-0.238	86	0.192	0.005
87	0.557	-0.119	88	0.951	0.446
89	0.103	-0.476	90	1.238	-0.192
91	1.355	-0.787	92	0.925	-0.815
93	1.324	-1.279	94	1.181	-2.072
95	0.971	-1.943	96	0.669	-2.117
97	0.187	-1.574	98	0.394	-1.285
99	-0.507	-1.302	100	-0.561	-0.317
101	-0.158	-0.001	102	-0.370	-0.114
103	-1.108	-0.799	104	-1.257	0.006
105	-0.874	-0.243	106	-1.109	0.130
107	-0.972	0.229	108	-1.046	0.343
109	-1.132	0.258	110	-0.942	0.486

Latitudes and longitudes of the new adjustment

(Same observations)

Stn.	Latitude D	Latitude M	Latitude Sec.	Longitude D	Longitude M	Longitude Sec.
01	9	34	3.142831	79	51	6.752304
02	9	35	55.069647	80	2	59.272577
03	9	35	4.064235	80	11	42.968165
04	9	30	37.481156	80	23	28.754690
05	9	28	53.806294	80	35	5.585413
06	9	23	36.531446	80	24	18.840463
07	9	24	34.118687	80	12	32.462759
08	9	23	46.175456	80	3	31.491551
09	9	28	49.237016	80	0	54.459082
10	9	11	45.972997	80	6	15.659998
11	9	5	34.982887	80	4	37.643670
12	8	55	20.503318	80	8	42.433615
13	8	43	2.956404	80	8	26.888840
14	8	35	16.663699	80	16	16.041228
15	8	34	12.777129	80	3	3.120603
16	8	42	51.851859	79	58	34.256076
17	8	52	4.727227	80	0	48.551711
18	8	57	14.533340	79	57	36.874039
19	9	4	8.269456	79	51	26.354697
20	9	18	9.953590	79	59	6.722702
21	9	26	18.656398	79	53	4.055783
22	9	18	5.785885	80	41	21.941701
23	9	13	24.974096	80	34	44.322767
24	9	11	51.332341	80	43	52.389772
25	9	1	6.121090	80	48	41.230744
26	9	2	20.118880	80	36	59.913225
27	8	50	3.366647	80	39	33.707205
28	8	45	35.260395	80	32	39.094625
29	9	2	11.140321	80	28	46.172836
30	9	16	13.221992	80	24	20.619973
31	8	57	52.239311	80	55	26.766107
32	8	45	34.564537	81	5	22.223902
33	8	39	26.759617	80	43	24.358131
34	8	42	23.876598	80	28	57.152504
35	8	43	7.545595	80	22	7.168877
36	8	34	5.217740	80	28	45.760723
37	8	19	16.573156	80	13	55.842415
38	8	19	42.937560	80	30	58.890890
39	8	4	34.297189	80	14	27.085516
40	8	6	33.151541	80	39	16.342424
41	8	1	42.191119	81	18	12.034090
42	8	26	27.724908	81	2	18.760454
43	8	32	51.392653	81	14	32.104360

44	8	30	28.330550	79	52	12.764103
45	8	22	22.059235	80	2	36.132775
46	8	16	29.380920	79	49	40.489539
47	8	5	3.539194	80	7	46.503079
48	8	2	33.504023	79	53	0.905703
49	7	54	22.259194	80	0	2.740921
50	7	34	35.781774	79	47	8.211219
51	7	34	26.272243	80	7	30.837909
52	7	21	55.978476	79	53	45.560412
53	7	28	33.377433	80	5	9.067767
54	7	14	6.838000	79	52	36.670000
55	7	18	13.603733	79	57	10.776079
56	7	19	50.752319	80	6	46.214924
57	7	14	6.353127	80	12	2.827299
58	7	17	46.965308	80	28	6.266452
59	7	36	16.740877	80	34	32.169687
60	7	34	58.064950	80	19	11.452583
61	7	56	3.317675	80	22	45.171724
62	7	51	36.969870	81	8	33.825714
63	7	50	4.682280	81	23	27.098280
64	7	24	58.205514	81	12	24.204869
65	7	0	1.736267	80	46	18.109007
66	7	42	37.290945	81	30	50.588729
67	7	41	42.663768	81	39	12.467060
68	7	37	2.722145	81	40	49.585433
69	7	32	30.352491	81	32	56.373256
70	7	26	31.579367	81	30	20.747231
71	7	19	42.343085	81	32	51.424736
72	7	5	56.385915	81	30	59.816133
73	7	5	58.354492	81	9	34.863741
74	7	23	45.843589	80	48	27.490447
75	6	59	15.473374	80	55	7.469654
76	6	55	57.706058	81	6	45.496038
77	6	52	18.495971	81	23	3.616348
78	6	42	25.003801	81	16	10.731433
79	6	41	35.555898	81	7	52.860862
80	6	46	25.178413	80	54	44.063118
81	6	47	55.670348	80	45	51.991990
82	6	37	16.728810	80	49	58.207231
83	6	23	7.773627	80	39	7.090805
84	6	19	47.833890	80	57	27.168162
85	6	23	11.589482	81	20	2.547581
86	6	14	15.519716	81	0	9.527095
87	6	12	44.317037	80	48	46.568408
88	6	0	53.670106	80	46	32.794068
89	6	7	18.135258	81	7	29.771202
90	6	5	22.344014	80	40	38.052171
91	6	10	4.736934	80	31	33.588603
92	6	17	3.132202	80	30	43.467792
93	6	7	30.531891	80	23	57.692734
94	6	1	4.496680	80	14	34.607476
95	6	4	44.977753	80	13	56.745888
96	6	7	1.363530	80	8	6.317087
97	6	16	11.196710	80	6	12.270286

98	6	19	54.294553	80	17	51.276721
99	6	34	48.174668	80	1	5.451987
100	6	39	48.859575	80	15	43.851652
101	6	48	32.561301	80	29	50.354755
102	6	56	42.484510	80	14	35.359050
103	6	46	44.572013	80	0	45.944699
104	6	56	3.813689	79	50	26.483182
105	6	54	39.672604	80	0	14.657271
106	7	3	1.676059	79	53	41.459824
107	7	6	31.584779	79	59	16.740164
108	7	12	17.820254	79	57	40.533207
109	7	9	20.660517	79	52	53.228472
110	7	15	4.525925	80	3	11.815395

APPENDIX E INPUT FILE FOR A GEOLAB ADJUSTMENT WITH NEW GPS OBSERVATIONS

```
*   Welcome to GeoLab, the survey laboratory of software tools you've
*   been waiting for. This file is an example of the text input file
*   which GeoLab reads, interprets, and processes.
*
*   GeoLab input files may have initial comments like this - the
*   records beginning with '*' are completely ignored.
*
*   The following title record must be the first (non *) record:
GeoLab Adjustment of Sri Lanka full figure with Fig 1&2 and HDC 10/11
*
*   The second record must be the options record:
0 0 5     01131 112              00 0 5 50  0.001    95.0  00
*
* Reference ellipsoid is Everest ellipsoid used in 1932 adjustment
80  6377299.151  6356098.145        0        0        0
*
*
*   Station                 Latitude   Longitude      Elevation
* 4      1001          ...           30  0  0.00000w  90  0  0.00000     1955.800

14      111     SAMADI_ISMD          6 49  2.70019e  80 57 40.91115  1160.8110
 4       34     IRATTAPEKULA         8 42 24.77066e  80 29  4.58251    33.8720
 4       36     ISSENBESSAWAGAL      8 34  6.65002e  80 28 52.70220    62.7633
 4       37     BOGAHAWEWA           8 19 17.53231e  80 14  3.42980     2.3157
14       38     KATUPOTAKANDA        8 19 43.89753e  80 31  6.36227   302.1105
14       39     TAMBUTTKANDA         8  4 35.29376e  80 14 34.66119   180.1575
14       40     RITIGALA             8  6 34.14244e  80 39 23.77113   666.6753
 4       47     MADAMOLA             8  5  4.53802e  80  7 54.10847   160.0959
 4       48     CROWS_NEST           8  2 34.51353e  79 53  8.59602    29.0962
 4       49     PARAMAKANDA          7 54 23.28671e  80  0 10.38595    59.5401
 4       51     MEDAGAMA             7 34 27.34187e  80 07 38.44112   217.0589
 4       56     NARANGAMA            7 19 51.86535e  80  6 53.82980    95.9161
14       57     ENGODA               7 14  7.48473e  80 12 10.40046   207.8175
 4       59     AMBOKKA              7 36 17.79764e  80 34 39.63208  1131.6784
14       60     YAKDESSAGALA         7 34 59.12325e  80 19 18.98921   422.2009
 4       61     GALGIRIYA            7 56  4.34335e  80 22 52.68669   470.9143
14       73     DORAPATAGALA         7  5 59.50407e  81  9 42.09889  1414.7603
14       74     KNUCKLES             7 23 46.93982e  80 48 34.85377  1762.2261
14       75     GOMMOLIYA            6 59 16.63785e  80 55 14.77851  1580.9060
14       76     NAMUNUKULA           6 55 58.84319e  81  6 52.71581  1933.7946
 4       77     MARAGALAKANDA        6 52 53.88094e  81 23 22.87867  1017.6951
 4       78     UGALA                6 42 26.21453e  81 16 17.91375   475.1076
 4       80     BERAGALA_NORTH       6 46 26.38687e  80 54 51.37911  1677.9687
14       82     KIRIOLUHENA          6 37 17.94763e  80 50  5.54051   632.4133
```

4	83	GONGALA	6 23 9.04441e	80 39 14.47894	1258.4147
14	84	HABURUGALA	6 19 49.09349e	80 57 34.43936	2.5440
4	85	KATARAGAMA_PEA	6 23 4.37495e	81 19 29.05240	263.4605
4	86	KARAMBAGALA	6 14 17.28161e	81 0 16.60445	18.6120
14	88	KADURUPOKUNA	6 0 54.57172e	80 46 40.74918	39.4073
4	89	HAMBANTOTA_T	6 7 19.40276e	81 7 36.96665	69.6527
14	90	AMANGALAKANDA	6 05 23.63198e	80 40 45.40534	123.8126
4	91	URUMUTTA	6 10 6.03930e	80 31 40.98585	329.5039
4	92	MORAWAKA	6 17 4.41334e	80 30 50.87691	623.2785
4	93	HINDEKNATU	6 7 31.85032e	80 24 5.12765	307.0154
14	94	GALLE_TOWER	6 1 5.79160e	80 14 43.42696	30.8270
4	95	KURUNDAKANDA	6 4 46.31613e	80 14 4.25130	12.8704
14	96	AMUNERIYAGODA	6 7 2.70254e	80 8 13.83751	66.5288
4	97	PANILKANDA	6 16 12.51503e	80 6 19.82441	48.6191
4	98	HAYCOCK	6 19 55.58962e	80 17 58.76868	558.2764
4	99	BOMBUWALA	6 34 49.42735e	80 1 13.06479	55.3614
14	103	OLEBODA	6 46 45.80610e	80 0 53.58709	11.1961
4	107	ASGIRIYA	7 6 32.73594e	79 59 24.39626	34.2910
14	108	ALUTAIPOLA	7 12 18.95729e	79 57 48.19502	47.2406
4	112	GONADENIYA	6 12 45.72071e	80 48 53.84195	163.2035
4	113	PUGALA	7 15 5.65395e	80 3 19.45020	66.5118

913DD

*96	1007	30 0 1.94870w	90 30 52.04613	1937.020
96	111	6 49 2.70019e	80 57 40.91115	1160.811
96	34	8 42 24.77066e	80 29 4.58251	33.872
96	36	8 34 6.65002e	80 28 52.70220	62.763
96	38	8 19 43.89686e	80 31 6.36009	302.154
96	39	8 4 35.29342e	80 14 34.66110	180.187
96	39	8 4 35.29436e	80 14 34.66166	180.100
96	40	8 6 34.14219e	80 39 23.76941	666.721
96	40	8 6 34.14255e	80 39 23.77384	666.606
96	48	8 2 34.51315e	79 53 8.59569	-29.100
96	57	7 14 7.48444e	80 12 10.39949	207.717
96	57	7 14 7.48503e	80 12 10.40143	207.918
96	59	7 36 17.79760e	80 34 39.63353	1131.621
96	60	7 34 59.12345e	80 19 18.98816	422.245
96	60	7 34 59.12276e	80 19 18.98898	422.208
96	61	7 56 4.34335e	80 22 52.68669	470.914
96	73	7 5 59.50428e	81 9 42.10051	1414.766
96	74	7 23 46.93920e	80 48 34.85430	1762.168
96	74	7 23 46.93942e	80 48 34.85203	1762.287
96	75	6 59 16.63839e	80 55 14.77946	1580.898
96	75	6 59 16.63924e	80 55 14.77632	1580.928
96	76	6 55 58.84337e	81 6 52.71589	1933.775
96	76	6 55 58.84316e	81 6 52.71723	1933.782
96	77	6 52 53.88094e	81 23 22.87767	1017.692
96	78	6 42 26.21453e	81 16 17.91275	475.112
96	80	6 46 26.38687e	80 54 51.37911	1677.968
96	82	6 37 17.94760e	80 50 5.54245	632.402
96	82	6 37 17.94783e	80 50 5.53927	632.404
96	82	6 37 17.94767e	80 50 5.53940	632.484

96	83		6 23 9.04441e	80 39 14.47894		1258.414
96	84		6 19 49.09342e	80 57 34.43936		2.504
96	85		6 2: 4.37478e	81 19 29.05270		263.431
96	88		6 0 54.57174e	80 46 40.74914		-39.470
96	88		6 0 54.57184e	80 46 40.74975		-39.373
96	90		6 05 23.63228e	80 40 45.4078		123.831
96	91		6 10 6.03934e	80 31 40.98552		329.502
96	92		6 17 4.41334e	80 30 50.87691		623.278
96	94		6 1 5.79271e	80 14 43.43383		-30.761
96	94		6 1 5.79061e	80 14 43.41904		-30.881
96	96		6 7 2.70188e	80 8 13.83983		-66.641
96	99		6 34 49.42828e	80 1 13.06606		55.346
96	103		6 46 45.80646e	80 0 53.58791		11.200
96	103		6 46 45.80551e	80 0 53.58542		11.168
96	108		7 12 18.95736e	79 57 48.19573		-47.196
96	108		7 12 18.95746e	79 57 48.19518		-47.261
96	113		7 15 5.65392e	80 3 19.45059		66.513
97pdvdiagonal						
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98	0.00009	0.00009	0.00009			
98	0.00009	0.00009	0.00009			

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98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613  1937.020
96  36               8 34  6.63212e  80 28 52.73633    62.724
96  34               8 42 24.75425e  80 29  4.62771    33.474
96  40               8  6 34.14248e  80 39 23.77345   666.603
97pdvdiagonal
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98    0.00009      0.00009      0.00009
75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613  1937.020
96  37               8 19 17.53231e  80 14  3.42980     2.315
96  38               8 19 43.89767e  80 31  6.36247   302.112
96  61               7 56  4.34337e  80 22 52.68688   470.913
96  111              6 49  2.70032e  80 57 40.91170  1160.808
97pdvdiagonal
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98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613  1937.020
96  38               8 19 43.89796e  80 31  6.36493   302.063
96  34               8 42 24.75373e  80 29  4.63073    33.492
96  36               8 34  6.63212e  80 28 52.73631    62.724
96  111              6 49  2.70048e  80 57 40.91197  1160.801
97pdvdiagonal
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98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613  1937.020
96  39               8  4 35.29355e  80 14 34.66090   180.185
96  37               8 19 17.53231e  80 14  3.42980     2.315
96  38               8 19 43.89756e  80 31  6.36236   302.110
97pdvdiagonal
```

