INTEGRATION OF GPS AND LEVELLING FOR SUBSIDENCE MONITORING STUDIES AT COSTA BOLIVAR OIL FIELDS, VENEZUELA

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PREFACE

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INTEGRATION OF GPS AND LEVELLING FOR SUBSIDENCE MONITORING STUDIES AT COSTA BOLIVAR OIL FIELDS, VENEZUELA

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PREFACE

This technical report is a reproduction of a thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Engineering in the Department of Surveying Engineering, April 1989. A few editorial modifications have been introduced by Dr. Y.Q. Chen, and by Dr. Adam Chrzanowski who supervised the research. Funding was provided partially by Maraven S.A. of Venezuela, the Natural Sciences and Engineering Research Council of Canada, and by the National Research Council of Canada's Industrial Research Assistance Program agreement with Usher Canada Ltd.

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ABSTRACT

Monitoring of ground subsidence has been traditionally performed by means of geodetic levelling techniques. Geodetic levelling is slow and costly, requiring long connection lines to stable areas, and higher densification in critical areas to properly depict the deformation behaviour. The Global Positioning System (GPS) has been envisioned as an attractive alternative in the domain of deformation monitoring, bringing about potential savings without significant deterioration in accuracy.

The Costa Bolivar oil fields in Venezuela have been subject to subsidence since 1926 at a rate of 20 cm/year. The monitoring scheme has been based on geodetic levelling and an already obsolete computational methodology. A full evaluation of the whole scheme has revealed a total uncertainty of 20 to 30 mm at the 95% confidence level for the subsidence determination and of 15 to 20 mm at the 95% confidence level for the absolute elevations.

A methodology to integrate GPS with levelling in order to modernize and optimize the present monitoring scheme has been designed. The results of pilot tests to evaluate the real accuracy of GPS in the area using WM101 receivers, show an accuracy of 29 mm independent of baseline length. Accuracy standards developed for the optimal integration reveal, however, that relative GPS accuracies in the order of 10 to 15 mm are needed for compatible results with levelling. The results of an economic analysis on the designed integration network shows savings in the order of 26% in the cost of one campaign which is an indication of the feasibility of GPS when used in combination with levelling for subsidence monitoring studies.

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1. INTRODUCTION

Ground subsidence deformation has been a common hindrance to mining operations and other engineering activities throughout the world. The most critical cases have been usually connected with oil, gas, or water withdrawal. Typical examples are areas of Wilmington in California, in the United States, Niigata in Japan, Mexico City in Mexico, and the Costa Bolivar oil fields in Venezuela [Poland and Davis, 1969]. Many other cases are discussed in Johnson et al. [1984].

In general, the problems associated with ground subsidence may be summarized as flooding, failure of engineering structures, devaluation of properties and reverse flow of drainage systems. Consequently, there is an obvious need for the evaluation and prediction of the deformation in order to minimize its impact on the surface environment.

In Venezuela, the first traces of subsidence deformation were detected in 1929 in the area of Lagunillas, located on the Costa Bolivar Oil Fields, along the eastern coast of Lake Maracaibo (Figure 1.1). The main cause of subsidence was reported to arise from the exploitation of relatively shallow (300 to 1000 m deep) oil reservoirs composed of highly porous and compressive unconsolidated material [Murria and Abi Saab, 1988]. The geomorphology of the area and its geographical location imposed serious limitations for future development. A quotation from Kugler [1933] serves as a good example of this limitation: "... subsidence was very bad, as you probably know, the wharf was disappearing under the lake..". Under such circumstances a monitoring levelling scheme was implemented in 1929. The collected information was, and still is, a source of valuable information for managers and engineers involved in the development of the Costa Bolivar oil fields. The information is generally used in the design of drainage master plans and coastal protection dykes, oil exploitation and urban planning, calibration of subsidence prediction

