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THE ADJUSTMENT AND ANALYSIS OF DAM MONITORING NETWORKS

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Abstract

Monitoring of dams is one of the classic topics for the application of geodesy to engineering. This research presents the results of the adjustment and analysis of a dam monitoring network that has been observed for the last ten years. The least squares adjustment technique has been used to carry out the 3-dimensional network adjustment.

A subroutine called DETECT has been developed for the comparison of results from different epochs to compute displacement occurred in the monitoring process. Morever, this subroutine is able to present the displacement in graphical form for a better interpretation.

On the data acquisition technique, we have proposed that an Automatic Theodolite Measuring System (ATMS) for the monitoring the dam structure and GPS surveying for backward control be implemented in the monitoring of a dam structure. It should be possible to transfer this concept with ease to different monitoring tasks as well.

1.0 INTRODUCTION

Due to changeable water level, seasonal temperature changes, gravitational body forces, and in some cases due to seismic activity, all dam structures, whether concrete or earthfilled, undergo deformations. Monitoring surveys serve a dual purpose (Chrzanowski, et al., 1990): (i) to check whether the behaviour of the dam and its environment follows the predicted pattern so that any unpredicted deformations could be detected at an early stage; and (ii) in the case of any abnormal behaviour, to give an account, as accurately as possible, of the actual deformation status which could be used for the determination of the causative effects which trigger the deformation. Failure of a dam, even when loss of life is not involved, is an expensive, if not a tragic event.

Basically, two types of measuring techniques and instrumentation are being used in monitoring the geometrical deformation parameters in dam structures: (i) **geodetic surveys** (terrestrial, photogrammetric, and Global Positioning System (GPS) techniques); and (ii) **geotechnical and structural measurements** of local deformations using tiltmeters, strainmeters, extensometers, plumb-lines, etc. (Chrzanowski, et al, ibid.). Geodetic surveys have traditionally been used mainly for determining the absolute displacements of selected object points with respect to some reference points which are assumed to be stable. On the other hand, geotechnical measurements have traditionally been used mainly for relative measurements within the deformable object and its surroundings.

This research will focus on the geodetic surveys technique using high precision electro-optical distance measuring instruments. A test site has been identified in the Ulu Terengganu area. The aim of this paper is to present the results of the adjustment and analysis of a dam monitoring network that have been observed spanning a period of over ten years.

2.0 THE ADJUSTMENT MODEL

Planning, setting out and geometric control of complex structures (bridges, tunnels, roofs, tall buildings, dams, etc) are becoming more frequent assignments for land surveyors. In order that these structures disturb the environment as little as possible the demands upon engineering design increase, resulting in complicated plans, and monitoring after the structure is built.

In order to obtain the greatest flexibility in the planning and setting out of the monitoring networks, sophisticated methods are necessary in which objective criteria relating to the quality of networks can be determined. A key role is played by an automated data flow and by global strategies to evaluate the data, i.e., **the adjustment and analysis of monitoring networks.**

Geodetic networks are analysed in order to determine the parameters of the networks: the coordinates of the points and additional parameters serving for fixing the datum of the geodetic control. In any geodetic network, the points to be determined are connected by observational elements and all measurements are to be used taking into account their individual statistical accuracy. In addition to the determination of the coordinates, accuracy information for the coordinates must be derived from the accuracy of the measurement elements.

2.1 Least Squares Adjustment Model

The observed data are, in general, more in number than the minimum needed for a unique evaluation of the quantities required. There are two main reasons for making more measurements than the minimum necessary. Firstly, the access or redundant measurements can provide a check on gross errors (blunders) in measurements and secondly they can give more precise evaluation of the quantities required than would the minimum number of measurements.