```
98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007           30  0  1.94870w  90 30 52.04613  1937.020
96  39              8  4 35.29436e  80 14 34.66166   180.100
96  47              8  5  4.53802e  80  7 54.10847   160.095
96  48              8  2 34.51347e  79 53  8.59622   -29.114
97pdvdiagonal
98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007           30  0  1.94870w  90 30 52.04613  1937.020
96  40              8  6 34.14255e  80 39 23.77384   666.606
96  34              8 42 24.75366e  80 29  4.63066    33.500
96  38              8 19 43.89796e  80 31  6.36493   302.063
97pdvdiagonal
98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007           30  0  1.94870w  90 30 52.04613  1937.020
96  47              8  5  4.53802e  80  7 54.10847   160.095
96  48              8  2 34.51359e  79 53  8.59602   -29.096
96  49              7 54 23.28671e  80  0 10.38595    59.540
96  111             6 49  2.69990e  80 57 40.91012  1160.835
97pdvdiagonal
98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007           30  0  1.94870w  90 30 52.04613  1937.020
96  49              7 54 23.28671e  80  0 10.38595    59.540
96  39              8  4 35.29400e  80 14 34.66123   180.131
96  111             6 49  2.69983e  80 57 40.91066  1160.837
97pdvdiagonal
98    0.00009      0.00009      0.00009
98    0.00009      0.00009      0.00009
75

913DD
*96 1007           30  0  1.94870w  90 30 52.04613  1937.020
96  51              7 34 27.34187e  80  7 38.44112   217.058
96  40              8  6 34.14240e  80 39 23.76992   666.727
96  111             6 49  2.70040e  80 57 40.91194  1160.809
97pdvdiagonal
```

```
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613 1937.020
96  56             7 19 51.86535e 80  6 53.82980   95.916
96  51             7 34 27.21157e 80  7 38.28770  212.564
96  111            6 49  2.70010e 80 57 40.91506 1160.728
97pdvdiagonal
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613 1937.020
96  57             7 14  7.48444e 80 12 10.39949  207.717
96  51             7 34 27.20682e 80  7 38.28499  212.492
96  56             7 19 51.86535e 80  6 53.82980   95.916
97pdvdiagonal
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613 1937.020
96  57             7 14  7.48503e 80 12 10.40143  207.918
96  108            7 12 18.95725e 79 57 48.19521  -47.215
96  113            7 15  5.65395e 80  3 19.45020   66.511
97pdvdiagonal
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613 1937.020
96  108            7 12 18.95725e 79 57 48.19521  -47.215
96  113            7 15  5.65509e 80  3 19.45028   66.519
97pdvdiagonal
98    0.00009    0.00009    0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613 1937.020
96  60             7 34 59.12276e 80 19 18.98898  422.208
96  40             8  6 34.14240e 80 39 23.76986  666.728
96  51             7 34 27.34187e 80  7 38.44112  217.058
97pdvdiagonal
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75
```

```
913DD
*96 1007          30  0  1.94870w 90 30 52.04613  1937.020
96  61             7 56  4.34335e 80 22 52.68669   470.914
96  39             8  4 35.29355e 80 14 34.66090   180.185
96  38             8 19 43.89776e 80 31  6.36240   302.110
97pdvdiagonal
98     0.00009    0.00009      0.00009
98     0.00009    0.00009      0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613  1937.020
96  73             7  5 59.50561e 81  9 42.09897  1414.757
96  75             6 59 16.63887e 80 55 14.77921  1580.892
96  111            6 49  2.70226e 80 57 40.91087  1160.831
97pdvdiagonal
98     0.00009    0.00009      0.00009
98     0.00009    0.00009      0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613  1937.020
96  73             7  5 59.50586e 81  9 42.09808  1414.765
96  74             7 23 46.94355e 80 48 34.85568  1762.223
97pdvdiagonal
98     0.00009    0.00009      0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613  1937.020
96  74             7 23 46.93942e 80 48 34.85203  1762.287
96  59             7 36 17.79764e 80 34 39.63208  1131.678
96  60             7 34 59.12361e 80 19 18.98774   422.257
97pdvdiagonal
98     0.00009    0.00009      0.00009
98     0.00009    0.00009      0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613  1937.020
96  75             6 59 16.63924e 80 55 14.77632  1580.928
96  73             7  5 59.50586e 81  9 42.09808  1414.765
96  74             7 23 46.94105e 80 48 34.85035  1762.249
97pdvdiagonal
98     0.00009    0.00009      0.00009
98     0.00009    0.00009      0.00009
75

913DD
*96 1007          30  0  1.94870w 90 30 52.04613  1937.020
96  76             6 55 58.84337e 81  6 52.71589  1933.775
96  73             7  5 59.50561e 81  9 42.09897  1414.757
```

```
96  75                    6 59 16.63686e  80 55 14.77936   1580.866
97pdvdiagonal
98    0.00009     0.00009       0.00009
98    0.00009     0.00009       0.00009
75

913DD
*96 1007                 30  0  1.94870w  90 30 52.04613   1937.020
96  76                    6 55 58.84316e  81  6 52.71723   1933.782
96  77                    6 52 53.88050e  81 23 22.87901   1017.728
96  78                    6 42 26.21442e  81 16 17.91310    475.107
96  82                    6 37 17.94755e  80 50  5.54262    632.372
97pdvdiagonal
98    0.00009     0.00009       0.00009
98    0.00009     0.00009       0.00009
98    0.00009     0.00009       0.00009
75

913DD
*96 1007                 30  0  1.94870w  90 30 52.04613   1937.020
96  78                    6 42 26.21442e  81 16 17.91310    475.107
96  77                    6 52 53.88061e  81 23 22.87899   1017.695
96  82                    6 37 17.94799e  80 50  5.54216    632.415
97pdvdiagonal
98    0.00009     0.00009       0.00009
98    0.00009     0.00009       0.00009
75

913DD
*96 1007                 30  0  1.94870w  90 30 52.04613   1937.020
96  80                    6 46 26.38687e  80 54 51.37911   1677.968
96  73                    7  5 59.50335e  81  9 42.09860   1414.789
96  75                    6 59 16.63654e  80 55 14.77898   1580.911
96  76                    6 55 58.84312e  81  6 52.71550   1933.825
97pdvdiagonal
98    0.00009     0.00009       0.00009
98    0.00009     0.00009       0.00009
98    0.00009     0.00009       0.00009
75

913DD
*96 1007                 30  0  1.94870w  90 30 52.04613   1937.020
96  82                    6 37 17.94760e  80 50  5.54245    632.402
96  77                    6 52 53.88027e  81 23 22.87779   1017.763
96  77                    6 52 53.88091e  81 23 22.88085   1017.623
97pdvdiagonal
98    0.00009     0.00009       0.00009
98    0.00009     0.00009       0.00009
75

913DD
*96 1007                 30  0  1.94870w  90 30 52.04613   1937.020
```

```
96  82                        6 37 17.94783e  80 50  5.53927     632.404
96  84                        6 19 49.09384e  80 57 34.43842       2.556
96  85                        6 23  4.37495e  81 19 29.05254     263.444
97pdvdiagonal
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
75

913DD
*96 1007                     30  0  1.94870w  90 30 52.04613    1937.020
96  83                        6 23  9.04441e  80 39 14.47894    1258.414
96  91                        6 10  6.03930e  80 31 40.98585     329.503
96  88                        6  0 54.57160e  80 46 40.74985     -39.478
97pdvdiagonal
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
75

913DD
*96 1007                     30  0  1.94870w  90 30 52.04613    1937.020
96  84                        6 19 49.09384e  80 57 34.43842       2.556
96  86                        6 14 17.28161e  81  0 16.60445      18.612
96  85                        6 23  4.37507e  81 19 29.05247     263.451
96  111                       6 49  2.70043e  80 57 40.91074    1160.833
97pdvdiagonal
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
75

913DD
*96 1007                     30  0  1.94870w  90 30 52.04613    1937.020
96  84                        6 19 49.09342e  80 57 34.43936       2.504
96  88                        6  0 54.57181e  80 46 40.74891     -39.476
96  89                        6  7 19.40276e  81  7 36.96665     -69.652
96  112                       6 12 45.72071e  80 48 53.84195     163.203
97pdvdiagonal
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
75

913DD
*96 1007                     30  0  1.94870w  90 30 52.04613    1937.020
96  86                        6 14 17.28161e  81  0 16.60445      18.612
96  82                        6 37 17.94723e  80 50  5.53988     632.490
96  85                        6 23  4.37450e  81 19 29.05293     263.460
96  111                       6 49  2.69987e  80 57 40.91113    1160.839
97pdvdiagonal
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
98   0.00009       0.00009        0.00009
```

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75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613   1937.020
96  89               6  7 19.40276e  81  7 36.96665   - 69.652
96  88               6  0 54.57167e  80 46 40.74921    -39.477
96  111              6 49  2.70014e  80 57 40.91126   1160.806
97pdvdiagonal
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613   1937.020
96  112              6 12 45.72071e  80 48 53.84195    163.203
96  88               6  0 54.57230e  80 46 40.74988    -39.475
96  89               6  7 19.40301e  81  7 36.96508    -69.647
96  111              6 49  2.70026e  80 57 40.91012   1160.801
97pdvdiagonal
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613   1937.020
96  90               6 05 23.63046e  80 40 45.40727    123.803
96  88               6  0 54.56990e  80 46 40.75083    -39.360
96  83               6 23  9.04325e  80 39 14.47939   1258.416
96  111              6 49  2.69902e  80 57 40.91161   1160.813
97pdvdiagonal
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
75

913DD
*96 1007            30  0  1.94870w  90 30 52.04613   1937.020
96  90               6 05 23.63228e  80 40 45.40785    123.831
96  93               6  7 31.85032e  80 24  5.12765    307.015
96  94               6  1  5.79207e  80 14 43.43294    -30.745
96  94               6  1  5.79181e  80 14 43.42442    -30.898
96  95               6  4 46.32268e  80 14  4.25087    -12.659
96  96               6  7  2.70285e  80  8 13.84309    -66.491
96  96               6  7  2.70269e  80  8 13.84092    -66.480
96  111              6 49  2.69930e  80 57 40.91475   1160.788
96  111              6 49  2.70203e  80 57 40.91437   1160.794
97pdvdiagonal
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
98     0.00009     0.00009        0.00009
```

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98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007              30  0  1.94870w  90 30 52.04613  1937.020
96  91                 6 10  6.03930e  80 31 40.98585   329.503
96  88                 6  0 54.57147e  80 46 40.75081   -39.378
96  90                 6 05 23.63046e  80 40 45.40727   123.803
97pdvdiagonal
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007              30  0  1.94870w  90 30 52.04613  1937.020
96  92                 6 17  4.41334e  80 30 50.87691   623.278
96  96                 6  7  2.70187e  80  8 13.84052   -66.651
96  97                 6 16 12.51500e  80  6 19.82472   -48.603
96  98                 6 19 55.58962e  80 17 58.76868   558.276
97pdvdiagonal
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007              30  0  1.94870w  90 30 52.04613  1937.020
96  93                 6  7 31.85032e  80 24  5.12765   307.015
96  111                6 49  2.70032e  80 57 40.91160  1160.823
97pdvdiagonal
98    0.00009    0.00009    0.00009
75

913DD
*96 1007              30  0  1.94870w  90 30 52.04613  1937.020
96  95                 6  4 46.31613e  80 14  4.25130   -12.870
96  96                 6  7  2.69588e  80  8 13.84720   -66.522
96  111                6 49  2.69147e  80 57 40.90359  1160.494
97pdvdiagonal
98    0.00009    0.00009    0.00009
98    0.00009    0.00009    0.00009
75

913DD
*96 1007              30  0  1.94870w  90 30 52.04613  1937.020
96  96                 6  7  2.69588e  80  8 13.84720   -66.522
96  93                 6  7 31.84344e  80 24  5.13196   306.978
96  94                 6  1  5.78475e  80 14 43.43927   -30.807
96  111                6 49  2.69350e  80 57 40.91590  1160.784
```

```
96  111                 6 49  2.69281e  80 57 40.92154  1160.798
96  111                 6 49  2.69360e  80 57 40.92872  1160.805
97pdvdiagonal
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
75

913DD
*96 1007               30  0  1.94870w 90 30 52.04613  1937.020
96  97                  6 16 12.51503e  80  6 19.82441   -48.619
96  96                  6  7  2.70066e  80  8 13.84052   -66.651
96  111                 6 49  2.70004e  80 57 40.90882  1160.805
97pdvdiagonal
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
75

913DD
*96 1007               30  0  1.94870w  90 30 52.04613  1937.020
96  98                  6 19 55.58962e  80 17 58.76868   558.276
96  96                  6  7  2.70188e  80  8 13.84009   -66.669
96  97                  6 16 12.51503e  80  6 19.82441   -48.619
96  111                 6 49  2.70020e  80 57 40.91188  1160.801
97pdvdiagonal
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
75

913DD
*96 1007               30  0  1.94870w 90 30 52.04613  1937.020
96  103                 6 46 45.80646e  80  0 53.58791    11.200
96  107                 7  6 32.73594e  79 59 24.39626   -34.291
96  108                 7 12 18.95740e  79 57 48.19498   -47.261
97pdvdiagonal
98    0.00009    0.00009        0.00009
98    0.00009    0.00009        0.00009
75

913DD
*96 1007               30  0  1.94870w 90 30 52.04613  1937.020
96  103                 6 46 45.80551e  80  0 53.58542    11.168
96  99                  6 34 49.42735e  80  1 13.06479    55.361
97pdvdiagonal
98    0.00009    0.00009        0.00009
75

913DD
*96 1007               30  0  1.94870w 90 30 52.04613  1937.020
```

```
96  107                     7  6 32.73594e  79 59 24.39626     -34.291
96  108                     7 12 18.95770e  79 57 48.19319     -47.182
96  111                     6 49  2.70051e  80 57 40.91265    1160.794
97pdvdiagonal
98    0.00009       0.00009       0.00009
98    0.00009       0.00009       0.00009
75
```

* The next record causes all data to the end of this file to be ignored.

```
99
```

APPENDIX F A PART OF THE OUTPUT FILE FOR AN ADJUSTMENT WITH NEW GPS OBSERVATIONS

```
---------------------------------------------------------------------------
          Iowa State University, Dept. of Civil Eng.
   GeoLab Adjustment of Sri Lanka full figure with Fig 1&2 and HDC 10/11
A= 6377299.151 B= 6356098.145 X0=     0.000 Y0=     0.000 Z0=     0.000
---------------------------------------------------------------------------
PREPARE:

09:51:04 - Tuesday, October 14, 1997

Input from:  <SriLanka.iob>
Output to:   <SriLanka.out>

PREPARE successfully completed.