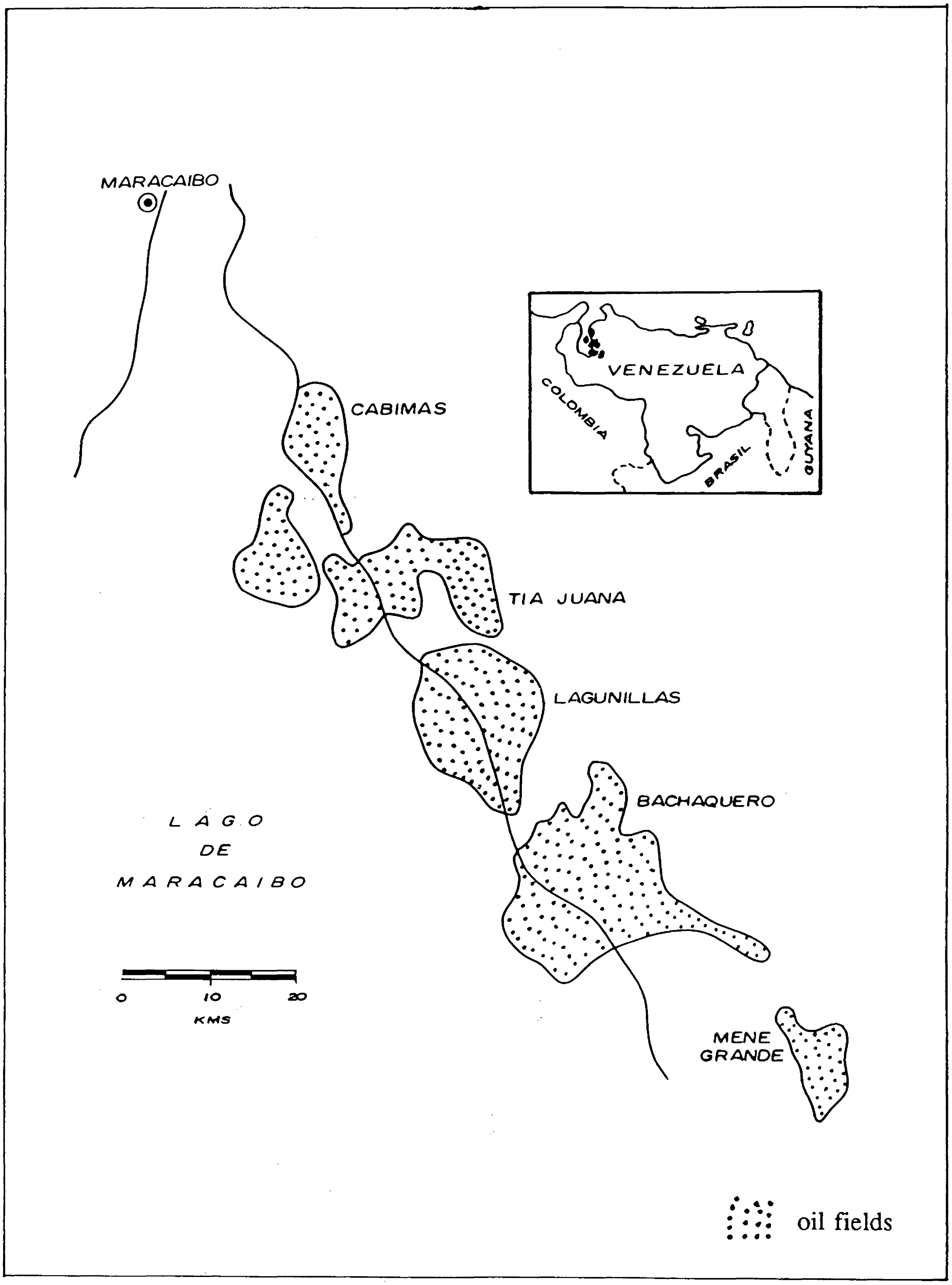

Figure 1.1. Relative Location of the Costa Bolivar Oil Field
[after Puig, 1984]

models and most recently in the design of a contingency plan for the area in case of damage to the protective dikes. The monitoring network has formed the basic vertical control for all engineering projects in the area.

Up to the present time, all of the monitoring activities have been undertaken by Maraven S.A., one of the major oil companies in Venezuela and a subsidiary of Petróleos de Venezuela S.A. (a state-owned holding company). Costly and slow geodetic levelling techniques have been used in the monitoring. Presently, the main surveys of the whole inland subsidence area of about 1300 km^2, consisting of 1624 benchmarks, are repeated at 2 year intervals with a portion of the network (Tia Juana section which is about 1/3 of the total area covered by the network) being remeasured every six months for the purpose of checking the stability of off-shore platforms [Leal, 1987]. The main survey requires about 2 months for 6 survey crews to complete. A detailed description and evaluation of the levelling scheme is given in Chapter 3.

Since 1984, as part of the efforts of Maraven S.A. to maximize productivity under the implementation of modern technology, consultants from the University of New Brunswick have been involved in the subsidence study to modernize and economize the present monitoring scheme. Major improvements were sought by modifying the field techniques. Motorized trigonometric height traversing emerged as a possibility but it was soon turned down as it provides little advantage over geodetic levelling in flat topographic conditions. Differential satellite "Global Positioning System (GPS)" techniques, however, seemed to offer a feasible alternative. The commonly known advantages of GPS and claimed achievable accuracies led to a proposal for replacing the main levelling network by GPS baselines in combination with lower order levelling surveys used for densification purposes. The GPS network was meant to replace all connecting lines to stable areas and to adjacent subnetworks in other subsidence fields as well as to add information on the horizontal behaviour of the deformation. As a first step, two test surveys were conducted on the Tia Juana section of the main monitoring network in April 1987 and in October 1987 with levelling surveys carried out at the same time to monitor subsidence of the offshore platforms. Evaluation of the real performance and accuracy of GPS surveys under the extreme climatic conditions of the Costa Bolivar oil fields was the main aim of the test surveys. Results of these test surveys are discussed in Chapter 4.

Despite some difficulties, encouraging results were obtained from the two test campaigns when comparing subsidence values obtained from GPS and from levelling surveys. As a result, a full survey of the whole main network using GPS was conducted in April 1988. Levelling surveys corresponding to the biannual monitoring campaign took place at more or less the same time.

On the basis of this antecedent, the main objective of this thesis has been to design a methodology for integrating GPS and levelling surveys as a future survey scheme for monitoring the subsidence in the whole area of the Costa Bolivar oil fields. The work has developed around several tasks which are considered to be the contributions of the author. They are listed as follows:

a) Choice of an adequate subsidence model;
b) accuracy evaluation of the Maraven monitoring scheme;
c) accuracy evaluation of the GPS derived height differences;
d) development of a model for the integration of GPS with levelling;
e) development of accuracy standards;
f) development of a general field and computational strategy to implement the new design;
g) economic analysis to study the feasibility of the new approach.

Principles of the UNB Generalized Method of deformation analysis [Chrzanowski et al., 1983; Chrzanowski et al., 1986] have been employed in the development of the mathematical model for the integrated surveys and in the accuracy evaluation.

The discussion is outlined as follows. Chapter 2 conveys a general idea on the aspects of subsidence modelling and describes in more detail the applicable model for the case at hand. Chapter 3 is devoted to fully evaluating the present monitoring scheme. Chapter 4 describes some aspects of the accuracy of the GPS derived height difference and gives a brief evaluation of the GPS test results. Chapter 5 describes the problems encountered in the combination of GPS with levelling and describes the mathematical model and strategy to be used in the integration process. Finally, conclusions and recommendations are given in chapter 6.

2. SUBSIDENCE DEFORMATION MODELLING

The evaluation and prediction of subsidence normally encompasses field monitoring and modelling techniques. Within the general modelling techniques one can distinguish between two distinct approaches — the physical modelling approach which considers the physical laws and properties of materials involved in the deformation, and the geometrical modelling approach based on the superficial geometry of the deformation [Vaníček, 1987]. This chapter deals with the general aspects of geometrical modelling as applied to ground subsidence, within the context of the UNB Generalized Method of deformation analysis [Chen, 1983; Chrzanowski et al., 1986]. A brief general background is first given, followed by a review of deformation models, the model-observations relationship and final remarks.