The method of least squares provides a widely applicable procedure for dealing with redundant data. The use of least squares technique gives results which are the "most probable values" of the estimates. The non-linear mathematical model relating the observations and the unknown parameters can be expressed as follows (Uotila,1986):

$$L_a = F(X_a) \qquad (1)$$

or

$$\hat{L}_a = F(\hat{X}_a) \qquad (2)$$

Where;

L_a = Theoretical values of observed quantities

$\hat{L}_a$ = Estimates or adjusted values of observed quantities

X_a = Theoretical values of parameters

$\hat{X}_a$ = Estimates or adjusted values of parameters

The solution for the parameters $\hat{X}_a$:

$$\hat{X}_a = -(A^tPA)^{-1}A^tPL \qquad (3)$$

where

$$P = \sigma_0^2 \, \Sigma_{L_b}^{-1} \quad \text{(weight matrix)}$$

σ_0^2 = a priori variance of unit weight

$L = L_o - L_b$

L_o is the approximate observation, $L_o=F(X_o)$, and L_b is the actual observation. The estimates of the accuracy of the parameter can be computed as follows:

$$\Sigma X_a = \sigma_o^2 \, (A^T P A)^{-1} \qquad \mathbf{(4)}$$

For further details on least squares technique please refer to Uotila, 1986.

2.2 Functional Model of 3-Dimensional Terrestrial Geodetic Observations

In geodetic control surveys the parameters to be estimated by least squares are usually the spatial rectangular coordinates of points. These parameters can be defined as the Northing (N) component, the Easting (E) component and the Vertical (Z) component.

Normal practice is to make the observations and correct them for physical effects such as atmospheric refraction, calibration factors, and other instrumental errors. The functional model, La=F(Xa), which expresses the relationship between the measured element and the parameters to be estimated can then be constructed. The functional models associated with a typical 3D terrestrial geodetic network are given as follows - see Figure 1.

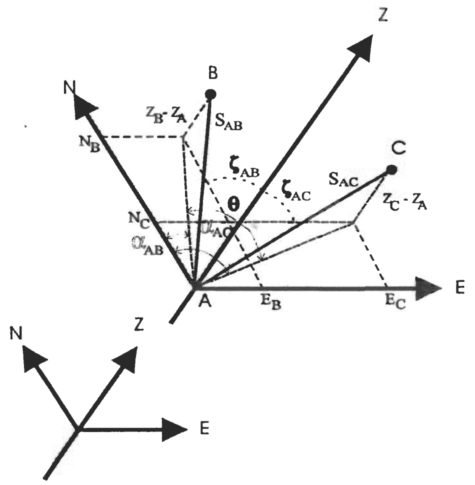

Figure 1 : Three Dimensional Representation of Conventional Terrestrial Measurements: Slope Distance (s), Horizontal Angle(θ), Zenith Angle (ζ).

The functional model for the **slope distance s** between points $A(N_A,E_A,Z_A)$ and $B(N_B,E_B,Z_B)$ is:

$$s \equiv [(E_B - E_A)^2 + (N_B - N_A)^2 + (Z_B - Z_A)^2]^{1/2} \qquad \mathbf{(5)}$$

The functional model for the **horizontal angle** θ is:

$$\theta \equiv \arctan [(E_C - E_A)/ (N_C - N_A)] - \arctan [(E_B - E_A)/ (N_B - N_A)] \qquad \mathbf{(6)}$$

The functional model for the **vertical angle** ζ is:

$$\zeta \equiv \arccos \{ (Z_B - Z_A) / [(E_B - E_A)^2 + (N_B - N_A)^2 + (Z_B - Z_A)^2]^{1/2} \} \qquad \mathbf{(7)}$$

2.4 The Determination of the Initial Coordinates

The calculation of initial values is the most essential problem of the network adjustment. Without suitable initial values the parameters of the network cannot be determined. The theoretical background for an automated determination of initial coordinates can be found in Grundig (1991).

2.5 Blunders Detection

Blunders or gross errors in the observations or fixed positions will falsify the adjustment results and cause large residuals. Large residuals will generally result for many observations. It seems natural to first remove observations dependent on the size of the residuals in order to arrive at a consistent set of observables first. For example, it is known that misidentifications of stations or targets and misreadings are a main source for blunders.

Automated blunders detection and removal technique has been incorporated into most of the least squares adjustment softwares. This module serves not only for the blunders detection but also for the calculation of initial coordinates.