-------------------------------------------------------------------------
|        PARAMETERS                    OBSERVATIONS
|------------------------------------------------------------------------
| Description       Number     Description        Number
|------------------------------------------------------------------------
| All Stations          45            Directions              0
| Fixed Stations        18            Distances               0
| Free 3-D Stations     27            Azimuths                0
| Free 2-D Stations     0             Vertical Angles         0
| Coord. Parameters     81            Angles                  0
| Astro. Latitudes      0             Heights                 0
| Astro. Longitudes     0             Height Differences      0
| Geoid Records         0             Auxiliary Params.       0
| All Aux. Pars.        0             2-D Coords.             0
| Direction Pars.       0             2-D Coord. Diffs.       0
| Scale Parameters      0             3-D Coords.             0
| Constant Pars.        0             3-D Coord. Diffs.       441
| Rotation Pars.        0
| Translation Pars.     0
|
|
| Total Parameters      81            Total Observations      441
|------------------------------------------------------------------------
|              Degrees of Freedom =     360
-------------------------------------------------------------------------
```

Iowa State University, Dept. of Civil Eng.
GeoLab Adjustment of Sri Lanka full figure with Fig 1&2 and HDC 10/11
A= 6377299.151 B= 6356098.145 X0= 0.000 Y0= 0.000 Z0= 0.000

GETUP:

SUMMARY OF SELECTED OPTIONS

OPTION	SELECTION
Computation Mode	Adjustment
Linear Unit	Metre
Maximum Iterations	5
Confidence Regions Selected	Point and Connected Relative
Confidence Region Dimensions	1-D, 2-D, and 3-D
Print Input Station Data	Off
Variance Factor Knowledge	Known
Confidence Level for Statistics	95.000
Dual-Height Mode	Off
Print Solution Vector	On All Iterations
Printed Ellipsoidal Coordinates	6 Decimal Places
Print Adjusted X, Y, Z	On
Print Histograms	On
Print Misclosures	All on First Pass Only
Print Residuals	All
Variance Factor Usage	Scale Confidence Regions
Residual Rejection Criterion	Tau Max
Angular Misclosure Limit Factor	5
Linear Misclosure Limit Factor	5
Convergence Criterion	0.001000

GETUP successfully completed.

Iowa State University, Dept. of Civil Eng.
GeoLab Adjustment of Sri Lanka full figure with Fig 1&2 and HDC 10/11
A= 6377299.151 B= 6356098.145 X0= 0.000 Y0= 0.000 Z0= 0.000

FORMEQ:

NOTE 6: Reordering was done.

AT	TO	OBS TYPE	OBSERVATION	APPROX.SIG.	MISCLOSURE
111	34	3-D X-Coord Diff	47162.8039	0.0095	0.0000
111	34	3-D Y-Coord Diff	-37431.0617	0.0095	0.0000
111	34	3-D Z-Coord Diff	206892.0978	0.0095	0.0000
111	36	3-D X-Coord Diff	47905.6901	0.0095	0.0000
111	36	3-D Y-Coord Diff	-35196.3934	0.0095	0.0000
111	36	3-D Z-Coord Diff	191767.7740	0.0095	0.0000
111	38	3-D X-Coord Diff	44555.5608	0.0095	-0.0734
111	38	3-D Y-Coord Diff	-30447.4693	0.0095	-0.0349
111	38	3-D Z-Coord Diff	165586.5749	0.0095	0.0140
111	39	3-D X-Coord Diff	75124.8979	0.0095	-0.0080
111	39	3-D Y-Coord Diff	-31711.7320	0.0095	-0.0303
111	39	3-D Z-Coord Diff	137942.2853	0.0095	0.0061
111	39	3-D X-Coord Diff	75124.8657	0.0095	0.0242
111	39	3-D Y-Coord Diff	-31711.8180	0.0095	0.0557
111	39	3-D Z-Coord Diff	137942.3017	0.0095	-0.0102
111	40	3-D X-Coord Diff	30163.6713	0.0095	-0.0595
111	40	3-D Y-Coord Diff	-24180.0591	0.0095	-0.0375
111	40	3-D Z-Coord Diff	141625.5592	0.0095	0.0011
111	40	3-D X-Coord Diff	30163.5188	0.0095	0.0930
111	40	3-D Y-Coord Diff	-24180.1510	0.0095	0.0544
111	40	3-D Z-Coord Diff	141625.5539	0.0095	0.0064
111	48	3-D X-Coord Diff	113961.2016	0.0095	-0.0095
111	48	3-D Y-Coord Diff	-38196.9196	0.0095	0.0041
111	48	3-D Z-Coord Diff	134239.3210	0.0095	0.0121
111	57	3-D X-Coord Diff	81590.3548	0.0095	-0.0126
111	57	3-D Y-Coord Diff	-20224.2344	0.0095	0.1017
111	57	3-D Z-Coord Diff	45764.1341	0.0095	0.0214
111	57	3-D X-Coord Diff	81590.3297	0.0095	0.0125
111	57	3-D Y-Coord Diff	-20224.0300	0.0095	-0.1027
111	57	3-D Z-Coord Diff	45764.1774	0.0095	-0.0219
111	59	3-D X-Coord Diff	40070.1160	0.0095	0.0531
111	59	3-D Y-Coord Diff	-17618.2362	0.0095	0.0483
111	59	3-D Z-Coord Diff	86409.8700	0.0095	0.0088
111	60	3-D X-Coord Diff	67835.2666	0.0095	-0.0391
111	60	3-D Y-Coord Diff	-22678.6666	0.0095	-0.0378
111	60	3-D Z-Coord Diff	83920.3035	0.0095	-0.0120
111	60	3-D X-Coord Diff	67835.2361	0.0095	-0.0086
111	60	3-D Y-Coord Diff	-22678.6957	0.0095	-0.0086

111	60	3-D Z-Coord Diff	83920.2776	0.0095	0.0139
111	61	3-D X-Coord Diff	60509.0329	0.0095	0.0000
111	61	3-D Y-Coord Diff	-26706.8700	0.0095	0.0000
111	61	3-D Z-Coord Diff	122439.2063	0.0095	0.0000
111	73	3-D X-Coord Diff	-22418.9951	0.0095	0.0483
111	73	3-D Y-Coord Diff	-49.9822	0.0095	-0.0127
111	73	3-D Z-Coord Diff	31040.4229	0.0095	-0.0071
111	74	3-D X-Coord Diff	15385.6793	0.0095	0.0248
111	74	3-D Y-Coord Diff	-9888.0424	0.0095	0.0518
111	74	3-D Z-Coord Diff	63619.4587	0.0095	0.0264
111	74	3-D X-Coord Diff	15385.7667	0.0095	-0.0626
111	74	3-D Y-Coord Diff	-9887.9379	0.0095	-0.0528
111	74	3-D Z-Coord Diff	63619.4807	0.0095	0.0043
111	75	3-D X-Coord Diff	4139.2964	0.0095	0.0304
111	75	3-D Y-Coord Diff	-2533.2529	0.0095	0.0052
111	75	3-D Z-Coord Diff	18776.3066	0.0095	-0.0155
111	75	3-D X-Coord Diff	4139.3958	0.0095	-0.0690
111	75	3-D Y-Coord Diff	-2533.2419	0.0095	-0.0058
111	75	3-D Z-Coord Diff	18776.3362	0.0095	-0.0451
111	76	3-D X-Coord Diff	-16854.9736	0.0095	0.0054
111	76	3-D Y-Coord Diff	1885.5190	0.0095	0.0189
111	76	3-D Z-Coord Diff	12786.4504	0.0095	-0.0032
111	76	3-D X-Coord Diff	-16855.0130	0.0095	0.0449
111	76	3-D Y-Coord Diff	1885.5329	0.0095	0.0049
111	76	3-D Z-Coord Diff	12786.4449	0.0095	0.0024
111	77	3-D X-Coord Diff	-46935.8113	0.0095	-0.0299
111	77	3-D Y-Coord Diff	6285.5867	0.0095	0.0075
111	77	3-D Z-Coord Diff	7034.6717	0.0095	0.0004
111	78	3-D X-Coord Diff	-33773.3387	0.0095	-0.0311
111	78	3-D Y-Coord Diff	6041.2763	0.0095	-0.0002
111	78	3-D Z-Coord Diff	-12176.4364	0.0095	-0.0006
111	80	3-D X-Coord Diff	5311.5773	0.0095	0.0000
111	80	3-D Y-Coord Diff	248.3012	0.0095	0.0000
111	80	3-D Z-Coord Diff	-4707.6271	0.0095	0.0000
111	82	3-D X-Coord Diff	14126.6123	0.0095	0.0606
111	82	3-D Y-Coord Diff	-228.9987	0.0095	0.0012
111	82	3-D Z-Coord Diff	-21564.0267	0.0095	0.0022
111	82	3-D X-Coord Diff	14126.7089	0.0095	-0.0361
111	82	3-D Y-Coord Diff	-229.0131	0.0095	0.0156
111	82	3-D Z-Coord Diff	-21564.0194	0.0095	-0.0051
111	82	3-D X-Coord Diff	14126.7177	0.0095	-0.0449
111	82	3-D Y-Coord Diff	-228.9334	0.0095	-0.0641
111	82	3-D Z-Coord Diff	-21564.0151	0.0095	-0.0094
111	83	3-D X-Coord Diff	34445.1419	0.0095	0.0000
111	83	3-D Y-Coord Diff	83.0110	0.0095	0.0000
111	83	3-D Z-Coord Diff	-47404.6732	0.0095	0.0000
111	84	3-D X-Coord Diff	984.5177	0.0095	0.8583
111	84	3-D Y-Coord Diff	4923.0260	0.0095	5.3944
111	84	3-D Z-Coord Diff	-53648.3877	0.0095	0.6081
111	85	3-D X-Coord Diff	-38993.7265	0.0095	0.0134
111	85	3-D Y-Coord Diff	10743.5440	0.0095	0.0265
111	85	3-D Z-Coord Diff	-47657.8898	0.0095	0.0084

111	88	3-D X-Coord Diff	21412.1570	0.0095	0.0088
111	88	3-D Y-Coord Diff	5392.5472	0.0095	0.0621
111	88	3-D Z-Coord Diff	-88299.0526	0.0095	0.0060
111	88	3-D X-Coord Diff	21412.1539	0.0095	0.0119
111	88	3-D Y-Coord Diff	5392.6451	0.0095	-0.0358
111	88	3-D Z-Coord Diff	-88299.0394	0.0095	-0.0072
111	90	3-D X-Coord Diff	32080.8756	0.0095	0.0717
111	90	3-D Y-Coord Diff	2932.5397	0.0095	-0.0299
111	90	3-D Z-Coord Diff	-80063.3347	0.0095	-0.0112
111	91	3-D X-Coord Diff	48475.9345	0.0095	-0.0098
111	91	3-D Y-Coord Diff	-512.0900	0.0095	0.0028
111	91	3-D Z-Coord Diff	-71416.1593	0.0095	-0.0011
111	92	3-D X-Coord Diff	49813.7399	0.0095	0.0000
111	92	3-D Y-Coord Diff	-1852.5610	0.0095	0.0000
111	92	3-D Z-Coord Diff	-58608.2083	0.0095	0.0000
111	94	3-D X-Coord Diff	79555.7396	0.0095	0.1977
111	94	3-D Y-Coord Diff	-4353.0015	0.0095	-0.0970
111	94	3-D Z-Coord Diff	-87955.3753	0.0095	-0.0408
111	94	3-D X-Coord Diff	79556.1688	0.0095	-0.2314
111	94	3-D Y-Coord Diff	-4353.1895	0.0095	0.0910
111	94	3-D Z-Coord Diff	-87955.4520	0.0095	0.0359
111	96	3-D X-Coord Diff	91155.4792	0.0095	0.0891
111	96	3-D Y-Coord Diff	-7570.6751	0.0095	0.0964
111	96	3-D Z-Coord Diff	-77057.7777	0.0095	0.0322
111	99	3-D X-Coord Diff	102937.6885	0.0095	0.0416
111	99	3-D Y-Coord Diff	-15255.7512	0.0095	0.0111
111	99	3-D Z-Coord Diff	-26162.4078	0.0095	-0.0267
111	103	3-D X-Coord Diff	103075.4900	0.0095	0.0243
111	103	3-D Y-Coord Diff	-17923.5475	0.0095	-0.0070
111	103	3-D Z-Coord Diff	-4311.8818	0.0095	-0.0115
111	103	3-D X-Coord Diff	103075.5604	0.0095	-0.0460
111	103	3-D Y-Coord Diff	-17923.5887	0.0095	0.0342
111	103	3-D Z-Coord Diff	-4311.9146	0.0095	0.0213
111	108	3-D X-Coord Diff	107671.7354	0.0095	0.0139
111	108	3-D Y-Coord Diff	-24615.4441	0.0095	-0.0465
111	108	3-D Z-Coord Diff	42424.7954	0.0095	-0.0077
111	108	3-D X-Coord Diff	107671.7407	0.0095	0.0086
111	108	3-D Y-Coord Diff	-24615.5110	0.0095	0.0203
111	108	3-D Z-Coord Diff	42424.7903	0.0095	-0.0025
111	113	3-D X-Coord Diff	97572.7128	0.0095	0.0114
111	113	3-D Y-Coord Diff	-23376.0918	0.0095	-0.0041
111	113	3-D Z-Coord Diff	47518.9382	0.0095	0.0007
36	34	3-D X-Coord Diff	-743.2797	0.0095	0.3935
36	34	3-D Y-Coord Diff	-2234.9679	0.0095	0.2996
36	34	3-D Z-Coord Diff	15124.3148	0.0095	0.0090
36	40	3-D X-Coord Diff	-17741.1386	0.0095	-0.9397
36	40	3-D Y-Coord Diff	11016.0226	0.0095	0.2742
36	40	3-D Z-Coord Diff	-50141.6731	0.0095	-0.5406
37	38	3-D X-Coord Diff	-30826.2458	0.0095	0.0058
37	38	3-D Y-Coord Diff	5408.8775	0.0095	-0.0023
37	38	3-D Z-Coord Diff	844.8153	0.0095	-0.0045
37	61	3-D X-Coord Diff	-14872.7003	0.0095	0.0059

37	61	3-D Y-Coord Diff	9149.5093	0.0095	0.0001
37	61	3-D Z-Coord Diff	-42302.5713	0.0095	-0.0005
37	111	3-D X-Coord Diff	-75381.7445	0.0095	0.0172
37	111	3-D Y-Coord Diff	35856.3786	0.0095	0.0008
37	111	3-D Z-Coord Diff	-164741.7745	0.0095	-0.0036
38	34	3-D X-Coord Diff	2605.9022	0.0095	1.4144
38	34	3-D Y-Coord Diff	-6983.5723	0.0095	0.0148