2.1 General Background

During the past few decades the geodetic community has directed new efforts into the analysis of crustal movements and deformations in general. In 1954 the International Association of Geodesy (IAG) appointed a special study group on crustal deformations and in 1960 established the Commission on Recent Crustal Movements [Pavoni, 1971]. In 1978, Commission 6 of the Fédération Internationale des Géomètres (FIG) created an *ad hoc* committee on the analysis of deformation measurements [Chrzanowski, 1981]. As a result, various modelling strategies and approaches into the analysis of deformations have been developed. Very comprehensive reviews of modelling strategies for vertical crustal movements (VCM) are presented in Holdahl [1978], Gubler [1984], and Vaníček and Sjöberg [1987]. A theoretical review on different approaches to deformation analysis developed within the last decade, is given by Chrzanowski and Chen [1986].

Most strategies have been developed for modelling regional vertical crustal movements based on scarce and heterogeneous data, such as: relevellings of national geodetic levelling networks and small networks used in engineering and mapping projects, sea level variations, lake tilt data and detached relevelled segments.

The models have found wide application in local subsidence studies with the advantage that the available subsidence data are characteristically more homogeneous and abundant in the form of complete levelling networks (with no configuration defects), observed at regular time intervals and confined to short periods of observation. Consequently, there is more flexibility for rigorous deformation analysis in the geometrical interpretation of local subsidence deformation than in the regional VCM studies.

Although subsidence deformation is within the realm of VCM, a clear distinction between the general objective of vertical crustal movements versus subsidence deformation studies must be made. Vertical crustal movement studies have been conducted to gain deeper knowledge on the pattern and behaviour of fairly extensive areas and to interpolate or extrapolate corrections to homogenize observations gathered over considerable periods of time. This is of particular interest to geodesists since it allows performance of simultaneous adjustments of very extensive networks, for instance, the adjustment of national vertical networks or continental networks. Subsidence studies, on the other hand, are generally conducted to evaluate the extent of man-induced subsidence in order to make decisions on exploitation policies and planning, and on the design of engineering projects. In addition, most engineering activities in the affected areas are usually tied to the vertical geodetic control defined by the subsidence monitoring networks which poses higher demands on the analysis of the subsidence deformation, to guarantee the results with a greater degree of confidence.

2.2 Deformation Models

According to Chen [1983] and Chrzanowski et al. [1983] the deformation of a body is fully described if the displacement field d (x, y, z; t - t_o) is known. The displacement field can be approximated by fitting a selected deformation model to displacements determined at discrete points

$$\mathbf{d}(x, y, z; t-t_o) = \mathbf{B}(x, y, z; t - t_o)\mathbf{c} \tag{2.1}$$

where **d** is the vector of displacement components of point (x, y, z) at time t with respect to a reference time t_o,

B is a matrix of base functional values and

c is the vector of unknown coefficients.

The mathematical model (2.1) can be explicitly written as

$$\mathbf{d} = \begin{pmatrix} \mathbf{u}(x, y, z; t-t_o) \\ \mathbf{v}(x, y, z; t-t_o) \\ \mathbf{w}(x, y, z; t-t_o) \end{pmatrix} = \begin{pmatrix} \mathbf{B}_u(x, y, z; t-t_o)\mathbf{c}_u \\ \mathbf{B}_v(x, y, z; t-t_o)\mathbf{c}_v \\ \mathbf{B}_w(x, y, z; t-t_o)\mathbf{c}_w \end{pmatrix} \tag{2.2}$$

where **u**, **v** and **w** represent displacement components in the x, y and z directions respectively. Since in subsidence studies we are mainly interested in the vertical component (w or z) and since subsidence is generally independent of height, the general model for subsidence deformation can be reduced to

$$w(x, y; t-t_o) = \mathbf{B}_W(x, y; t-t_o)\mathbf{c}_W \tag{2.3}$$

which in short form can be written as:

$$w = \mathbf{Bc} \tag{2.4}$$

where **B** is a row vector.

Different suitable functions may be used to approximate the deformation. A common approach is to use algebraic polynomials.

Consider the general three dimensional polynomial

$$w(x, y; t-t_o) = \sum_{k=1}^{n_t} \sum_{i=0}^{n_y} \sum_{j=0}^{n_x} x^j y^i (t-t_o)^k c_{jik} \tag{2.5}$$

where n_x, n_y and n_t are the maximum degree of the polynomial in the x, y and time coordinates respectively, and c_{jik} is the polynomial coefficient with total number $n = (n_y+1)(n_x+1)n_t$. Depending on the variations in n_x, n_y and n_t, different models may be derived. Typical models using polynomials are given below.

a) Velocity surface model

The model results from considering a linear deformation with time equivalent to $n_t = 1$, n_x and n_y vary according to the spatial shape of the deformation.

For the deformation of one continuous block, equation (2.5) becomes

$$w(x, y; t-t_o) = \sum_{i=0}^{n_y} \sum_{j=0}^{n_x} x^j y^i (t-t_o)\, \mathbf{c}_{ji} \, . \tag{2.6}$$

Examples on the application of polynomial velocity surfaces may be found in Vaníček and Christodulidis [1974] and Vaníček and Nagy [1981].

b) Time-varying surface

This model applies for the cases where the deformation is non-linear with time. The model is of the same form as equation (2.5) but with $n_t > 1$. An example on the application of this approach with additional considerations for episodic movements can be found in Vaníček et al. [1979].

c) Discontinuity model

In the presence of discontinuities or to accommodate local anomalies, the area may be divided into several blocks and explicit models written for each block depending on its behaviour. If for example, two Blocks A and B are considered (Figure 2.1a), where all points in B moved together and linearly with respect to A during the interval $(t-t_o)$ (relative rigid body movements), the model will be

$$\begin{aligned} w_A(t) &= 0 \\ w_B(t) &= (t-t_o)\dot{H}_B \end{aligned} \tag{2.7}$$

where subscripts A and B represent the points on the block A and B, respectively, and $\dot{H}$ will be the velocity of vertical movement and equivalent to c in the above equations.