3.0 THE LEAST SQUARES ADJUSTMENT SOFTWARE

Recently, many least squares adjustment programs have been improved to allow them to be more user friendly software and capable of dealing with a large and heterogeneous data set. Furthermore, many of these softwares allow automated processing such as:

- data preprocessing
- weighting of observations
- network design and preanalysis
- calculation of initial coordinate values
- detection of blunders due to the misidentification of points and data input
- statistical analysis
- computations of error ellipses
- graphical display of network and network quality

One of the many adjustment program is Star-Net, which is specially designed to perform 2-dimensional (2D) and 3-dimensional (3D) survey network using personal computers. Star-Net fulfill the criteria of user friendliness in which many of the typical tasks mentioned above are menu driven. For a 3D network, requirement for input data may consist of the following data types: horizontal angle, azimuth, slope distance, zenith angle, height differences, instrument height and target height.

4.0 CASE STUDY

4.1 Test Site

A dam site in Terengganu has been chosen for the purpose of the deformation monitoring study. The dam is an earth-filled dam. Figure 2 shows the plan view of the dam which consists of 4 reference pillars and 31 monitoring (object) points. Most of the monitoring points are located where maximum deformations have been predicted, particularly at the interface between the dam structure and surrounding material, and on the dam slopes.

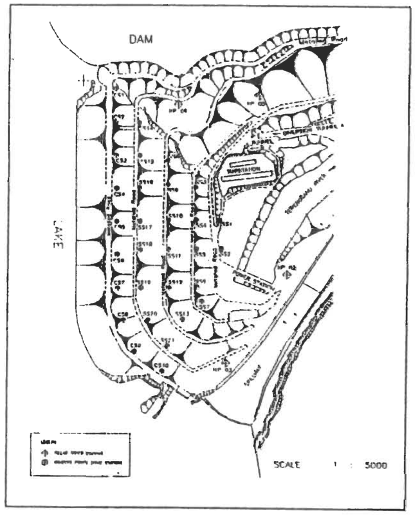

Figure 2: Plan View of the Study Area (Dam Site)

4.2 Field Observations

Two observation epochs are available for the study, one is observed in October 1984 and the other in December 1994. The observations consist of (i) reference control stations, and (ii) the observations of object points.

4.2.1 Observations of the Reference Control Stations

One of the problems which are frequently encountered in practice is the instability of the reference geodetic points in the process of determining the absolute displacements of the object points. Any unstable reference points must be identified. Otherwise, the calculated displacement of the object points and the subsequent analysis and interpretation of the deformation of the object may be significantly distorted.

In the present study, geodetic control survey has been carried out to identify the stability of the reference stations KP02, KP03, KP04 and KP05 (see Figure 2). The results obtained from the October 1984 and December 1994 surveys show that there are no significant differences in the coordinates of the reference pillars between the two epochs (see the following Tables 1-3).

	Northing (m)	Easting (m)	Height (m)
KP2	555464.340	546588.241	61.006
KP3	555269.850	546425.678	124.305
KP4	555873.335	546326.441	133.253
KP5	555894.330	546520.897	65.427

Table 1: Coordinate of the Reference Stations for the First Epoch- October 1984

	N orthing (m)	Easting (m)	Height (m)
KP2	555464.349	546588.240	61.008
KP3	555269.862	546425.676	124.301
KP4	555873.329	546326.453	133.245
KP5	555894.330	546520.897	65.435

Table 2: Coordinate of the Reference Stations for the Second Epoch - December 1994

	Northing (mm)	Easting (mm)	Height (mm)
KP2	9	-1	2
KP3	12	-2	-4
KP4	-6	12	-8
KP5	0	0	8

Table 3: Differences In Reference Stations Coordinates Between the Two Epochs

4.2.2 Observation of the Object Points

The terrestrial geodetic observations made from the control pillars (KP3-KP4 and KP2-KP5) to the object points consist of measurements of slope distances, horizontal angles, zenith angles, instrument heights and target heights. All observations are made using high precision geodetic theodolite and EDM equipment.

4.3 Network Adjustment and Analysis

The network adjustment of data from both epochs is accomplished using the Star-Net program. The standard deviations assigned to the various types of observations are shown in Table 4. The adjustment process converged after 4 iterations for the first epoch and after 3 iterations for the second epoch.

Distances	0.001m
Angles	4 Seconds
Directions	3 Seconds
Azimuth / Bearings	4 Seconds
Zeniths	10 Seconds
Centering Error: Instrument	0.000 m
Centering Error: Target	0.000 m

Table 4: Observation Standard Deviations and Other Settings

The results of the adjustment are summarised in Tables 5-7 and Tables 8-10 for the first epoch and second epoch, respectively.