38	34	3-D Z-Coord Diff	41304.9310	0.0095	0.5779
38	36	3-D X-Coord Diff	3349.2696	0.0095	0.9332
38	36	3-D Y-Coord Diff	-4748 `396	0.0095	-0.2496
38	36	3-D Z-Coord Diff	26180.6293	0.0095	0.5558
38	111	3-D X-Coord Diff	-44555.4257	0.0095	-0.0617
38	111	3-D Y-Coord Diff	30447.5316	0.0095	-0.0275
38	111	3-D Z-Coord Diff	-165586.5875	0.0095	-0.0014
39	37	3-D X-Coord Diff	256.8238	0.0095	0.0136
39	37	3-D Y-Coord Diff	-4144.6438	0.0095	0.0267
39	37	3-D Z-Coord Diff	26799.4891	0.0095	-0.0025
39	38	3-D X-Coord Diff	-30569.4189	0.0095	0.0163
39	38	3-D Y-Coord Diff	1264.2316	0.0095	0.0264
39	38	3-D Z-Coord Diff	27644.3008	0.0095	-0.0034
39	47	3-D X-Coord Diff	12058.1884	0.0095	-0.0242
39	47	3-D Y-Coord Diff	-2233.8095	0.0095	-0.0557
39	47	3-D Z-Coord Diff	886.6239	0.0095	0.0102
39	48	3-D X-Coord Diff	38836.3173	0.0095	-0.0151
39	48	3-D Y-Coord Diff	-6485.1138	0.0095	-0.0395
39	48	3-D Z-Coord Diff	-3702.9729	0.0095	0.0146
40	34	3-D X-Coord Diff	16997.7860	0.0095	1.4061
40	34	3-D Y-Coord Diff	-13250.9521	0.0095	-0.0130
40	34	3-D Z-Coord Diff	65265.9713	0.0095	0.5661
40	38	3-D X-Coord Diff	14391.8804	0.0095	-0.0048
40	38	3-D Y-Coord Diff	-6267.3875	0.0095	-0.0201
40	38	3-D Z-Coord Diff	23961.0412	0.0095	-0.0127
47	48	3-D X-Coord Diff	26778.1379	0.0095	0.0000
47	48	3-D Y-Coord Diff	-4 251.2884	0.0095	0.0003
47	48	3-D Z-Coord Diff	-4589.5906	0.0095	-0.0018
47	49	3-D X-Coord Diff	14441.7647	0.0095	0.0000
47	49	3-D Y-Coord Diff	151.6267	0.0095	0.0000
47	49	3-D Z-Coord Diff	-19521.2386	0.0095	0.0000
47	111	3-D X-Coord Diff	-87183.0190	0.0095	-0.0351
47	111	3-D Y-Coord Diff	33945.6470	0.0095	-0.0196
47	111	3-D Z-Coord Diff	-138828.9316	0.0095	0.0060
49	39	3-D X-Coord Diff	-26499.9347	0.0095	0.0057
49	39	3-D Y-Coord Diff	2082.2123	0.0095	0.0262
49	39	3-D Z-Coord Diff	18634.6081	0.0095	-0.0036
49	111	3-D X-Coord Diff	-101624.7997	0.0095	-0.0191
49	111	3-D Y-Coord Diff	33794.0251	0.0095	-0.0244
49	111	3-D Z-Coord Diff	-119307.6949	0.0095	0.0079
51	40	3-D X-Coord Diff	-58819.8073	0.0095	-0.0449
51	40	3-D Y-Coord Diff	2217.9061	0.0095	-0.0450
51	40	3-D Z-Coord Diff	58700.1153	0.0095	-0.0061
51	111	3-D X-Coord Diff	-88983.4884	0.0095	0.0244
51	111	3-D Y-Coord Diff	26397.9588	0.0095	-0.0011

51	111	3-D Z-Coord Diff	-82925.4449	0.0095	-0.0062
56	51	3-D X-Coord Diff	-1921.3686	0.0095	-3.9595
56	51	3-D Y-Coord Diff	-3088.2330	0.0095	4.6753
56	51	3-D Z-Coord Diff	26677.4416	0.0095	4.5601
56	111	3-D X-Coord Diff	-90908.9236	0.0095	0.1315
56	111	3-D Y-Coord Diff	23314.3379	0.0095	0.0622
56	111	3-D Z-Coord Diff	-56243.4620	0.0095	0.0126
57	51	3-D X-Coord Diff	7397.1416	0.0095	-4.0198
57	51	3-D Y-Coord Diff	-6178.4643	0.0095	4.6392
57	51	3-D Z-Coord Diff	37156.6027	0.0095	4.6928
57	56	3-D X-Coord Diff	9318.4373	0.0095	0.0126
57	56	3-D Y-Coord Diff	-3090.1658	0.0095	-0.1017
57	56	3-D Z-Coord Diff	10479.3153	0.0095	-0.0214
57	108	3-D X-Coord Diff	26081.4182	0.0095	-0.0111
57	108	3-D Y-Coord Diff	-4391.4351	0.0095	0.0771
57	108	3-D Z-Coord Diff	-3339.3877	0.0095	0.0199
57	113	3-D X-Coord Diff	15982.3946	0.0095	-0.0125
57	113	3-D Y-Coord Diff	-3152.0660	0.0095	0.1027
57	113	3-D Z-Coord Diff	1754.7615	0.0095	0.0219
108	113	3-D X-Coord Diff	-10099.0254	0.0095	0.0004
108	113	3-D Y-Coord Diff	1239.3730	0.0095	0.0217
108	113	3-D Z-Coord Diff	5094.1850	0.0095	-0.0338
60	40	3-D X-Coord Diff	-37671.5774	0.0095	-0.0383
60	40	3-D Y-Coord Diff	-1501.3552	0.0095	-0.0370
60	40	3-D Z-Coord Diff	57705.2889	0.0095	-0.0201
60	51	3-D X-Coord Diff	21148.2279	0.0095	0.0086
60	51	3-D Y-Coord Diff	-3719.2620	0.0095	0.0086
60	51	3-D Z-Coord Diff	-994.8266	0.0095	-0.0139
61	39	3-D X-Coord Diff	14615.8705	0.0095	-0.0136
61	39	3-D Y-Coord Diff	-5004.8655	0.0095	-0.0267
61	39	3-D Z-Coord Diff	15503.0827	0.0095	0.0025
61	38	3-D X-Coord Diff	-15953.5497	0.0095	0.0041
61	38	3-D Y-Coord Diff	-3740.6346	0.0095	0.0004
61	38	3-D Z-Coord Diff	43147.3895	0.0095	-0.0070
73	75	3-D X-Coord Diff	26558.2532	0.0095	0.0202
73	75	3-D Y-Coord Diff	-2483.2585	0.0095	0.0058
73	75	3-D Z-Coord Diff	-12264.1419	0.0095	0.0172
73	111	3-D X-Coord Diff	22418.9609	0.0095	-0.0142
73	111	3-D Y-Coord Diff	50.0141	0.0095	-0.0191
73	111	3-D Z-Coord Diff	-31040.3969	0.0095	-0.0189
73	74	3-D X-Coord Diff	37804.5659	0.0095	0.0849
73	74	3-D Y-Coord Diff	-9837.9983	0.0095	0.0025
73	74	3-D Z-Coord Diff	32579.1273	0.0095	-0.0581
74	59	3-D X-Coord Diff	24684.4023	0.0095	0.0626
74	59	3-D Y-Coord Diff	-7730.2500	0.0095	0.0528
74	59	3-D Z-Coord Diff	22790.3980	0.0095	-0.0043
74	60	3-D X-Coord Diff	52449.5145	0.0095	0.0089
74	60	3-D Y-Coord Diff	-12790.7197	0.0095	0.0061
74	60	3-D Z-Coord Diff	20300.8293	0.0095	-0.0228
75	73	3-D X-Coord Diff	-26558.3182	0.0095	0.0447
75	73	3-D Y-Coord Diff	2483.2413	0.0095	0.0115
75	73	3-D Z-Coord Diff	12264.1348	0.0095	-0.0101

75	74	3-D X-Coord Diff	11246.4148	0.0095	-0.0374
75	74	3-D Y-Coord Diff	-7354.7478	0.0095	0.0049
75	74	3-D Z-Coord Diff	44843.1893	0.0095	0.0046
76	73	3-D X-Coord Diff	-5563.9769	0.0095	-0.0017
76	73	3-D Y-Coord Diff	-1935.5222	0.0095	-0.0106
76	73	3-D Z-Coord Diff	18254.0119	0.0095	-0.0434
76	75	3-D X-Coord Diff	20994.2689	0.0095	0.0260
76	75	3-D Y-Coord Diff	-4418.7981	0.0095	0.0125
76	75	3-D Z-Coord Diff	5989.8056	0.0095	0.0383
76	77	3-D X-Coord Diff	-30080.8334	0.0095	-0.0397
76	77	3-D Y-Coord Diff	4400.0968	0.0095	-0.0405
76	77	3-D Z-Coord Diff	-5751.7823	0.0095	0.0071
76	78	3-D X-Coord Diff	-16918.3370	0.0095	-0.0647
76	78	3-D Y-Coord Diff	4155.7405	0.0095	-0.0023
76	78	3-D Z-Coord Diff	-24962.8853	0.0095	0.0010
76	82	3-D X-Coord Diff	30981.6155	0.0095	0.0255
76	82	3-D Y-Coord Diff	-2114.5600	0.0095	0.0247
76	82	3-D Z-Coord Diff	-34350.4766	0.0095	0.0048
78	77	3-D X-Coord Diff	-13162.5008	0.0095	0.0293
78	77	3-D Y-Coord Diff	244.3234	0.0095	-0.0053
78	77	3-D Z-Coord Diff	19211.1023	0.0095	0.0067
78	82	3-D X-Coord Diff	47899.9729	0.0095	0.0697
78	82	3-D Y-Coord Diff	-6270.2621	0.0095	-0.0114
78	82	3-D Z-Coord Diff	-9387.5729	0.0095	-0.0146
80	73	3-D X-Coord Diff	-27730.5104	0.0095	-0.0136
80	73	3-D Y-Coord Diff	-298.2663	0.0095	-0.0298
80	73	3-D Z-Coord Diff	35748.0245	0.0095	0.0184
80	75	3-D X-Coord Diff	-1172.2633	0.0095	0.0127
80	75	3-D Y-Coord Diff	-2781.5368	0.0095	-0.0120
80	75	3-D Z-Coord Diff	23483.8789	0.0095	0.0393
80	76	3-D X-Coord Diff	-22166.5313	0.0095	-0.0142
80	76	3-D Y-Coord Diff	1637.2659	0.0095	-0.0292
80	76	3-D Z-Coord Diff	17494.0760	0.0095	-0.0016
82	77	3-D X-Coord Diff	-61062.4164	0.0095	-0.0977
82	77	3-D Y-Coord Diff	6514.6580	0.0095	-0.0663
82	77	3-D Z-Coord Diff	28598.6864	0.0095	0.0101
82	77	3-D X-Coord Diff	-61062.5304	0.0095	0.0163
82	77	3-D Y-Coord Diff	6514.5323	0.0095	0.0594
82	77	3-D Z-Coord Diff	28598.6892	0.0095	0.0074
82	84	3-D X-Coord Diff	-13142.1548	0.0095	0.8579
82	84	3-D Y-Coord Diff	5152.0842	0.0095	5.3337
82	84	3-D Z-Coord Diff	-32084.3497	0.0095	0.5946
82	85	3-D X-Coord Diff	-53120.4287	0.0095	0.0427
82	85	3-D Y-Coord Diff	10972.5685	0.0095	-0.0005
82	85	3-D Z-Coord Diff	-26093.8637	0.0095	0.0068
83	91	3-D X-Coord Diff	14030.7828	0.0095	0.0000
83	91	3-D Y-Coord Diff	-595.0983	0.0095	0.0000
83	91	3-D Z-Coord Diff	-24011.4872	0.0095	0.0000
83	88	3-D X-Coord Diff	-13033.0076	0.0095	0.0316
83	88	3-D Y-Coord Diff	5309.5322	0.0095	0.0660
83	88	3-D Z-Coord Diff	-40894.3846	0.0095	0.0111
84	86	3-D X-Coord Diff	-4744.9556	0.0095	-0.8219

84	86	3-D Y-Coord Diff	1898.8033	0.0095	-5.3493
84	86	3-D Z-Coord Diff	-10129.1106	0.0095	-0.5896
84	85	3-D X-Coord Diff	-39978.2708	0.0095	-0.8183
84	85	3-D Y-Coord Diff	5820.4905	0.0095	-5.3404
84	85	3-D Z-Coord Diff	5990.4905	0.0095	-0.5922
84	111	3-D X-Coord Diff	-984.5384	0.0095	-0.8376
84	111	3-D Y-Coord Diff	-4923.0524	0.0095	-5.3680
84	111	3-D Z-Coord Diff	53648.3791	0.0095	-0.5995
84	88	3-D X-Coord Diff	20427.6453	0.0095	-0.8554
84	88	3-D Y-Coord Diff	469.5139	0.0095	-5.3250
84	88	3-D Z-Coord Diff	-34650.6634	0.0095	-0.6036
84	89	3-D X-Coord Diff	-17916.1757	0.0095	-0.8583
84	89	3-D Y-Coord Diff	5279.4244	0.0095	-5.3944
84	89	3-D Z-Coord Diff	-22899.6512	0.0095	-0.6081
84	112	3-D X-Coord Diff	16048.6905	0.0095	-0.8583
84	112	3-D Y-Coord Diff	-973.6772	0.0095	-5.3944
84	112	3-D Z-Coord Diff	-12909.2877	0.0095	-0.6081
86	82	3-D X-Coord Diff	17887.1058	0.0095	-0.0315
86	82	3-D Y-Coord Diff	-7050.7981	0.0095	-0.0738
86	82	3-D Z-Coord Diff	42213.4520	0.0095	0.0033
86	85	3-D X-Coord Diff	-35233.3275	0.0095	0.0159
86	85	3-D Y-Coord Diff	3921.7000	0.0095	-0.0040
86	85	3-D Z-Coord Diff	16119.5847	0.0095	0.0137
86	111	3-D X-Coord Diff	3760.4066	0.0095	-0.0052
86	111	3-D Y-Coord Diff	-6821.8459	0.0095	-0.0285
86	111	3-D Z-Coord Diff	63777.4734	0.0095	0.0064
89	88	3-D X-Coord Diff	38343.8118	0.0095	0.0120
89	88	3-D Y-Coord Diff	-4809.9095	0.0095	0.0684
89	88	3-D Z-Coord Diff	-11751.0166	0.0095	0.0089
89	111	3-D X-Coord Diff	16931.6538	0.0095	0.0041
89	111	3-D Y-Coord Diff	-10202.4546	0.0095	0.0042
89	111	3-D Z-Coord Diff	76548.0368	0.0095	0.0021
112	88	3-D X-Coord Diff	4378.9253	0.0095	0.0324
112	88	3-D Y-Coord Diff	1443.1953	0.0095	0.0651
112	88	3-D Z-Coord Diff	-21741.3607	0.0095	-0.0106
112	89	3-D X-Coord Diff	-33964.8178	0.0095	-0.0483
112	89	3-D Y-Coord Diff	6253.0982	0.0095	0.0033
112	89	3-D Z-Coord Diff	-9990.3554	0.0095	-0.0082
112	111	3-D X-Coord Diff	-17033.1786	0.0095	-0.0296
112	111	3-D Y-Coord Diff	-3949.3639	0.0095	0.0150
112	111	3-D Z-Coord Diff	66557.6763	0.0095	-0.0009
90	88	3-D X-Coord Diff	-10668.7640	0.0095	-0.0176
90	88	3-D Y-Coord Diff	2460.1539	0.0095	-0.0544
90	88	3-D Z-Coord Diff	-8235.7040	0.0095	0.0033
90	83	3-D X-Coord Diff	2364.2410	0.0095	-0.0465
90	83	3-D Y-Coord Diff	-2849.4963	0.0095	-0.0025
90	83	3-D Z-Coord Diff	32658.6849	0.0095	-0.0122
90	111	3-D X-Coord Diff	-32080.9011	0.0095	-0.0462
90	111	3-D Y-Coord Diff	-2932.5071	0.0095	-0.0027
90	111	3-D Z-Coord Diff	80063.3578	0.0095	-0.0119
90	93	3-D X-Coord Diff	30296.1621	0.0095	-0.0733
90	93	3-D Y-Coord Diff	-5288.0770	0.0095	0.0302

90	93	3-D Z-Coord Diff	3935.6454	0.0095	0.0112
90	94	3-D X-Coord Diff	47474.8955	0.0095	0.0944
90	94	3-D Y-Coord Diff	-7285.5284	0.0095	-0.0799
90	94	3-D Z-Coord Diff	-7892.0585	0.0095	-0.0118