If for instance, the blocks experience linear temporal deformation within themselves (Figure 2.1b) as well as relative body movement, the model for all points in each block will be of the form:

$$w_A(x, y, t-t_o) = \sum_{i=0}^{n_y} \sum_{j=0}^{n_x} (x-x_o)^j (y-y_o)^i (t-t_o)\, c_{ij}$$

$$w_B(x, y, t-t_o) = \sum_{i=0}^{n'_y} \sum_{j=0}^{n'_x} (x-x_o)^j (y-y_o)^i (t-t_o) b_{ij}, \tag{2.8}$$

where point $x_0\, y_0$ is a reference point. Different combinations of all the above cases may be used depending on the following factors: the desired accuracy of the modelling, the redundancy of the observations, the number of available epochs, and the distribution of the data.

An alternative approach in some cases has been the use of multiquadric analysis (Hardy [1978], and Holdahl and Hardy [1979]), whereby the polynomial is replaced by a suitable quadric form. According to Holdahl and Hardy [1979] this has the advantage of producing more appropriate automated graphic representations of the subsidence at extreme values outside the data area.

For the case of the Costa Bolivar in Venezuela, the subsidence monitoring network constitutes the basic vertical control for all the engineering activities undertaken in the area. Therefore, a knowledge of the subsidence of each individual benchmark is the immediate goal of the subsidence study. Spatial modelling obtained through a surface fitting, although suitable for most general purposes, will not provide the most accurate elevations at discrete points, especially if one considers the irregular shape of the Costa Bolivar subsidence deformation (see Figure 2.2 below). Thus, the subsidence values for each individual