Summaries of Adjustment Results for the First Epoch

Observation	Count	Sum Squares of StdRes	Error Factor
Angles	62	5.39	0.42
Distances	62	40.20	1.14
Zeniths	62	208.38	2.59
Total	186	253.98	1.65

Table 5: Observation Summary

	Magnitude								
Observations	**Residual**			**Std Err**			**Std Res**		
	Max	**Min**	**RMS**	**Max**	**Min**	**RMS**	**Max**	**Min**	**RMS**
Adjusted Angle	1."58	-0."69	0."40	2."00	0."30	1."01	0."80	0."00	0."30
Adjusted Distance(mm)	1.2	-9.8	5.3	9.0	6.0	6.8	1.4	0.1	0.8
Adjusted Zenith	2."42	-6."89	2."75	1."50	1."50	1."50	4."60	0."10	1."84

Table 6: Residuals

	Standard Deviations		
Stations	**Max (mm)**	**Min (mm)**	**RMS (mm)**
Northing	4.3	0.0	2.4
Easting	6.3	0.0	3.2
Elevation	3.3	0.0	2.4

Table 7: Standard Deviations of Adjusted Coordinates

Summaries of Adjustment Results for the Second Epoch

Observation	Count	Sum Squares of StdRes	Error Factor
Angles	62	238.84	2.78
Distances	62	212.41	2.62
Zeniths	62	149.36	2.20
Total	186	600.61	2.54

Table 8: Observation Summary

	Magnitude								
Observations	**Residual**			**Std Err**			**Std Res**		
	Max	**Min**	**RMS**	**Max**	**Min**	**RMS**	**Max**	**Min**	**RMS**
Adjusted Angle	8."58	-5."39	2."88	1."71	1."14	1."41	5."50	0."20	1."96
Adjusted Distance (mm)	1.2	-1.7	2.6	1.7	1.1	1.3	8.2	0.0	1.8
Adjusted Zenith	2."62	-7."93	2."80	2."00	1."00	1."81	4."40	0."00	1."55

Table 9: Residuals

	Standard Deviations		
Stations	**Max (mm)**	**Min (mm)**	**RMS (mm)**
Northing	3.1	0.0	2.5
Easting	3.6	0.0	2.9
Elevation	6.1	0.0	4.4

Table 10: Standard Deviations of Adjusted Coordinates

Notes

The residual is the difference **between** the value **we observed in the field, and the value that** fits **best into the** final adjusted **network**. The **standardized residual** (StdRes) is the actual residual **divided by its standard** error value. While the Sum Squares of StdRes means that each StdRes is squared and summed, it is **functional to the number of observations of that data type**. The total displayed **in the error factor column are adjusted by the number of observations, and** it is an **indication of how well each** data **type fits into the adjustment. The error factor** should be roughly **equal or approximately be in a range of 0.5 to 1.5. The error factor** may be large for several **reasons as there may** be one or more large errors in the input data, a systematic error, or the **assigned standard** errors are unrealistically small.

The Star-Net software also provides graphical display for network quality analysis in terms of the absolute and relative error ellipses (see Figures 3 and 4).