90	94	3-D X-Coord Diff	47475.1281	0.0095	-0.1381
90	94	3-D Y-Coord Diff	-7285.7219	0.0095	0.1137
90	94	3-D Z-Coord Diff	-7892.0825	0.0095	0.0122
90	95	3-D X-Coord Diff	48544.2284	0.0095	-0.1183
90	95	3-D Y-Coord Diff	-8175.5086	0.0095	-0.1534
90	95	3-D Z-Coord Diff	-1154.0316	0.0095	-0.2112
90	96	3-D X-Coord Diff	59074.5314	0.0095	0.0896
90	96	3-D Y-Coord Diff	-10503.0540	0.0095	-0.0345
90	96	3-D Z-Coord Diff	3005.6026	0.0095	-0.0022
90	96	3-D X-Coord Diff	59074.5991	0.0095	0.0219
90	96	3-D Y-Coord Diff	-10503.0542	0.0095	-0.0343
90	96	3-D Z-Coord Diff	3005.5989	0.0095	0.0015
90	111	3-D X-Coord Diff	-32080.9864	0.0095	0.0390
90	111	3-D Y-Coord Diff	-2932.5419	0.0095	0.0322
90	111	3-D Z-Coord Diff	80063.3048	0.0095	0.0411
90	111	3-D X-Coord Diff	-32080.9755	0.0095	0.0281
90	111	3-D Y-Coord Diff	-2932.5477	0.0095	0.0379
90	111	3-D Z-Coord Diff	80063.3888	0.0095	-0.0429
91	88	3-D X-Coord Diff	-27063.8036	0.0095	0.0447
91	88	3-D Y-Coord Diff	5904.7338	0.0095	-0.0373
91	88	3-D Z-Coord Diff	-16882.8908	0.0095	0.0046
91	90	3-D X-Coord Diff	-16395.0365	0.0095	0.0592
91	90	3-D Y-Coord Diff	3444.6027	0.0095	-0.0057
91	90	3-D Z-Coord Diff	-8647.2329	0.0095	0.0474
92	96	3-D X-Coord Diff	41341.7167	0.0095	0.1117
92	96	3-D Y-Coord Diff	-5718.1202	0.0095	0.1025
92	96	3-D Z-Coord Diff	-18449.5708	0.0095	0.0336
92	97	3-D X-Coord Diff	44484.8982	0.0095	0.0066
92	97	3-D Y-Coord Diff	-8096.7767	0.0095	-0.0174
92	97	3-D Z-Coord Diff	-1658.1108	0.0095	-0.0008
92	98	3-D X-Coord Diff	23291.7416	0.0095	0.0000
92	98	3-D Y-Coord Diff	-4588.1276	0.0095	0.0000
92	98	3-D Z-Coord Diff	5219.4411	0.0095	0.0000
93	111	3-D X-Coord Diff	-62377.0481	0.0095	0.0118
93	111	3-D Y-Coord Diff	2355.5505	0.0095	-0.0135
93	111	3-D Z-Coord Diff	76127.6947	0.0095	-0.0054
95	96	3-D X-Coord Diff	10530.2221	0.0095	0.2887
95	96	3-D Y-Coord Diff	-2327.3482	0.0095	-0.0784
95	96	3-D Z-Coord Diff	4159.6404	0.0095	0.2028
95	111	3-D X-Coord Diff	-80624.8727	0.0095	-0.1848
95	111	3-D Y-Coord Diff	5242.8362	0.0095	0.3159
95	111	3-D Z-Coord Diff	81217.2850	0.0095	0.3036
96	93	3-D X-Coord Diff	-28778.3764	0.0095	-0.1557
96	93	3-D Y-Coord Diff	5214.9714	0.0095	0.0703
96	93	3-D Z-Coord Diff	930.0449	0.0095	0.0113
96	94	3-D X-Coord Diff	-11599.7083	0.0095	0.0772
96	94	3-D Y-Coord Diff	3217.5074	0.0095	-0.0271
96	94	3-D Z-Coord Diff	-10897.6750	0.0095	0.0044

96	111	3-D X-Coord Diff	-91155.4241	0.0095	-0.1443
96	111	3-D Y-Coord Diff	7570.5209	0.0095	0.0578
96	111	3-D Z-Coord Diff	77057.7410	0.0095	0.0045
96	111	3-D X-Coord Diff	-91155.5925	0.0095	0.0241
96	111	3-D Y-Coord Diff	7570.5643	0.0095	0.0144
96	111	3-D Z-Coord Diff	77057.7216	0.0095	0.0239
96	111	3-D X-Coord Diff	-91155.8096	0.0095	0.2412
96	111	3-D Y-Coord Diff	7570.6029	0.0095	-0.0242
96	111	3-D Z-Coord Diff	77057.7465	0.0095	-0.0010
97	96	3-D X-Coord Diff	-3143.1874	0.0095	0.1111
97	96	3-D Y-Coord Diff	2378.6778	0.0095	0.0986
97	96	3-D Z-Coord Diff	-16791.4961	0.0095	0.0705
97	111	3-D X-Coord Diff	-94298.5749	0.0095	-0.0698
97	111	3-D Y-Coord Diff	9949.3385	0.0095	0.0166
97	111	3-D Z-Coord Diff	60266.3147	0.0095	0.0053
98	96	3-D X-Coord Diff	18049.9850	0.0095	0.1018
98	96	3-D Y-Coord Diff	-1130.0125	0.0095	0.1224
98	96	3-D Z-Coord Diff	-23669.0135	0.0095	0.0352
98	97	3-D X-Coord Diff	21193.1632	0.0095	0.0000
98	97	3-D Y-Coord Diff	-3508.6665	0.0095	0.0000
98	97	3-D Z-Coord Diff	-6877.5527	0.0095	0.0000
98	111	3-D X-Coord Diff	-73105.5052	0.0095	0.0237
98	111	3-D Y-Coord Diff	6440.6823	0.0095	0.0063
98	111	3-D Z-Coord Diff	53388.7664	0.0095	0.0009
103	107	3-D X-Coord Diff	1922.8517	0.0095	-0.0243
103	107	3-D Y-Coord Diff	-4860.8134	0.0095	0.0070
103	107	3-D Z-Coord Diff	36186.1523	0.0095	0.0115
103	108	3-D X-Coord Diff	4596.2567	0.0095	-0.0218
103	108	3-D Y-Coord Diff	-6691.9643	0.0095	0.0281
103	108	3-D Z-Coord Diff	46736.6703	0.0095	0.0107
103	99	3-D X-Coord Diff	-137.8304	0.0095	0.0460
103	99	3-D Y-Coord Diff	2667.8486	0.0095	-0.0342
103	99	3-D Z-Coord Diff	-21850.5198	0.0095	-0.0213
107	108	3-D X-Coord Diff	2673.4726	0.0095	-0.0650
107	108	3-D Y-Coord Diff	-1831.0844	0.0095	-0.0453
107	108	3-D Z-Coord Diff	10550.5371	0.0095	-0.0198
107	111	3-D X-Coord Diff	-104998.3900	0.0095	0.0483
107	111	3-D Y-Coord Diff	22784.3503	0.0095	0.0106
107	111	3-D Z-Coord Diff	-31874.2627	0.0095	-0.0077

FORMEQ successfully completed.

Iowa State University, Dept. of Civil Eng.
GeoLab Adjustment of Sri Lanka full figure with Fig 1&2 and HDC 10/11
A= 6377299.151 B= 6356098.145 X0= 0.000 Y0= 0.000 Z0= 0.000

SOLVE:

Adjusted Values (Iteration Count = 2):

CODE IDENT. TYPE INITIAL DX ADJUSTED

14	84	LATITUDE	6 19 49.093490	FIXED
14	84	LONGITUDE	80 57 34.439360	FIXED
14	84	HEIGHT	8.00000	FIXED
14	38	LATITUDE	8 19 43.897530	FIXED
14	38	LONGITUDE	80 31 6.362270	FIXED
14	38	HEIGHT	302.11000	FIXED
14	39	LATITUDE	8 4 35.293760	FIXED
14	39	LONGITUDE	80 14 34.661190	FIXED
14	39	HEIGHT	180.15700	FIXED
14	111	LATITUDE	6 49 2.700190	FIXED
14	111	LONGITUDE	80 57 40.911150	FIXED
14	111	HEIGHT	1160.81100	FIXED
14	103	LATITUDE	6 46 45.806100	FIXED
14	103	LONGITUDE	80 0 53.587090	FIXED
14	103	HEIGHT	11.19600	FIXED
14	108	LATITUDE	7 12 18.957290	FIXED
14	108	LONGITUDE	79 57 48.195020	FIXED
14	108	HEIGHT	-47.24000	FIXED
14	40	LATITUDE	8 6 34.142440	FIXED
14	40	LONGITUDE	80 39 23.771130	FIXED
14	40	HEIGHT	666.67500	FIXED
14	60	LATITUDE	7 34 59.123250	FIXED
14	60	LONGITUDE	80 19 18.989210	FIXED
14	60	HEIGHT	422.20000	FIXED
14	90	LATITUDE	6 5 23.631980	FIXED
14	90	LONGITUDE	80 40 45.405340	FIXED
14	90	HEIGHT	123.81200	FIXED
14	82	LATITUDE	6 37 17.947630	FIXED
14	82	LONGITUDE	80 50 5.540510	FIXED
14	82	HEIGHT	632.41300	FIXED
14	73	LATITUDE	7 5 59.504070	FIXED
14	73	LONGITUDE	81 9 42.098890	FIXED
14	73	HEIGHT	1414.76000	FIXED
14	57	LATITUDE	7 14 7.484730	FIXED
14	57	LONGITUDE	80 12 10.400460	FIXED
14	57	HEIGHT	207.81700	FIXED
14	74	LATITUDE	7 23 46.939820	FIXED
14	74	LONGITUDE	80 48 34.853770	FIXED
14	74	HEIGHT	1762.22600	FIXED

14	75	LATITUDE	6 59 16.637850	FIXED	
14	75	LONGITUDE	80 55 14.778510	FIXED	
14	75	HEIGHT	1580.90600	FIXED	
14	76	LATITUDE	6 55 58.843190	FIXED	
14	76	LONGITUDE	81 6 52.715810	FIXED	
14	76	HEIGHT	1933.79400	FIXED	
14	88	LATITUDE	6 0 54.571720	FIXED	
14	88	LONGITUDE	80 46 40.749180	FIXED	
14	88	HEIGHT	-39.40700	FIXED	
14	94	LATITUDE	6 1 5.791600	FIXED	
14	94	LONGITUDE	80 14 43.426960	FIXED	
14	94	HEIGHT	-30.82700	FIXED	
14	96	LATITUDE	6 7 2.702540	FIXED	
14	96	LONGITUDE	80 8 13.837510	FIXED	
14	96	HEIGHT	-66.52800	FIXED	
4	34	LATITUDE	8 42 24.759336	-0.000000	8 42 24.759336
4	34	LONGITUDE	80 29 4.613210	0.000000	80 29 4.613210
4	34	HEIGHT	33.63347	0.00000	33.63347
4	36	LATITUDE	8 34 6.637746	-0.000000	8 34 6.637746
4	36	LONGITUDE	80 28 52.722915	-0.000000	80 28 52.722915
4	36	HEIGHT	62.80333	0.00000	62.80333
4	37	LATITUDE	8 19 17.532289	0.000000	8 19 17.532289
4	37	LONGITUDE	80 14 3.429650	-0.000000	80 14 3.429650
4	37	HEIGHT	2.30618	-0.00000	2.30618
4	47	LATITUDE	8 5 4.537831	0.000000	8 5 4.537831
4	47	LONGITUDE	80 7 54.108621	0.000000	80 7 54.108621
4	47	HEIGHT	160.10834	-0.00000	160.10834
4	48	LATITUDE	8 2 34.513141	-0.000000	8 2 34.513141
4	48	LONGITUDE	79 53 8.595875	0.000000	79 53 8.595875
4	48	HEIGHT	-29.07986	0.00000	-29.07986
4	49	LATITUDE	7 54 23.286694	0.000000	7 54 23.286694
4	49	LONGITUDE	80 0 10.386147	0.000000	80 0 10.386147
4	49	HEIGHT	59.54445	0.00000	59.54445
4	51	LATITUDE	7 34 27.294462	-0.000000	7 34 27.294462
4	51	LONGITUDE	80 7 38.385883	-0.000000	80 7 38.385883
4	51	HEIGHT	215.45975	0.00000	215.45975
4	56	LATITUDE	7 19 51.893230	-0.000000	7 19 51.893230
4	56	LONGITUDE	80 6 53.861550	-0.000000	80 6 53.861550
4	56	HEIGHT	96.93900	0.00000	96.93900

4	59	LATITUDE	7 36 17.797824	-0.000000	7 36 17.797824
4	59	LONGITUDE	80 34 39.633671	0.000000	80 34 39.633671
4	59	HEIGHT	1131.61892	0.00000	1131.61892
4	61	LATITUDE	7 56 4.343344	-0.000000	7 56 4.343344
4	61	LONGITUDE	80 22 52.686741	-0.000000	80 22 52.686741
4	61	HEIGHT	470.90456	0.00000	470.90456
4	77	LATITUDE	6 52 53.880656	-0.000000	6 52 53.880656
4	77	LONGITUDE	81 23 22.877557	0.000000	81 23 22.877557
4	77	HEIGHT	1017.70810	0.00000	1017.70810
4	78	LATITUDE	6 42 26.214378	-0.000000	6 42 26.214378
4	78	LONGITUDE	81 16 17.911887	0.000000	81 16 17.911887
4	78	HEIGHT	475.11392	0.00000	475.11392
4	80	LATITUDE	6 46 26.387393	0.000000	6 46 26.387393
4	80	LONGITUDE	80 54 51.379140	-0.000000	80 54 51.379140
4	80	HEIGHT	1677.95166	0.00000	1677.95166
4	83	LATITUDE	6 23 9.044654	0.000000	6 23 9.044654
4	83	LONGITUDE	80 39 14.478108	0.000000	80 39 14.478108
4	83	HEIGHT	1258.43458	-0.00000	1258.43458
4	85	LATITUDE	6 23 4.374667	0.000000	6 23 4.374667
4	85	LONGITUDE	81 19 29.052956	-0.000000	81 19 29.052956
4	85	HEIGHT	265.25578	-0.00000	265.25578
4	86	LATITUDE	6 14 17.281846	-0.000000	6 14 17.281846
4	86	LONGITUDE	81 0 16.604902	-0.000000	81 0 16.604902
4	86	HEIGHT	20.39566	-0.00000	20.39566
4	89	LATITUDE	6 7 19.403033	-0.000000	6 7 19.403033
4	89	LONGITUDE	81 7 36.966143	0.000000	81 7 36.966143
4	89	HEIGHT	-67.79383	-0.00000	-67.79383
4	91	LATITUDE	6 10 6.039814	0.000000	6 10 6.039814
4	91	LONGITUDE	80 31 40.984669	-0.000000	80 31 40.984669
4	91	HEIGHT	329.50302	-0.00000	329.50302
4	92	LATITUDE	6 17 4.413811	-0.000000	6 17 4.413811
4	92	LONGITUDE	80 30 50.875534	-0.000000	80 30 50.875534
4	92	HEIGHT	623.33939	0.00000	623.33939
4	93	LATITUDE	6 7 31.850103	-0.000000	6 7 31.850103
4	93	LONGITUDE	80 24 5.124870	-0.000000	80 24 5.124870
4	93	HEIGHT	306.99007	0.00000	306.99007
4	95	LATITUDE	6 4 46.323388	-0.000000	6 4 46.323388
4	95	LONGITUDE	80 14 4.249645	0.000000	80 14 4.249645
4	95	HEIGHT	-12.70449	0.00000	-12.70449

4	97	LATITUDE	6 16 12.515761	-0.000000	6 16 12.515761
4	97	LONGITUDE	80 6 19.823637	0.000000	80 6 19.823637
4	97	HEIGHT	-48.54992	0.00000	-48.54992
4	98	LATITUDE	6 19 55.590078	-0.000000	6 19 55.590078
4	98	LONGITUDE	80 17 58.767313	0.000000	80 17 58.767313
4	98	HEIGHT	558.34642	-0.00000	558.34642
4	99	LATITUDE	6 34 49.428112	-0.000000	6 34 49.428112
4	99	LONGITUDE	80 1 13.066260	-0.000000	80 1 13.066260
4	99	HEIGHT	55.36747	0.00000	55.36747
4	107	LATITUDE	7 6 32.735574	-0.000000	7 6 32.735574