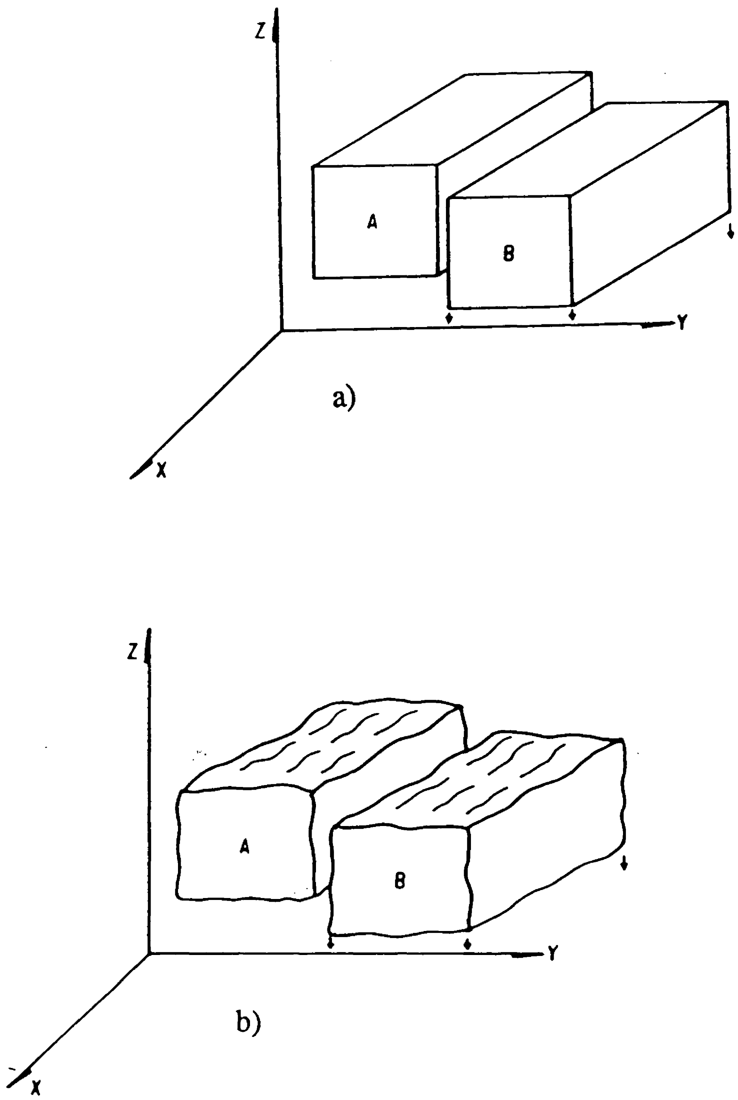

Figure 2.1. Examples of deformation with discontinuity

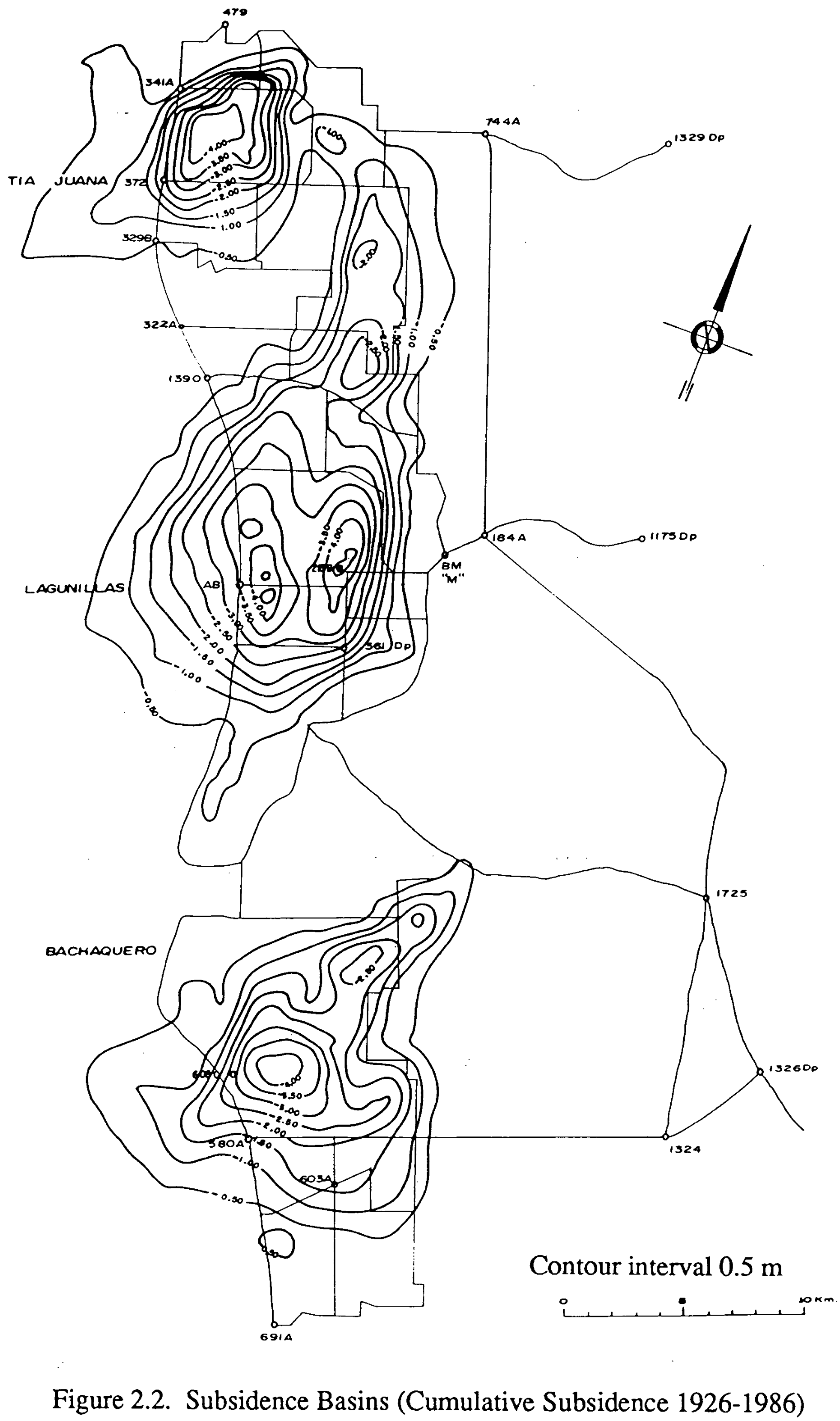

Figure 2.2. Subsidence Basins (Cumulative Subsidence 1926-1986)

benchmark must be modelled and derived first. Later on, any desired surface fitting to the vertical displacements may be performed either analytically or graphically in order to obtain a graphical representation of the subsidence basin.

As it will be discussed in Chapter 3, the subsidence along the Costa Bolivar, at least within a time span of a few years (approximately 10 years), seems to follow a linear trend. Therefore, each particular benchmark, at least initially, may be considered as a rigid block undergoing linear displacement in time with respect to a stable block represented by the benchmarks located in a stable area. The constant velocity model, equation (2.7), has been selected for the subsidence modelling which is discussed in more detail in the next section as well as in Chapter 5.

2.3 Discrete Point Constant Velocity Model

In the case of subsidence monitoring studies, most observables fall under two general types: either height difference or tilt observations. They encompass all the geometrical data such as: tide gauge observations, relevellings, direct tilt measurements and spatial position changes.

From the principles of the generalized approach to deformation analysis [Chrzanowski et. al., 1983], it follows that for the model estimation, the relationship between the deformation model and the observables could be established through the general equation

$$\ell(t_o) + \mathbf{v}(t_o) = \mathbf{A}_0\xi \qquad (2.9a)$$

$$\ell(t_i) + \mathbf{v}(t_i) = \mathbf{A}_i\xi + \mathbf{A}_i\mathbf{B}'_i\mathbf{c} \qquad (2.9b)$$

where $\ell(t_i)$ is the vector of observations in epoch t_i ($i = 1, 2, ..., k$)

ξ is a vector of unknown quantities, which may be the coordinates or expected values of the observables or the combination of both at reference time t_o

A is a transfromation matrix from ξ to ℓ and

v is the vector of residuals

B'_i is constructed from the matrix B of deformation model (2.1) relating the unknown coefficients to the change in ξ.

Thus, for a levelled height difference between any two points p_k and p_j at epochs (t_o) and (t_1) the general observation equations may be written in the form

$$\Delta H_{kj}(t_o) + v_{kj}(t_o) = H_j\,(t_o) - H_k(t_o) \qquad (2.10a)$$

$$\Delta H_{kj}(t_1) + v_{kj}(t_1) = H_j\,(t_o) - H_k(t_o) + [\mathbf{b}(x_j, y_j; t_1\text{-}t_o) - \mathbf{b}(x_k, y_k; t_1\text{-}t_o)]\mathbf{c}\ . \qquad (2.10b)$$

Considering the constant velocity model equation (2.7), the general observation equations (2.10) may be rewritten as

$$\Delta H_{kj}(t_o) + v_{kj}(t_o) - H_k(t_o) = \Delta H_j(t_o) \qquad (2.11a)$$

$$\Delta H_{kj}(t_1) + v_{kj}(t_1) = H_j(t_o) - H_k(t_o) + (t_1\text{-}t_o)(\dot{H}_j\text{-}\dot{H}_k)\ . \qquad (2.11b)$$

where the point velocities $\dot{H}_j$ and $\dot{H}_k$ are elements of the vector **c** of the model parameters to be estimated, and $H_j(t_o)$, $H_k(t_o)$ are elements of the vector of unknown constants ξ.

In the case of the subsidence studies of the Costa Bolivar, the only observables used in the subsidence surveys are height differences of individual levelling or GPS lines. Therefore, only the model expressed by equations (2.11) will be discussed.

Taking $\mathbf{A}_i\mathbf{B}'_i = \widetilde{\mathbf{B}}_i$ in the general equation (2.9), a (k+1) multi-epoch solution may be expressed in matrix form as:

$$\begin{pmatrix} \ell(t_o) \\ \ell(t_1) \\ \cdot \\ \cdot \\ \cdot \\ \ell(t_k) \end{pmatrix} + \begin{pmatrix} v(t_o) \\ v(t_1) \\ \cdot \\ \cdot \\ \cdot \\ v(t_k) \end{pmatrix} = \begin{pmatrix} A_o & 0 \\ A_1 & \tilde{\mathbf{B}}_1 \\ \cdot & \cdot \\ \cdot & \cdot \\ \cdot & \cdot \\ A_k & \tilde{\mathbf{B}}_k \end{pmatrix} \begin{pmatrix} \xi \\ \mathbf{c} \end{pmatrix} \qquad (2.12)$$

If only point velocities are desired, ξ could be treated as a vector of nuisance parameters which can be eliminated in the process of the least squares estimation of the **c** parameters using the well known elimination methods. On the other hand, if one is interested in estimating a set of homogeneous heights at a chosen reference epoch (t_o) together with the solution of **c**, then, both ξ and **c** will form part of the estimated parameters in the adjustment.