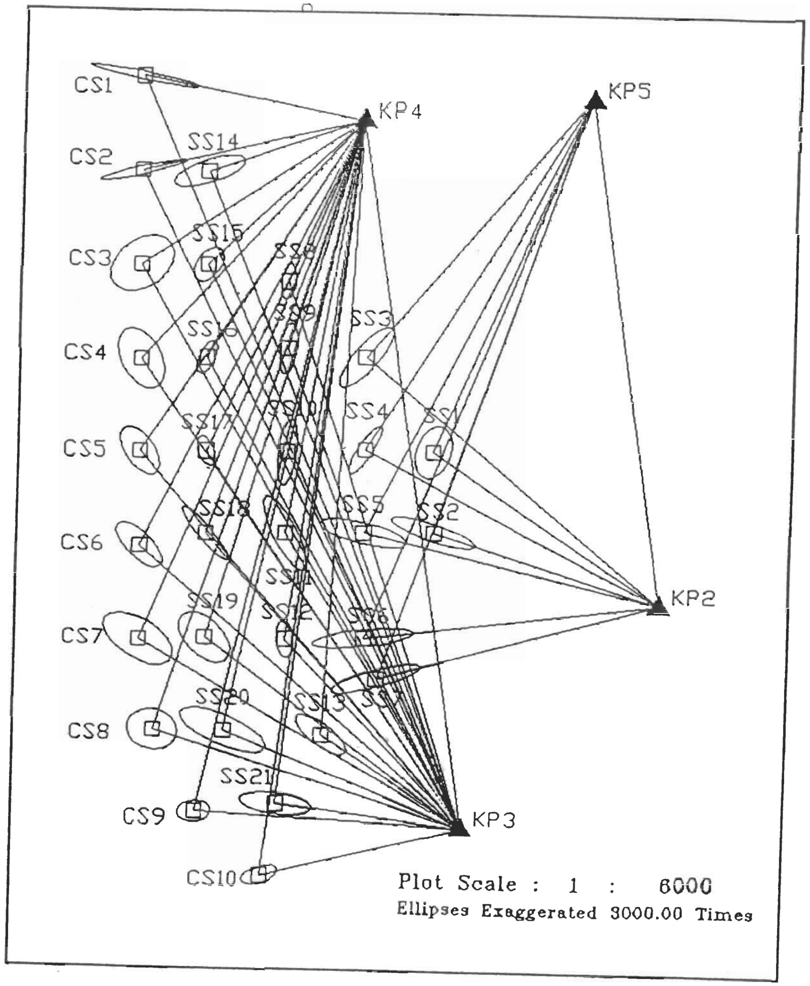

Figure 3 : Graphical Display of Station Error Ellipses (First Floor)

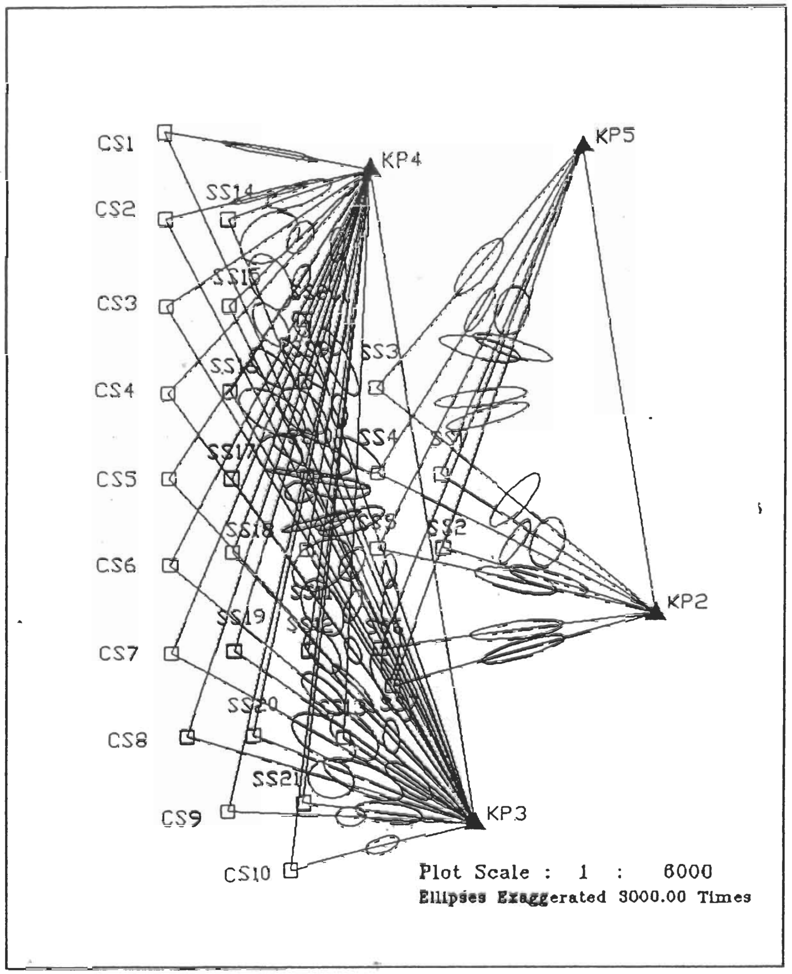

Figure 4: Graphical Display of Relative Error Ellipse (First Epoch)

4.4 Displacement Analysis

In order comparison of results as obtained from the Star-Net can be performed, a subroutine called "DETECT" has been written to enable the calculations of the displacement vectors. The equations used to compute the necessary quantities are shown in the following:

The total horizontal displacement magnitude, MAG:

$$MAG = \sqrt{\Delta N^2 + \Delta E^2} \tag{8}$$

where $\Delta N = N_1 - N_2$ and $\Delta E = E_1 - E_2$; the subscripts indicate the different epochs.