4	107	LONGITUDE	79 59 24.396100	-0.000000	79 59 24.396100
4	107	HEIGHT	-34.30576	0.00000	-34.30576
4	112	LATITUDE	6 12 45.720638	0.000000	6 12 45.720638
4	112	LONGITUDE	80 48 53.842491	-0.000000	80 48 53.842491
4	112	HEIGHT	165.05970	0.00000	165.05970
4	113	LATITUDE	7 15 5.654234	0.000000	7 15 5.654234
4	113	LONGITUDE	80 3 19.449968	0.000000	80 3 19.449968
4	113	HEIGHT	66.47236	-0.00000	66.47236

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GeoLab Adjustment of Sri Lanka full figure with Fig 1&2 and HDC 10/11

A= 6377299.151 B= 6356098.145 X0= 0.000 Y0= 0.000 Z0= 0.000

Adjusted Cartesian Coordinates:

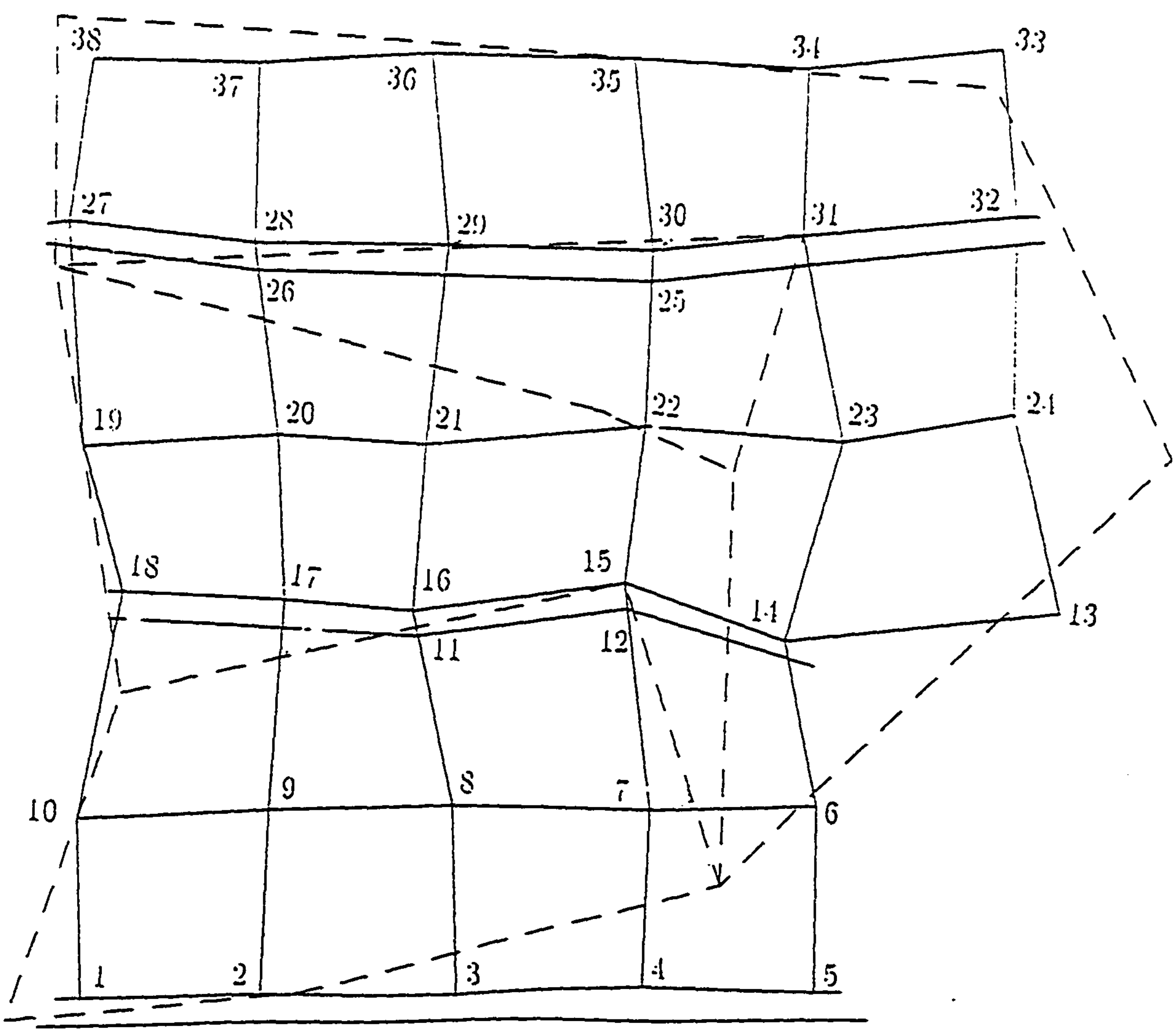
CODE	IDENT.	X-COORDINATE	Y-COORDINATE	Z-COORDINATE
14	111	995020.3642	6255009.6592	752170.3884
4	34	1042182.2124	6217578.5720	959062.1062
4	36	1042925.4455	6219813.4653	943937.7955
4	37	1070402.0946	6219153.2706	916912.1646
14	38	1039575.8516	6224562.1550	917756.9773
14	39	1070145.2541	6223297.8970	890112.6799
14	40	1025183.9760	6230829.5626	893795.9487
4	47	1082203.4162	6221064.0464	890999.3101
4	48	1108981.5637	6216812.7603	886409.7119
4	49	1096645.1778	6221215.6639	871478.0756
4	51	1084005.2575	6228610.0394	835094.1852
4	56	1085928.3525	6231696.3182	808414.8177
14	57	1076610.7064	6234785.5266	797934.5440
4	59	1035090.4754	6237391.4208	838580.2649
14	60	1062855.5917	6232330.9549	836090.6799
4	61	1055529.3941	6228302.7803	874609.5932
14	73	972601.4175	6254959.6643	783210.8042
14	74	1010406.0683	6245121.6685	815789.8735

14	75	999159.6910	6252476.4115	770946.6795
14	76	978165.3961	6256895.1971	764956.8356
4	77	948084.5588	6261295.2622	759205.0533
4	78	961247.0521	6261050.9339	739993.9475
4	80	1000331.9377	6255257.9426	747462.7753
14	82	1009147.0371	6254780.6617	730606.3639
4	83	1029465.5345	6255092.6855	704765.7250
14	84	996005.7402	6259938.0796	698522.6088
4	85	956026.9035	6265754.9974	704512.6981
4	86	991260.2261	6261833.2863	688393.1096
14	88	1016432.5301	6260402.2685	663871.3418
4	89	978089.0065	6265213.9318	675622.5560
14	90	1027101.3116	6257942.1690	672107.0425
4	91	1043496.3244	6254497.5643	680754.2437
4	92	1044834.1557	6253157.1498	693562.2012
4	93	1057397.4807	6252654.0842	676042.6898
14	94	1074576.3015	6250656.5608	664214.9723
4	95	1075645.4958	6249766.6374	670953.0389
14	96	1086175.9326	6247439.0805	675112.6429
4	97	1089319.0437	6245060.3652	691904.0983
4	98	1068125.8987	6248569.0309	698781.6429
4	99	1097958.0504	6239753.9306	726007.9780
14	103	1098095.8785	6237086.1047	747858.4951
4	107	1100018.7084	6232225.2844	784044.6458
14	108	1102692.1135	6230394.1685	794595.1762
4	112	1012053.8507	6258960.8331	685612.9117
4	113	1092593.0887	6231633.5232	799689.3311

APPENDIX G RESULTS OF SCANNING AND VECTORIZATION OF CADASTRAL MAPS

Simulated cadastral map used for scanning and vectorization

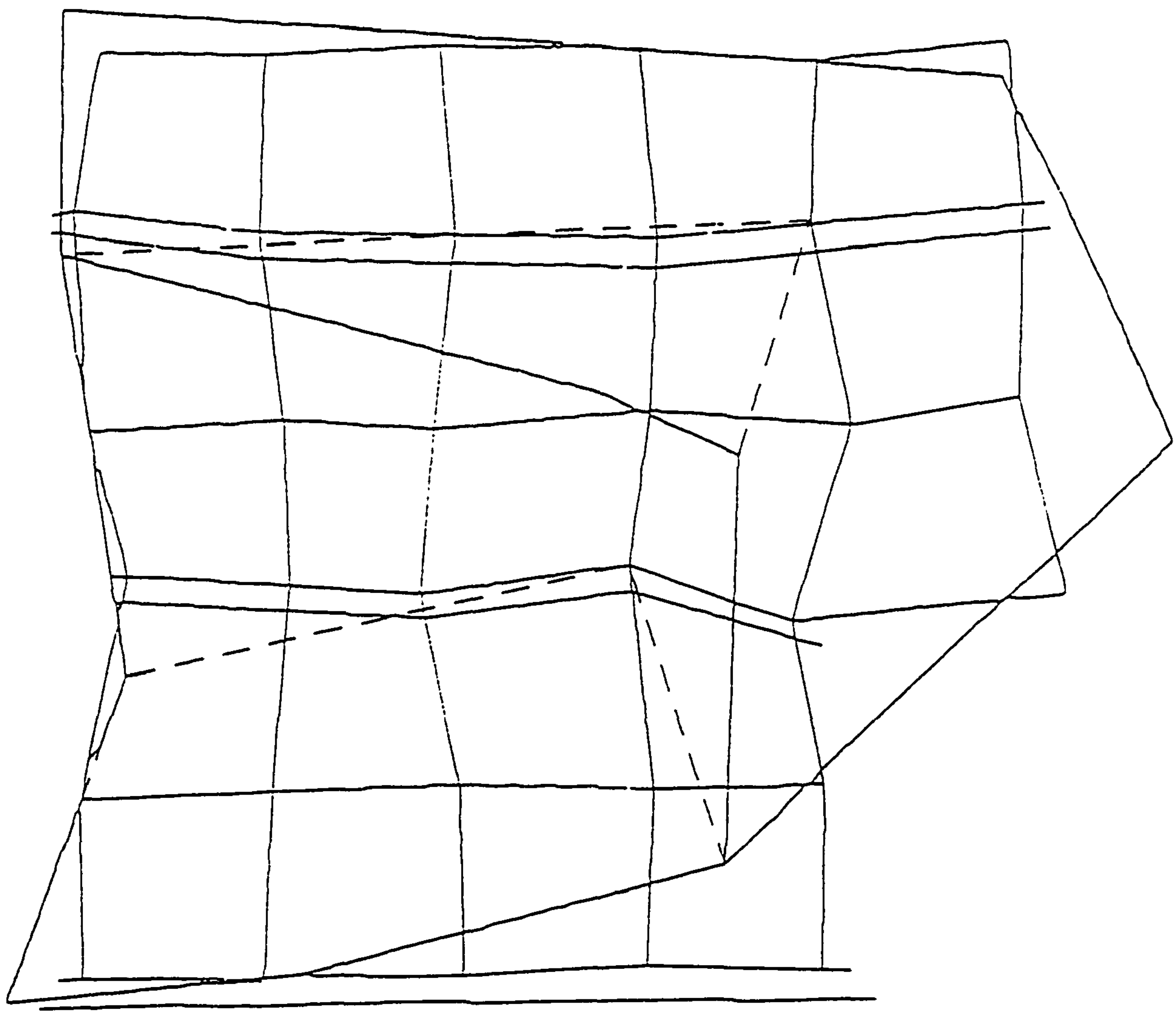
Scale 1 : 2000



Coordinates of property corners of the simulated map

Station Number	X coordinate (m)	Y coordinate (m)
1	1.01	0.97
2	48.23	1.22
3	101.12	0.64
4	150.71	2.46
5	198.37	0.41
6	199.01	50.22
7	152.87	49.64
8	100.43	51.26
9	50.96	50.55
10	0.73	49.21
11	91.32	96.43
12	146.65	103.21
13	262.16	101.32
14	190.32	94.31
15	146.27	110.21
16	90.46	103.26
17	55.72	106.56
18	13.24	109.56
19	3.44	148.26
20	54.44	150.35
21	94.17	147.32
22	152.11	151.77
23	206.14	147.14
24	250.32	154.31
25	154.23	190.19
26	48.22	194.23
27	0.16	208.22
28	48.36	201.43
29	100.41	200.32
30	154.21	198.46
31	195.34	202.21
32	250.94	206.92
33	247.36	251.26
34	196.77	246.31
35	149.44	249.32
36	96.52	251.33
37	50.02	249.47
38	6.73	251.22

Map obtained after scanning and vectorization



Node coordinates obtained by manual digitizing

arc#	node#	stn. #	x-coord	y-coord
1	1	33	246.728	251.717
6	2	38	6.788	250.952
7	3		247.099	250.884
22	4	36	96.821	250.726
3	5		141.556	250.066
20	6	35	149.396	248.870
23	7	37	50.359	248.853
2	8	34	196.995	246.401
1	9		196.979	245.992
4	10		196.946	245.130
7	11		248.548	233.228
15	12	27	-0.478	208.189
16	13		-2.870	207.701
9	14		255.548	207.492
18	15		-4.579	207.352
9	16	32	250.702	206.998
29	17		-5.290	202.269
28	18		-2.949	201.863
23	19	28	48.156	201.855
13	20	31	195.422	201.743
30	21		-0.044	201.360
11	22		256.768	200.182
21	23	29	100.204	200.073
11	24		250.626	199.584
19	25	30	154.095	198.507
39	26		-3.033	195.632
40	27		0.380	194.684
26	28		196.911	194.348
35	29	26	49.236	194.199
34	30		99.331	192.237
32	31	25	153.699	190.137
42	32		50.869	180.647
44	33		96.622	167.927
46	34	24	250.247	154.184
47	35	22	151.690	151.516
59	36		3.133	151.399
48	37		149.524	151.370
55	38	20	54.671	150.725
50	39		151.556	150.449
61	40	19	3.563	148.314
56	41	21	94.363	147.662
52	42	23	206.238	147.247
63	43		3.885	146.007

arc#	node#	stn. #	x-coord	y-coord
65	44		259.261	112.235
67	45	15	146.517	110.447
73	46		8.953	109.646
74	47	18	12.696	109.419
77	48	17	55.598	106.418
81	49	16	90.551	103.444
76	50	12	147.254	102.848
85	51		10.179	102.660
84	52		11.303	102.576
68	53	13	262.181	101.804
68	54		261.517	101.736
70	55		261.563	101.523
71	56		246.549	100.182
79	57		174.810	100.018
91	58		55.003	99.303
96	59		10.493	98.596
93	60	11	91.903	96.540
89	61		174.623	94.680
86	62	14	190.658	94.379
98	63		191.560	89.449
100	64		197.833	87.476
101	65		198.019	54.170
110	66	8	100.747	51.392
112	67	9	50.945	50.702
103	68	6	198.665	50.636
104	69		194.154	50.506
106	70		173.057	49.899
108	71	7	152.435	49.470
114	72	10	0.847	49.397
116	73		0.567	48.609
118	74		172.302	29.825
120	75		151.537	24.673
122	76		100.896	12.107
125	77		150.750	2.916
127	78	4	150.743	2.711
133	79	1	0.927	1.511
124	80	5	198.278	1.165
132	81	2	47.797	1.015
134	82		-4.251	0.906
130	83		55.076	0.738
129	84	3	100.940	0.670
128	85		204.807	0.181
136	86		-9.392	-6.244
136	87		211.668	-7.565

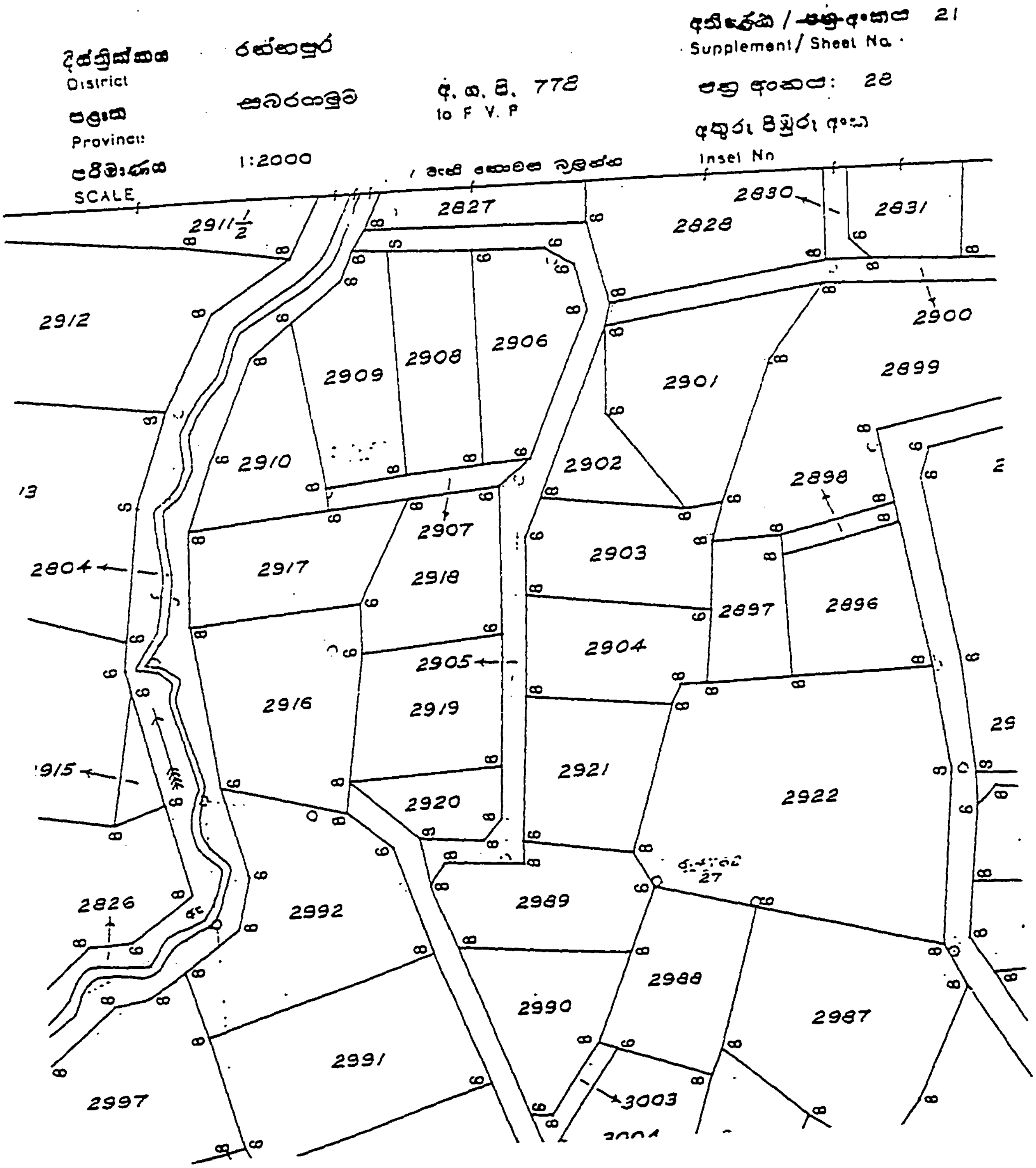
Node coordinates obtained by scanning and vectorization

arc#	node#	stn. #	x-coord	y-coord
136	1		116.600	252.429
1	2		121.492	251.656
1	3		122.673	251.493
139	4		96.426	251.152
2	5		126.385	250.835
3	6		128.830	250.765
6	7		122.676	250.470
4	8		121.664	250.463
5	9		116.605	250.430
141	10	37	50.536	249.579
10	11	35	149.577	249.193
144	12		170.830	247.595
147	13		199.418	246.189
147	14	34	196.889	246.063
150	15		248.185	235.125
71	16		249.459	231.752
11	17	27	-0.310	208.204
69	18		-3.639	207.628
70	19		-5.873	207.315
72	20		256.359	206.797
11	21		10.824	205.740
78	22		11.162	206.572
71	23	32	250.880	206.335
74	24		-3.724	202.380
75	25		-6.365	202.363
12	26		-0.123	201.600
80	27	31	195.581	201.539
78	28	28	48.367	201.465
13	29		100.052	199.981
153	30		257.812	199.895
13	31	29	99.884	199.809
14	32		99.715	199.805
73	33		250.983	199.765
16	34		99.884	199.639
15	35		99.547	199.636
18	36		99.885	199.297
153	37		250.985	199.116
20	38	30	154.021	198.283
155	39		-3.439	195.824
77	40		0.200	195.261
83	41	26	49.044	194.401
85	42		196.806	194.124
87	43		99.062	192.125

88	44		144.430	190.712
23	45		145.948	190.381
22	46	25	153.707	190.090
25	47		50.684	180.697
92	48		96.385	167.970
82	49		1.371	165.168
95	50		1.762	161.033
158	51	24	250.256	154.547
24	52		151.959	152.024
161	53		148.546	151.706
25	54	22	151.623	151.339
26	55	20	54.153	150.536
27	56	19	151.794	150.467
164	57		3.236	148.499
28	58	21	93.960	147.552
97	59	23	205.767	147.253
105	60		4.860	138.698
99	61		259.564	112.427
102	62	15	146.343	110.178
167	63		8.179	109.510
105	64	18	12.363	109.309
29	65		55.458	106.517
106	66	17	55.121	106.344
30	67	16	90.205	103.841
31	68		89.532	103.154
35	69		146.700	103.013
103	70		90.544	102.990
35	71	12	147.292	102.590
113	72		91.052	102.482
33	73		11.286	102.476
34	74		9.685	102.210
33	75		10.781	102.132
36	76		10.111	100.591
. 38	77		247.890	100.427
108	78		174.703	99.807
111	79		54.840	99.332
38	80		245.280	99.130
170	81		10.291	97.532
113	82	11	91.490	96.512
110	83		174.556	95.080
119	84	14	190.528	94.466
118	85		177.080	93.995
123	86		177.418	93.827
122	87		191.132	89.521
124	88		198.009	87.646
125	89		2.131	59.840
125	90		2.131	59.754
127	91		198.062	53.815
39	92		99.964	51.515
42	93		100.302	51.347
40	94	8	100.133	51.345
39	95		99.796	51.343
44	96		195.162	50.682

131	97	9	50.406	50.426
44	98	6	198.620	50.278
45	99		193.140	49.986
48	100		172.820	49.940
47	101		146.175	49.938
43	102		144.826	49.929
46	103	7	152.416	49.637
49	104	10	0.389	49.589
49	105		-0.362	46.769
67	106		-0.698	45.596
56	107		-0.361	46.427
173	. 108		171.746	29.141
50	109		151.475	24.374
51	110		151.224	23.776
52	111		100.751	11.759
54	112	4	150.356	2.353
55	113		60.306	1.769
57	114		59.548	1.336
56	115	1	0.273	1.209
177	116		20.510	0.999
176	117		-6.135	0.997
61	118		20.847	0.831
58	119		60.815	0.749
63	120		198.086	0.615
178	121		47.409	0.580
60	122		32.569	0.460
60	123		30.798	0.383
62	124		100.783	0.325
186	125		205.338	0.321
180	126		47.158	-0.364
65	127		32.487	-0.459
64	128		30.801	-0.470
68	129		-11.005	-6.372
59	130		212.105	-7.314

APPENDIX H PART OF A SRI LANKAN CADASTRAL MAP



APPENDIX I GEO-REFERENCING ERRORS DUE TO MOUNTAIN PEAKS IN 1 : 500,000 MAP

Transforming coordinates for coverage boun-orig

Scale (X, Y) = (12806.772, 12861.982) Translation = (-179101.258, -136746.029)
Rotation (degrees) = (0.358) RMS Error (input, output) = (0.078, 997.219)

tic id	input x output x	input y output y	x error	y error
98	9.854	4.808		
	-52448.699	-73922.000	-841.803	-268.586
83	13.004	5.243		
	-13241.900	-68000.898	257.130	-365.927
85	18.809	5.262		
	62202.199	-67850.500	-846.626	149.857
79	17.143	7.849		
	39758.102	-33962.500	54.802	-585.785
81	14.053	8.720		
	-801.400	-22301.600	972.468	-1268.603
100	9.632	7.688		
	-56327.898	-37224.898	-36.097	60.099
101	11.760	9.065		
	-30323.301	-21159.600	1101.406	1860.085
65	14.071	10.549		
	0.000	0.000	255.233	-44.712
76	16.920	10.027		
	37671.102	-7482.400	-888.320	930.866
70	20.459	14.337		
	81020.102	43899.898	740.090	239.971
64	17.859	14.079		
	48016.898	45988.500	466.975	-355.885
61	10.683	18.573		
	-43267.000	103276.500	-444.277	-364.134
62	17.230	17.929		
	40911.500	95092.000	-790.983	12.755

Arc:

APPENDIX J SUBROUTINES USED FOR "*Con_cord*" AND THE SEQUENTIAL LEAST SQUARES ADJUSTMENT