Another approach to estimate the unknown coefficients **c** is to rise the differences $d\ell$ in a two epoch comparison. In this case, equation (2.9b) is subtracted from (2.9a) if the observables in two epochs are identical, i.e., $A_i = A_o$, and the following equation is obtained:

$$\mathbf{d}\boldsymbol{\ell}(\Delta t_{i-o}) + \mathbf{dv}(\Delta t_{i-o}) = \mathbf{A}_i\mathbf{B}'_i\mathbf{c} \qquad (2.13)$$

where Δt_{i-o} is the time interval between any epoch t_i and the reference epoch t_o. The height differences observation equations (2.11) can then be re-written as:

$$d\Delta H_{kj}(\Delta t_{i-o}) + dv_{kj}(\Delta t_{i-o}) = (t_i - t_o)(\dot{H}_j - \dot{H}_k)\,. \qquad (2.14)$$

The general solution of all the previously discussed cases can be achieved through the application of the least squares criteria. For details on the estimation process and selection of model parameters, the reader is referred to Chrzanowski et al. [1983] and Chrzanowski et al. [1986].

One can adjust the observations for each campaign separately in a static mode and then fit the deformation model to the derived displacements. This has the advantage of allowing data screening and statistical evaluation, as well as trend analysis for the appropriate selection of the deformation model. However, a major limitation is that significant deformation may take place during the data collection period within each survey campaign. Therefore, the kinematic adjustment case discussed above is found to be the most appropriate, especially where large subsidence rates are expected.

In general, the discussion has relied on various assumptions which have been implicitly made and are listed as follows:

i) For the case of discrete point models, it has been assumed that the observations correspond to a complete network with no configuration defects. Otherwise the existence of detached segments will cause singularities in the solution.

ii) At least two observation campaigns of the same network geometry have been assumed to exist.

iii) The gravity variation in the area has been assumed to be sufficiently small to allow observed (levelled) height differences to closely approximate the corresponding geopotential or orthometric difference [Hein, 1986].

2.4 Remarks on Other Approaches

For the sake of completeness another less common approach to modelling, referred to as the stochastic approach [Hein, 1986], has also been used in vertical crustal movements. This approach is based on the method of least squares collocation. The deformation is segregated into three basic parts: a global trend, a regional signal and the noise which includes measuring errors and the individual movements of the benchmarks On this basis the observation equations are set up and solved using the approach of Moritz [1972] as referred to by Hein and Keistermann [1981]. Hein [1986] compares a so-called "mixed" model using this approach against a combined point velocity- multiquadric model showing slight advantages in the results obtained with the mixed model and a major drawback in the error information given by the multiquadric approach. A more general approach to include a wider variety of geodetic data into modelling by this technique is discussed in Hein and Keistermann [1981]. Another approach may be the use of splines but very little has been done in this respect. Additional discussion on the subsidence modelling is presented in Chapter 5.

3. EVALUATION OF THE PRESENT MONITORING SCHEME

This chapter is intended to convey a clear picture of the state of the Maraven subsidence monitoring scheme presently in use, beginning with a brief historical synopsis and general description of the existing monitoring network and computational technique used. It touches briefly on field procedures, discusses economic aspects and presents a fairly complete accuracy evaluation of the last three campaigns.

3.1 Historic Synopsis

The oil extraction in the Costa Bolivar oil fields (see Figure 1.1) began on a small scale in the field of Mene Grande in 1914, followed by Cabimas in 1922 and by commercial exploitation in the field of Lagunillas in 1926 [PDVSA, 1984]. According to Collins [1935], the land adjacent to the village of Lagunillas was mostly swamps and marshes that required the development of a drainage system prior to the development of the oil fields. Trutmann [1949] reports that in 1927 a levelling survey (swamp survey) was conducted for preliminary drainage studies in the area by the Topographical Department of the Venezuelan Oil Concessions Company Ltd. (V.O.C.), part of the Shell Caribbean Consortium in Venezuela. Later on in 1929 the observation of permanent flooding in the production areas raised the suspicion of subsidence in the field, which according to Trutmann [1949] was confirmed by a check on the swamp level survey of 1927 showing subsidence values of the order of 42 cm. This was cause of general alarm and lead to the immediate implementation of a preliminary monitoring scheme. Long connecting lines to the supposedly stable areas were established, and after several campaigns by the middle of 1934 a subsidence rate of 20 cm/year was confirmed [Trutmann, 1949].

As exploitation continued to expand into neighbouring areas over land and offshore, expansion of the monitoring surveys became necessary. Monitoring began in the area of Tia Juana and Bachaquero in the years 1937 and 1938 respectively (taken from the subsidence records available at Maraven S.A.). During a few years, from 1934 to 1942, monitoring was generally carried out annually. After 1942 the surveys were spaced at intervals of two years. It is believed that in the early 1940's the whole monitoring scheme was redesigned, since the VOC company took over from Creole the responsibility for the offshore subsidence monitoring. The offshore subsidence has been monitored by means of water level transfers to well platforms using temporarily installed tide gauges [Leal, 1987].

As time went on, three subsidence basins (polders) have developed above the areas of major exploitation, as depicted in Figure 2.2, where the contour lines represent cumulative subsidence. Consequently, and in response to the requirements of reservoir and construction engineers, the monitoring network has been further expanded and densified. Presently, there exists a main monitoring network covering the fields of Tia Juana, Lagunillas and Bachaquero, and two smaller subnetworks connected to the main network and located in the fields of Cabimas and Mene Grande, whose geographical locations are shown in Figure 1.1.

3.2 Network Description

The main levelling frame is shown in Figure 3.1. It covers a geographical area of about 1300 km^2 and consists of 618.9 km of first order class II (U.S. specifications) levelling lines of which 167.3 km are used for connections to the assumed stable area.