The standard deviation of horizontal displacement:

$$\sqrt{\left(\frac{\Delta N}{MAG}\sigma_{\Delta N}\right)^2 + \left(\frac{\Delta E}{MAG}\sigma_{\Delta E}\right)^2} \tag{9}$$

where

$$\sigma_{\Delta E} = \sqrt{\sigma_{E_1}^{\ 2} + \sigma_{E_2}^{\ 2}}$$

$$\sigma_{\Delta E} = \sqrt{\sigma_{E_1}^{\ 2} + \sigma_{E_2}^{\ 2}}$$

The vertical displacement:

$$\Delta H = H_1 - H_2 \tag{10}$$

and the standard deviation of the vertical displacement :

$$(\sigma_{\Delta H}) = \sqrt{\sigma_{H_1}^{\ 2} + \sigma_{H_2}^{\ 2}} \tag{11}$$

Direction of the displacement;

$$DIRECTION = TAN^{-1}\left[\frac{\Delta E}{\Delta N}\right] \tag{12}$$

The results from the above displacement calculations are shown in Table 11. Figures 5 and 6 depict the magnitude of the horizontal and vertical displacements of the dam tha has occurred within the last ten years, respectively. The maximum vertical displacement of -487.1mm was observed at point CS6, while the maximum horizontal displacement of 162.1mm was observed at point CS8. Both points CS6 and CS8 are located at the dam crest. On the other hand, point SS7 and and SS1 showed a minimum vertical and a minimum horizontal displacements of -1.4mm and 18.1 mm, respectively. Both points SS1 and SS7 are located at the lower end of the dam structure (see Figure 2). The results shown in Figures 5 and 6 indicate that the displacements that has occurred over the last ten years is a result of the settlement process of the dam structure and the displacement patterns did not indicate any abnormal behaviour.

STN	HORIZONTAL			VERTICAL	
	MAG (mm)	SD MAG (mm)	AZI (Deg)	MAG (mm)	SD MAG (mm)
CS1	28.4	3.2	167.38362	-44.7	4.8
CS2	69.2	3.0	187.97781	-138.8	4.7
CS3	123.1	4.4	161.77115	-216.4	5.3
CS4	148.5	4.4	152.31334	-332.1	5.8
CS5	105.1	3.9	134.22891	-450.7	6.4
CS6	68.7	4.1	47.88889	-487.1	6.9
CS7	153.0	4.3	8.79741	-400.0	6.8
CS8	162.1	3.7	3.78585	-261.8	6.3
CS9	102.8	3.0	350.37282	-150.3	5.2
CS10	49.0	2.8	337.46101	-57.5	4.3
SS1	18.1	3.6	75.27255	-7.9	4.9
SS2	24.5	5.5	71.19541	-2.7	5.0
SS3	44.0	4.1	89.60935	-17.7	4.1
SS4	75.7	3.3	84.01080	-45.7	5.9
SS5	90.5	5.7	79.17425	-46.0	5.9
SS6	51.4	6.1	66.23008	-18.8	5.9
SS7	20.8	4.3	30.01837	-1.4	6.1
SS8	64.9	2.9	96.19390	-39.3	4.0
SS9	97.8	3.4	93.57633	-79.5	4.8
SS10	128.2	3.8	89.55308	-116.5	5.5
SS11	138.0	4.5	81.95800	-134.8	5.8
SS12	105.9	3.4	61.45001	-89.8	5.1
SS13	20.9	3.4	28.65431	-7.7	2.7
SS14	29.5	4.5	119.43179	-29.6	3.6
SS15	107.3	3.3	120.72556	-124.1	4.1
SS16	151.4	3.5	113.75504	-226.0	5.2
SS17	158.5	3.5	102.94044	-329.1	6.3
SS18	157.4	4.4	77.93321	-364.5	6.4
SS19	159.5	4.3	50.90266	-274.6	6.0
SS20	126.7	4.6	34.96979	-134.3	5.2
SS21	47.6	2.6	8.82859	-35.1	4.2