```
      Subroutine TOXYZ(N)

c Calculation of Global X, Y, Z coordinates
c Given geodetic latitude and longotude and height of a
c stations
c Lat, Lon are in "input.d" file (stn, LatD, LatM, LatS,
c LonD, LonM, LonS)
c Outpur X,Y,Z coordinates are in "output.d"file
c ( stn, X, Y, Z)
c  N is the number of stations.

        Implicit real*8(a-h,p-z)
        Dimension istn(N), latd(N),latm(N),alats(N),
     +      lond(N),lonm(N),alons(N),alat(N),alon(N),
     +      deno(N),an(N),x(N),y(N),z(N)

         open (unit=1, file='input_f.d')
         open (unit=2, file='output_f.d')

         pi = 3.141592653589793 2d0
c  Coordinates are calculated as on the ellipsoid. So,
c  height is taken as 0.0
c  otherwise heights of stations have to be read and
c   assigned to h.
          h = 0.0d0

         write(*,2)n
 2       format(//,12x,'Number of Stations are',i3)

         do 100 i=1,n

         read(1,*)istn(i),latd(i),latm(i),alats(i),
     +         lond(i),lonm(i),alons(i)

      alat(i) = (latd(i)+latm(i)/60.0d0+alats(i)/3600.0d0)
     +          *pi/180.0d0
      alon(i) = (lond(i)+lonm(i)/60.0d0+alons(i)/3600.0d0)
     +          *pi/180.0d0

         smajor = 20922931.80d0 * 0.3047995d0
         sminor = 20853374.58d0 * 0.3047995d0

         esq = (smajor**2 - sminor**2)/smajor**2
```

```
c calculating the Radius of curvature in the prime vertical
         deno(i) = (1.0d0-esq*dsin(alat(i))**2)**0.5
         an(i) = smajor/deno(i)

          x(i)  =  (an(i)+h)*dcos(alat(i))*dcos(alon(i))
          y(i)  =  (an(i)+h)*dcos(alat(i))*dsin(alon(i))
          z(i)  =  (an(i)*(1.0d0-esq)+h)*dsin(alat(i))


c    writing data

 15    format(/,i3,i5,i5,f8.5,i3,i5,f8.5)
      write(*,15)istn(i),latd(i),latm(i),alats(i),lond(i),
     +             lonm(i),alons(i)

c     Writing the calculated values

          write(2,16)istn(i),x(i),y(i),z(i)
          write(*,16)istn(i),x(i),y(i),z(i)
 16        format(i3,3f20.8)
 100      continue

           return
           end
```

```
              SUBROUTINE PARAM(N,M)
c Program for the calculation of Transformation parameters
c to transform old global XYZ coordinates  to new XYZ
c global  coordinates
c 12 parameter linear affine transformation
c at least 4  points are required for the transformation
c N is the number of common points and M=3*N
c Old XYZ coordinates are in "from.d" file and
c New XYZ coordinates are in "to.d" file.
c Parameters will be written to "para.d" file


       implicit real *8(a-h,o-z)
       dimension istnf(N),xf(N),yf(N),zf(N),istnt(N),
     +       xt(N),yt(N),zt(N),a(M,12),al(M,1),at(12,M),
     +       ata(12,12),atl(12,1),b(12),x(12,1)

           OPEN (UNIT=1, FILE = 'from.d')
           OPEN (UNIT=2, FILE = 'to.d')
           OPEN (UNIT=3, FILE = 'para.d')


              DO 10 i = 1,n
             read(1,*)istnf(i),xf(i),yf(i),zf(i)
             read(2,*)istnt(i),xt(i),yt(i),zt(i)

             xf(i)=xf(i)-1000000.0d0
             yf(i)=yf(i)-6200000.0d0
             zf(i)=zf(i)-800000.0d0
             xt(i)=xt(i)-1000000.0d0
             yt(i)=yt(i)-6200000.0d0
             zt(i)=zt(i)-800000.0d0
 10         CONTINUE

c   Writing old and new XYX values on the screen
            do 111 i=1,10
             write(*,*)istnf(i),xf(i),yf(i),zf(i)
 111         write(*,*)istnt(i),xt(i),yt(i),zt(i)

c  forming A matrix for 12 parameters

           j=1
           do 200 i=1,m
           a(i,1)=xf(j)
           a(i,2)=yf(j)
           a(i,3)=zf(j)
           a(i,10)=1.0d0

           a(i+1,4)=xf(j)
           a(i+1,5)=yf(j)
           a(i+1,6)=zf(j)
           a(i+1,11)=1.0d0
```

```
            a(i+2,7)=xf(j)
            a(i+2,8)=yf(j)
            a(i+2,9)=zf(j)
            a(i+2,12)=1.0d0

            j=j+1
            i=i+2
  200       continue


 c   forming L matrix for 12 parameter transformation
             j=1


          do 300 i=1,m
          al(j,1)=xt(i)
          al(j+1,1)=yt(i)
          al(j+2,1)=zt(i)
             j=j+3
 300      continue


c Subroutine TRANS is to transform  a matrix
           CALL TRANS(A,AT,m,12)

c Subroutine AB is to multiply A and B matrices
           CALL AB(AT,A,ATA,12,m,12)

           CALL AB(AT,AL,ATL,12,m,1)

c Subroutine INVERT calculate the inversion of a matrix
c and replaces the original matrix by the inversion.
           CALL INVERT(ATA,B,12)

c   After inversioin ATA is replaced by ATAinverse
            CALL AB(ATA,ATL,X,12,12,1)

C Writing calculated parameters.
          print*,'   Parameters'
          do 17 i=1,12
          write(3,*)x(i,1)
          write(*,*)x(i,1)

 17       CONTINUE

             CLOSE(UNIT=1)
             CLOSE(UNIT=2)
             CLOSE(UNIT=3)

         RETURN
         END
```

```
            SUBROUTINE convert(N,M)

c Subroutine to  transform  global XYZ XYZ coordinates
c from one system to another system, using available
c parameters at least 4  common points are required for
c the transformation

c N is number of points and M=3*N
           IMPLICIT REAL *8(A-H,O-Z)

           dimension istn(N),xf(N),yf(N),zf(N),
     +       a(M,12),al(M),at(12,M),istnt(N),xt(N),
     +       ata(12,12),atl(12,1),B(12),x(12),yt(N),zt(N),
     +       xn(N),yn(N),zn(N),xd(N),yd(N),zd(N)

           OPEN (UNIT=1, FILE = 'from.d')
           OPEN (UNIT=3, FILE = 'para.d')
           open (unit=4, file='to.d')

              DO 10 i = 1,n
             read(1,*)istn(i),xf(i),yf(i),zf(i)

             xf(i)=xf(i)-1000000.0d0
             yf(i)=yf(i)-6200000.0d0
             zf(i)=zf(i)-800000.0d0
             xt(i)=xt(i)-1000000.0d0
             yt(i)=yt(i)-6200000.0d0
             zt(i)=zt(i)-800000.0d0