Within the network itself there exists an array of second order class II (U.S. specifications) levelling lines for densification. Figure 3.2 shows in detail a small section of the network illustrating the pattern followed by these second order lines. This pattern is denser in the areas of larger subsidence rates which correspond to larger exploitation zones near the centers of the main "polders" shown above. The total length of the second order

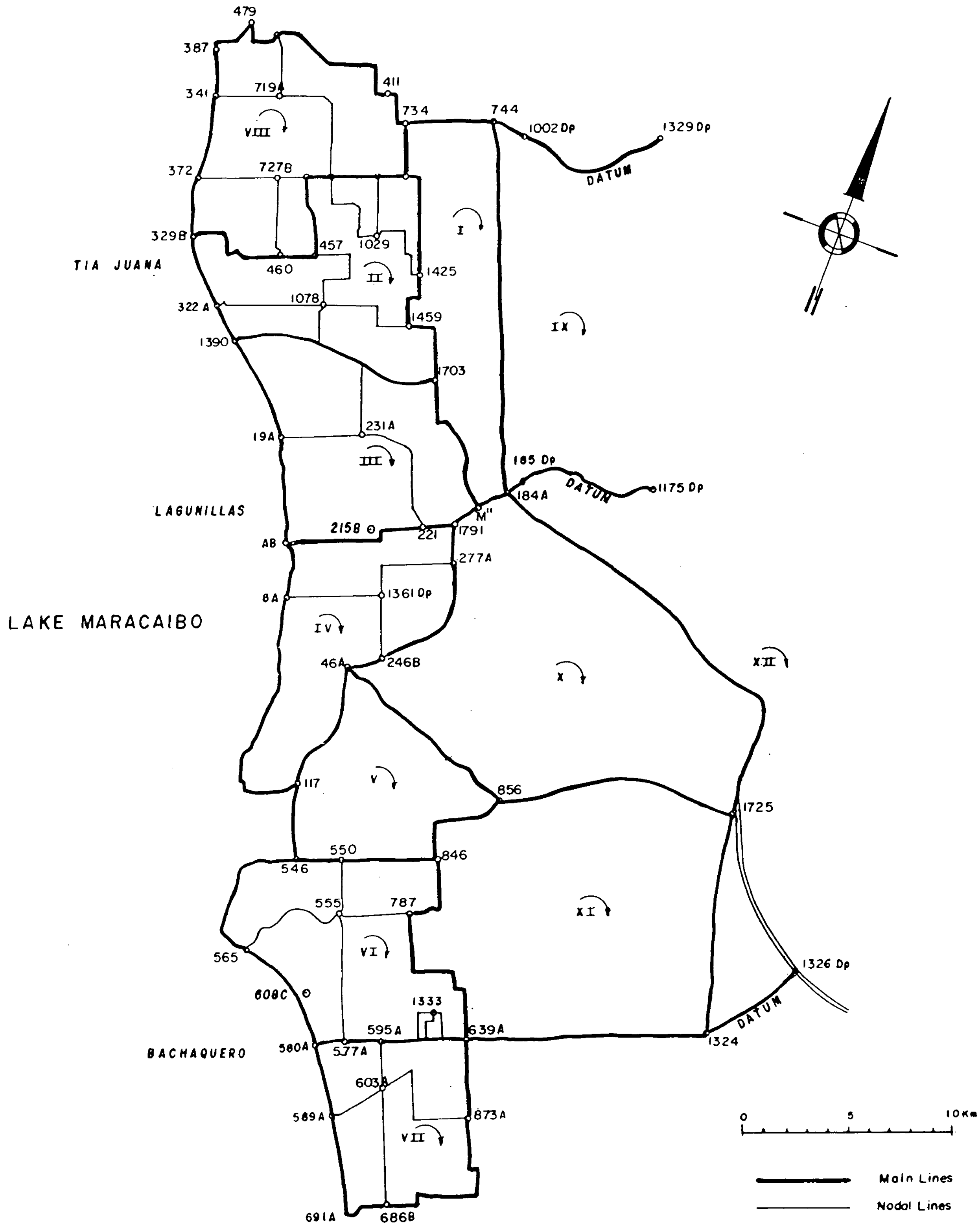

Figure 3.1. Main Levelling Net.