Table 11: Horizontal and Vertical Displacement Calculations

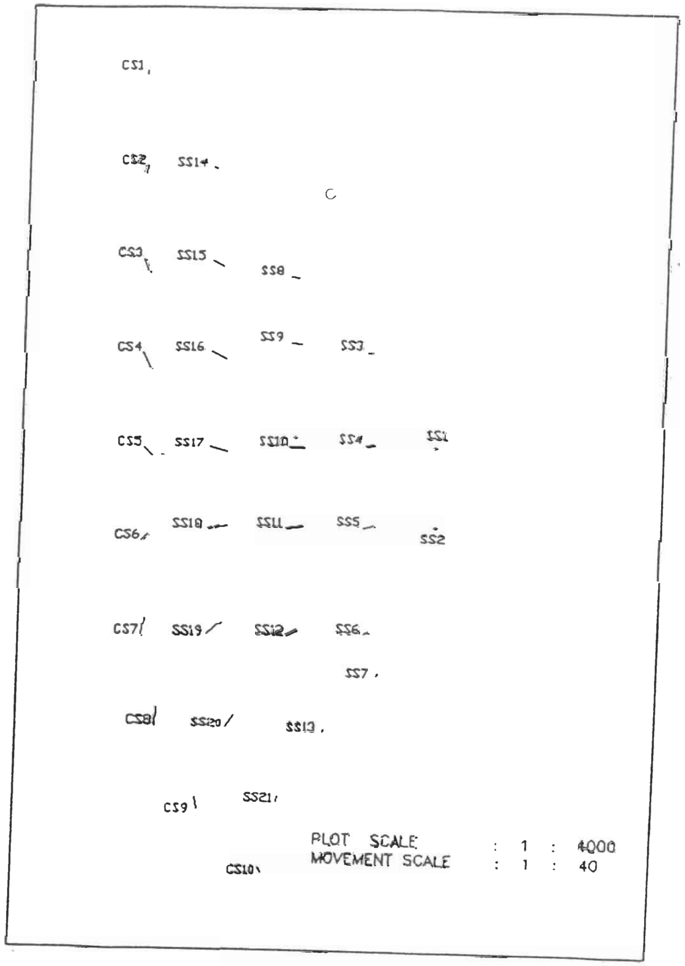

Figure 5: Horizontal Displacement Field

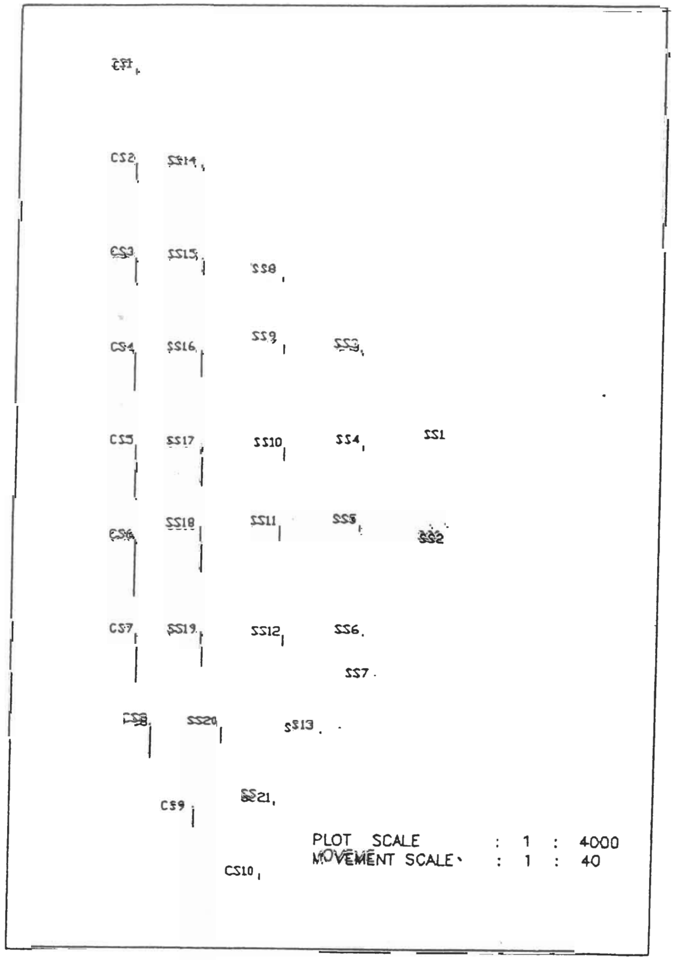

Figure 6: Vertical Dispcement Field

5.0 CONCLUSIONS

Monitoring of dams is one of the classics topics for the application of geodesy to engineering. Three dimensional (3D) network calculation and adjustment are tasks which occur in many areas of engineering technology. We have shown this paper the application of least squares technique for the adjustment of a large 3D dam monitoring network. It is extremely important that the monitoring network is properly adjusted and analysed before the results are used in the deformation analysis.

The analysis of deformations surveys include: (i) geometrical analysis which describes the geometrical status of the deformable body, its change in shape and dimensions, as well as the translation and rotation of the whole deformable body with respect to a stable reference frame; and (ii) physical interpretation. We have presented in this study a simple direct comparison technique which merely serves to detect the displacements (horizontal and vertical) of the dam structure. Our displacements results conform to the belief that over the last ten years the dam has undergone a substantial settlement process.

On the data acquisition technique, we proposed that an Automatic Theodolite Measuring System (ATMS) for the monitoring the dam structure and GPS surveying for backward control be implemented in the monitoring of a dam structure. It should be possible to transfer this concept with ease to different monitoring tasks as well.

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Integration of Remote Sensing-GIS Techniques for Mapping and Monitoring Seagrass and Ocean Colour off Malaysian Coasts

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Abstract

This paper describes seagrass and ocean colour mapping off Peninsular Malaysia. The seagrass were extracted from visible bands of Landsat TM using the depth invariant index of the seabottom type. The ocean colour which much referred to plankton concentration is derived by regressing samples from known site collected at time of satellite overpass. Oth these information were then input into GIS database which were also being established to assist the Marine Fisheries Management and Development Centre in managing and monitoring coastal areas. This paper also addresses the experience gained in building spatial database for coastal areas various data collected from various mapping environments were carried out.