 10         CONTINUE

             do 20 i=1,12
             read(3,*)x(i)
 20          continue
c  forming A matrix for 12 parameters

           j=1
           do 200 i=1,m
           a(i,1)=xf(j)
           a(i,2)=yf(j)
           a(i,3)=zf(j)
           a(i,10)=1.0d0

           a(i+1,4)=xf(j)
           a(i+1,5)=yf(j)
           a(i+1,6)=zf(j)
           a(i+1,11)=1.0d0

           a(i+2,7)=xf(j)
           a(i+2,8)=yf(j)
           a(i+2,9)=zf(j)
           a(i+2,12)=1.0d0

           j=j+1
           i=i+2
 200       continue
```

```
c Subroutine AB is for matrix multiplication
            CALL AB(A,x,AL,m,12,1)
             j=1
            do 300 i=1,m
            xn(j)=al(i)
            yn(j)=al(i+1)
            zn(j)=al(i+2)
            i=i+2

           j=j+1
300        continue

           do 400 i=1,n
           xd(i)=xt(i)-xn(i)
           yd(i)=yt(i)-yn(i)
           zd(i)=zt(i)-zn(i)

            xn(i)=xn(i)+1000000.0d0
            yn(i)=yn(i)+6200000.0d0
            zn(i)=zn(i)+800000.0d0

400        continue

C Writing calculated new coordinates
          j=1
          do 17 i=1,n
      write(2,*)istn(i),xn(i),yn(i),zn(i),xd(i),yd(i),zd(i)
      write(*,*)istn(i),xn(i),yn(i),zn(i),xd(i),yd(i),zd(i)
16        format(i5,3f15.4,3f8.3)
17        continue

            CLOSE(UNIT=1)
            CLOSE(UNIT=2)
            CLOSE(UNIT=3)

        return
        end
```

```
      SUBROUTINE FROMXYZ(N)

c Calculation of Latitudes and Longitudes from Global X, Y,
c Z coordinates
c Input data is in "input_r.d" file (format : station, X, Y, Z)
c Output values are in "output_r.d" file
c  N is the number of stations

        implicit real*8(a-h,p-z)

      dimension istn(N),alat(N),alon(N),x(N),y(N),z(N),
     +     an(N),deno(N),latd(N),latm(N),alats(N),lond(N),
     +     lonm(N),alons(N)

        open(unit=1, file='input_r.d')
        open(unit=2, file='output_r.d')

        pi = 3.14159265358979 32d0
         h = 0.0d0

        do 100 i=1,n
        read(1,*)istn(i),x(i),y(i),z(i)

        smajor = 20922931.80d0 * 0.3047995d0
        sminor = 20853374.58d0 * 0.3047995d0

           esq = (smajor**2 - sminor**2)/smajor**2

c calculating longitude
       alon(i) = datan(y(i)/x(i))

c calculating an approximate value for latitude
       alat(i) = atan(z(i)/(sqrt(x(i)**2+y(i)**2)))

        do 52 j=1,10

c calculating the Radius of curvature in the prime vertical
       deno(j) = (1.0d0-esq*dsin(alat(i))**2)**0.5
         an(j) = smajor/deno(j)

c calculating latitude
       alat(i) = datan(z(i)/(sqrt(x(i)**2+y(i)**2))*
     +              (an(j)+h)/(an(j)*(1.0d0-esq)+h))

 52     continue

c Subroutine "RATODMS" converts radians to Degrees, minutes
c and seconds.
         call ratodms(alat(i),latd(i),latm(i),alats(i))
         call ratodms(alon(i),lond(i),lonm(i),alons(i))
```

```
      WRITE (2,11)ISTN(I), LATD(I), LATM(I), ALATS(I),
     +          LOND(I), LONM(I), ALONS(I)

      WRITE (*,11)ISTN(I), LATD(I), LATM(I), ALATS(I),
     +          LOND(I), LONM(I), ALONS(I)

11         FORMAT(I4,I5, I5,F20.8,5X,I5,I5,F20.8)

100        continue

        return
        end
```

```
      SUBROUTINE RATODMS(RAD, LDD, MM, SS)
c     Subroutine to convert radians to dd mm ss

      IMPLICIT REAL *8(A-H,P-Z)
      PI = 4.0D0*DATAN(1.0D0)
      DEG = RAD / PI * 180.0D0

       LDD = IDINT(DEG)
       AMM = (DEG - LDD)*60.0D0
       MM = IDINT(AMM)
       SS = (AMM-MM)*60.0D0

       RETURN
       END
```

```
      SUBROUTINE MERCA(ALAT, ALON, X, Y)
C  Subroutine to calculate plane coordinates from geodetic
C   coordinates for Sri lanka
C  Transverse mercator projection
C  Central meridian and standard parallel through
C  PIDURUTALAGALA.
C  Equations Bomford, Geodesy, 4th edition, 2.114 and
C  and A.68 (Latitudes and Longitudes must be in radians)

          IMPLICIT REAL*8(A-H,P-Z)

           SMAJOR = 20922931.80D0 * 0.3047995D0
           SMINOR = 20853374.58D0 * 0.3047995D0


c Calculating e-squared (E)
          E = (SMAJOR**2 - SMINOR**2) / SMAJOR**2

c Calculating radius of curvature in prime vertical
      AN = SMAJOR / (1.0D0 -E*DSIN(ALAT)**2)**0.5
      AM = SMAJOR*(1.0D0-E)/((1.0D0-E*DSIN(ALAT)**2)**1.5)

c Calculating coefficents for Bomford A.68
      A0 = 1.0D0-1.0D0/4.0D0*E-3.0D0/64.0D0*E**2+5.0D0/
     +     256.0D0*E**3
      A2 = 3.0D0/8.0D0*(E+1.0D0/4.0D0*E**2+15.0D0/
     +     128.0D0*E**3)
      A4 = 15.0D0/256.0D0* (E**2 + 3.0D0/4.0D0*E**3)
      A6 = 35.0D0/3072.0D0*E**3

c Calculating the meridian distance for a station
      S = SMAJOR*(A0*ALAT-A2*DSIN(2.0D0*ALAT)+A4*
     +    DSIN(4.0D0*ALAT)-A6*DSIN(6.0D0*ALAT))

      PI = 3.14159265358979320D0

      CLAT = (7.0D0+ 00/60. + 1.729D0/3600.0D0)*PI/180.0D0
      CLON = (80.0D0+46.0D0/60.0D0+18.160D0/3600.0D0)*
     +       PI/180.0D0

c Calculating the meridian distance to Pidurutalagala
      SC = SMAJOR* (A0*CLAT -A2*DSIN(2.0D0*CLAT)+A4*
     +     DSIN(4.0D0*CLAT)-A6*DSIN(6.0D0*CLAT))

c Coefficents in Bomford 2.114

      EPS = E/(1-E)
        G = (DCOS(ALAT)**2)*(1.0D0-DTAN(ALAT)**2 +
     +       EPS*DCOS(ALAT)**2)
        H = DSIN(ALAT)*(DCOS(ALAT)**2)*(5.0D0-
     +      DTAN(ALAT)**2 +9.0D0*EPS*DCOS(ALAT)**2 +
     +      4.0D0*EPS**2*DCOS(ALAT)**4)
       AJ =(DCOS(ALAT)**4)*(5.0D0-18.0D0*(DTAN(ALAT)**2)+
     +     (DTAN(ALAT)**4)+14.0D0*EPS*(DCOS(ALAT)**2) +
     +   58.0D0*EPS*(DSIN(ALAT)**2))
```

```
c Scale factor from the Jackson's report
            Z = 0.9999238418D0

c Calculating  northings (from the equator first)

      Y = S + 0.5D0*((ALON-CLON)**2)*AN*(DCOS(ALAT))*
     +    (DSIN(ALAT)) + H/24.0D0*((ALON - CLON)**4)*
     +    AN*DCOS(ALAT)

      YC = SC
       Y = (Y - YC)*Z

c Calculating eastings

      X = Z*AN*DCOS(ALAT)*((ALON-CLON) + (G*(ALON - CLON)
     +    **3)/6.0D0 + (AJ*(ALON - CLON)**5)/120.0D0)

        RETURN
        END




            SUBROUTINE INVERT(A,B,I)

C Subroutine to invert the matrix A and replace A by its inverse
C Subroutine provided by the  Ohio State University.

          IMPLICIT REAL*8 (A-H,O-Z)
          DIMENSION A(I,I), B(I)
          IF ( I .EQ. 1 ) GO TO 10
          IM = I-1
             DO 5 K = 1,I
             DO 2 J = 1,IM
2         B(J) = A(1,J+1)/A(1,1)
          B(I) = 1./A(1,1)
            DO 4 L = 1,IM
            DO 3 J = 1,IM
3         A(L,J) = A(L+1,J+1) - A(L+1,1) * B(J)
4         A(L,I) = -A(L+1,1) * B(i)
             DO 5 J =1,I
5         A(i,j) = B(J)
           RETURN
10        A(1,1) =1./A(1,1)
           RETURN
           END
```

S Subroutine used for sequential least squares adjustment

```
      SUBROUTINE SEQUENTIAL(N,M)
c Subroutine to upgrade digitized  and georeferenced
c coordinates of property corners using linear, angular
c and area measurements.

c  N is the number of property corners, and M=2*N
c  Coordinates of property corners are in "node.d" file
c  Upgraded values will be written to "new.d" file

        implicit real*8 (a-h,o-z)

       dimension inode(N), x(N), y(N), a1(M,M), a11(M,1),
     +  a2(5,M),a12(5,1),a1t(M,M),a2t(M,5),a1ta1(M,M),
     +  a2ta2(M,M),ata(M,M),a1t11(M,1), a2t12(M,1),
     +  at1(M,1),xhat(M,1),b(M)

       open(unit=1,file='node.d')
       open(unit=2,file='new.d')

       print*,' Number of nodes are:'
       write(*,*)N

c reading # of available liner, angular and area measurements

      print*,'No of distance conditions ?'
      read(*,*)nd
      print*,'No of angle conditions ?'
      read(*,*)na
      print*,'No of area conditions ?'
      read(*,*)nr

c  reading nodes and their X,Y
       do 10 i=1,n
       read(1,*)inode(i), x(i), y(i)
 10    continue

      nn2=2*n
      nt=nd+na+nr
c A  and Lmatrices for first set of observations (I)
c First set of observations are the digitized coordinates.
       do 20 i=1,nn2
       a1(i,i)=1.0
 20    continue
```

```
c  Formation of L1 matrix
        do 30 i=1,nn2,2
        all(i,1)=0.0
        all(i+1,1)=0.0
 30     continue

c A and L matrices for linear, angular and area measurements
c first for distances (Uotila page 67)
        if (nd .eq. 0) go to 41
        do 40 i=1,nd
         print*,'Enter both nodes for distance condition ?'
         read(*,*)k1,k2
         L1=2*k1-1
         L2=2*k2-1
        print*,'enter the distance (same units as coordinates) ?'
        read(*,*)d
c  Calculating approximate distances

        ddo = dsqrt((x(k1)-x(k2))**2+(y(k1)-y(k2))**2)


       a2(i,L1)=(x(k1)-x(k2))/d
       a2(i,L1+1)=(y(k1)-y(k2))/d
       a2(i,L2)=(x(k2)-x(k1))/d
       a2(i,L2+1)=(y(k2)-y(k1))/d
       al2(i,1)=ddo-d

 40    continue
 41    continue

c  Formation of A2 and L2 matrices for angles
c  Uotila page 68
            ro=180*60*60*7/22.0
            j=nd+1
       if (na .eq. 0) go to 51
       do 50 i=1,na
       print*,'Select end-middle-end nodes for angles'
             read(*,*)k1,k2,k3
              L1=2*k1-1
              L2=2*k2-1
              L3=2*k3-1
       print*,'Enter the angle in degrees'
       read(*,*)ang

c Calculating distances among 3 points
      dis1 = dsqrt((x(k1)-x(k2))**2+(y(k1)-y(k2))**2)
      dis2 = dsqrt((x(k3)-x(k2))**2+(y(k3)-y(k2))**2)

      az1= datan((x(k3)-x(k2)) /(y(k3)-y(k2)))
      az2= datan((x(k1)-x(k2)) /(y(k1)-y(k2)))
      ango = abs(az1-az2)*180*7/22.0

      a2(j,L1)    = -ro*(y(k1)-y(k2))/dis1**2
      a2(j,L1+1)  = -ro*(x(k1)-x(k2))/dis1**2
      a2(j,L2)    = -ro*(y(k2)-y(k3))/dis2**2 +
```

```
     +                 (y(k1)-y(k2))/dis1**2
      a2(j,L2+1)  = -ro*(x(k3)-x(k2))/dis2**2 +
     +                 x(k1)-x(k2))/dis1**2
      a2(j,L3)    = -ro*(y(k3)-y(k2))/dis2**2
      a2(j,L3+1)  = -ro*(x(k3)-x(k2))/dis2**2

      a12(j,1 )= (ango-ang)*3600.0

          j=j+1
 50       continue
 51       continue

c Formation of A2 and L2 for area conditions
           if (nr .eq. 0) go to 61
              j=nd+na+1
           do 60 i=1,nr
           print*,'Select 4 points for area condition'
           print*,'In clockwise or anti-clockwise order'
           read(*,*)k1,k2,k3,k4
               L1=2*k1-1
               L2=2*k2-1
               L3=2*k3-1
               L4=2*k4-1
           print*,'enter the area in square meters'
           read(*,*)area
c Calculating approximate value for the area
       areao=abs((x(k1)*y(k2)+x(k2)*y(k3)+x(k3)*y(k4)+
     +        x(k4)*y(k1)-x(k1)*y(k4)-x(k4)*y(k3)-x(k3)*
     +        y(k2)-x(k2)*y(k1))/2.0)

           a2(j,L1)= y(k2)-y(k4)
           a2(j,L1+1)=x(k4)-x(k2)

           a2(j,L2)=y(k3)-y(k1)
           a2(j,L2+1)=x(k1)-x(k3)
           a2(j,L3)=y(k4)-y(k2)
           a2(j,L3+1)=x(k2)-x(k4)
           a2(j,L4)=y(k1)-y(k3)
           a2(j,L4+1)=x(k3)-x(k1)
           a12(j,1)=areao-area

           j=j+1
 60        continue
 61        continue

c Subroutine "TRANS" transforms  a matrix
c Subroutine "AB" is to multiply two matrices
c Subroutine  "ADD" is to add two matrices
c Subroutine "INVERT" is to invert a matrix and replace
c thc the original martix by the inverse.

           call trans(a1,a1t,nn2,nn2)
           call trans(a2,a2t,nt,nn2)
           call ab(a1t,a1,a1ta1,nn2,nn2,nn2)
           call ab(a2t,a2,a2ta2,nn2,nt,nn2)
           call add(a1ta1,a2ta2,ata,nn2,nn2)
```

```
            call ab(a1t,a11,a1t11,nn2,nn2,1)
            call ab(a2t,a12,a2t12,nn2,nt,1)
            call add(a1t11,a2t12,at1,nn2,1)
c ata is replaced by ata-inverse
            call invert(ata,b,nn2)
            call ab(ata, at1,xhat,nn2,nn2,1)

            do 70 i=1,nn2
            xhat(i,1)=-1*xhat(i,1)
 70         continue

            do 80 i=1,nn2,2

c Writing dx and dy on the screen
            write(*,*)istn(i),xhat(i,1),xhat(i+1,1)
 80         continue

            do 90 i=1,nn
            x(i)=x(i)+ xhat(i,1)
            y(i)=y(i)+xhat(i+1,1)

c   Writing  updated property corners.

            write(13,*)istn(i),x(i),y(i)
            write(2,*)istn(i),x(i),y(i)
 90         continue


            return
            end
```

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