INTRODUCTION

The ocean is an important asset for a country because of its varius valuables resources such as food, source of energy, tourism and as an entrance into the country. As a marine source of food, seagrass and planktons are two important parameters in fisheries. Seagrass are commonly found in shallow coastal marine locations, salt-marshes and estuaries, and in the topics they are often associated with mangroves. Seagrass ecosystems provide habitats for a wide variety of marine organisms, both plant and animal, these including fauna and flora, bethis flora and fauna, epiphytic organisms, plankton and fish, not to mention microbial and parasitic organisms. The relatively high rate of primary production of seagrass also drives detrius-based chains, which help to support many of these organisms.

Plankton, on the other hand, is an important part of these ocean, which often associated with the optical proeprties of water i.e. the ocean or sea colour (colourization of water due pigmentation found in plankton). Plankton, a unicell plants that floats in areas near the sea surface also plays an important part in the food cycle of fishery environment. The plankton populations are dependent on variety of factors, including ocean, temperature, availability of nutrient, amount of sunlight and ocean depth.

The importance of seagrass and plankton are that they are often correlated to the fish breeding grounds, hence, measuring these two factors can assist in indentifying both fish breeding and fishing grounds. In this paper, both ocean colour can seagrass mapping from satellite remote sensing data off eastern coast of Malaysia comprising of clustered islands priorily gazetted by the Malaysian Government as one of the 5 national marine parks.

MATERIAL & METHODS

Geographic Information System

Initiative and precious studies carried out at Centre for Remote Sensing (CRS), UTM has given high awarness and sentivity of the appropriate authority in using GIS for assisting marine parks management. Recently, CRS was given a research grant under Intensification of Research in Priority Areas (IRPA), Ministry of Science Technology & the Environment to undertake pilot study to