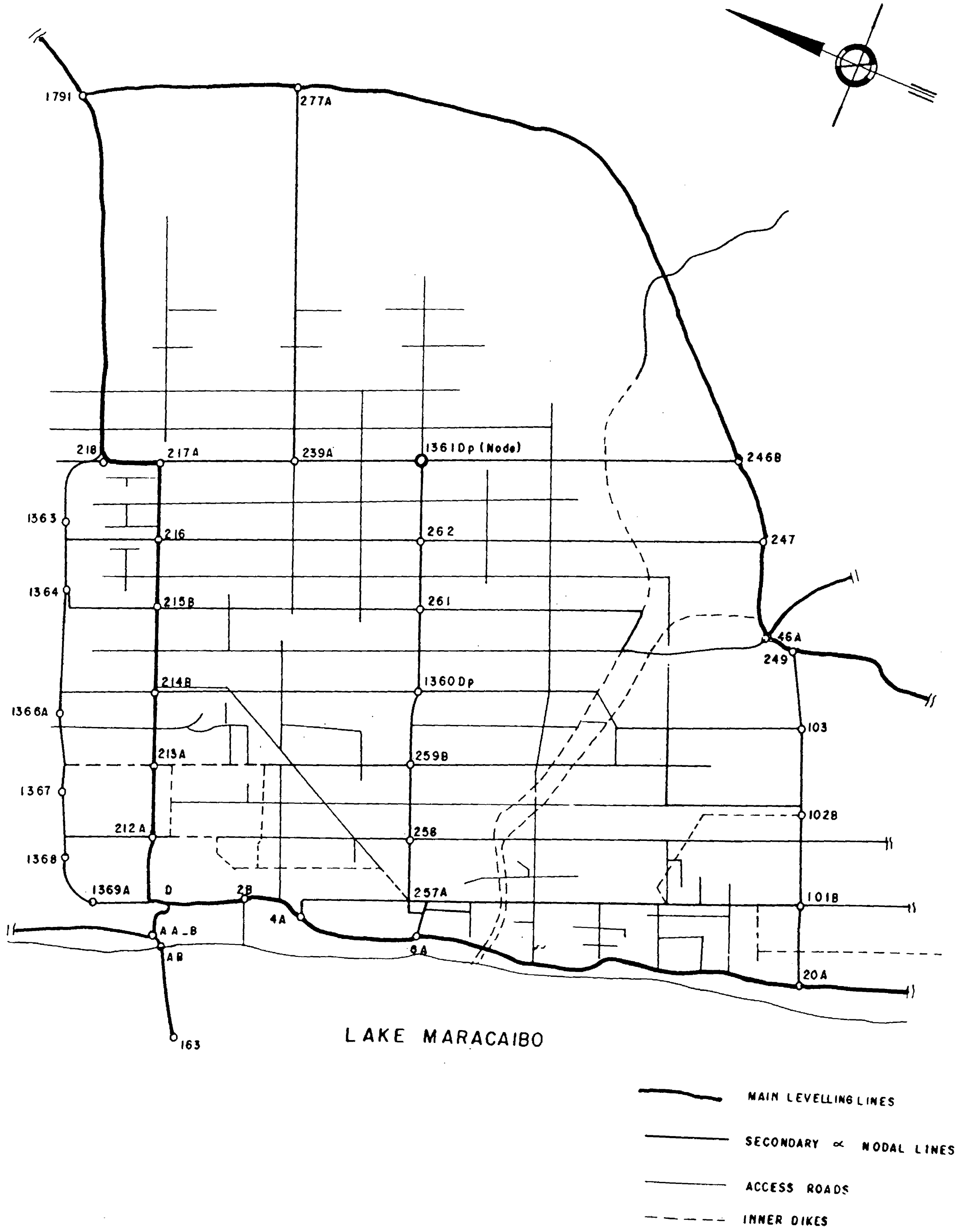

Figure 3.2. Detail showing Secondary and Nodal Lines

levelling lines adds up to 553.7 km. Additionally, the two subnetworks in Cabimas and Mene Grande also consist of first and second order lines which add 160 km of first order lines and 67.3 km of second order lines. The levelling lines which connect both subnetworks to the main network total 68.9 km.

The whole monitoring network, including Cabimas and Mene Grande, consists of 1624 bench marks (BM's), from which two types of monuments could be distinguished: the deep BM's located mainly along the connections to the stable areas (20 ~ 30 km inland) and anchored to a depth of 30 m, and shallow BM's used for densification purposes and connections to the subnetworks. The shallow BM's are cast in concrete inside steel pipes to a depth of approximately 1.7 m. The average spacing between BM's in the network is approximately 400 m.

The offshore subsidence is monitored through an array of 306 well platforms. For the purpose of the analysis herein, only the inland network is considered. A summary of the network characteristics is given in Table 3.1.

3.3 Field Procedures

A total of about one month is needed by 6 survey crews to survey the first order framenet. In order to minimize the accumulation of temporal heterogeneities that could contaminate the observations due to the dynamic behaviour of the subsiding surface, all survey crews work simultaneously starting from outside the subsidence basins toward the areas of maximum subsidence. The field levelling procedures follow closely the requirements outlined in the NOAA [1984] standards and specifications for the U.S. first order class II geodetic control networks. The only exceptions are that temperature gradients are not measured for refraction corrections since the area is mostly flat and the effect of refraction is expected to be greatly minimized by balanced lengths of sight . Gravity measurements have not been taken either. The instrumentation used includes Wild N-3 and NA2 and Zeiss Ni2 levelling instruments, with parallel plate micrometers and invar

Table 3.1. Summary of Costa Bolivar Network Statistics.

DESCRIPTION	MAIN NETWORK	CABIMAS SUB-NETWORK	MENE GRANDE SUB-NETWORK
Total km of first order levelling lines	618.9	97.6	62.5
Total km of second order levelling lines	553.7	38.5	28.8
Total km of levelling lines connecting to stable areas	167.3	--	--
Total km of levelling lines in connections to main network	--	18.4	50.5
Total number of BM's in connecting lines	205	8	37
Total number of BM's	1436	102	86
Area covered (km^2)	1296	66.5	24.8

rods with one or one half centimetre divisions. The second order levelling is performed according to the U.S. standards for second order class II surveys. The same instrumentation as in the first order levelling is used. Measurement of the second order lines takes approximately one month when using 4 survey crews. No specific measuring pattern is followed with the exception of the "nodal lines" which are the lines connecting the main network to specific junction BM's. The nodal lines are measured simultaneously by several crews since they are generally located at places where larger subsidence rates occur. The same procedures are used in the survey of the two subnetworks, Cabimas and Mene

Grande, which require about one month time with one levelling crew. HP 41CV calculators have been used as field data collectors during the last three campaigns, increasing the speed of the field work.

3.4 Data Processing Technique

The basic principles of the data processing method which is described below are believed to have been in effect since the early monitoring times. The general computational sequence presently used at Maraven S.A. is outlined in flowchart form in Figure 3.3. Each step will be briefly explained excluding the computation of the subnetworks of Cabimas and Mene Grande, in lake (offshore) subsidence and graphical representation. The method will be referred throughout this thesis as the Maraven method.

3.4.1 Computation of datum lines

The monitoring network is connected to the assumed stable area through three connecting lines consisting mainly of deep BM's. They are called "datum lines" since they provide the fixed constraints for the network adjustment (see Figure 3.1). The elevations are computed by the following procedure [Shell, 1954]:

(a) A set of provisional elevations is computed for the deep BM's on each "datum line" starting from the elevation obtained in the previous campaign for each extreme BM farthest inland and adding algebraically the averaged height differences observed.

(b) The sum of the provisional elevations of all the deep BM's in each line is compared with the corresponding sum of the elevations in the previous year and the differences are computed.

(c) Based on the assumption that the deep BM's remain completely stable, the differences from (b) are divided by the number of deep BM's in each line. The definitive elevations are finally obtained by adding the estimated correction to the provisional elevations in (a).

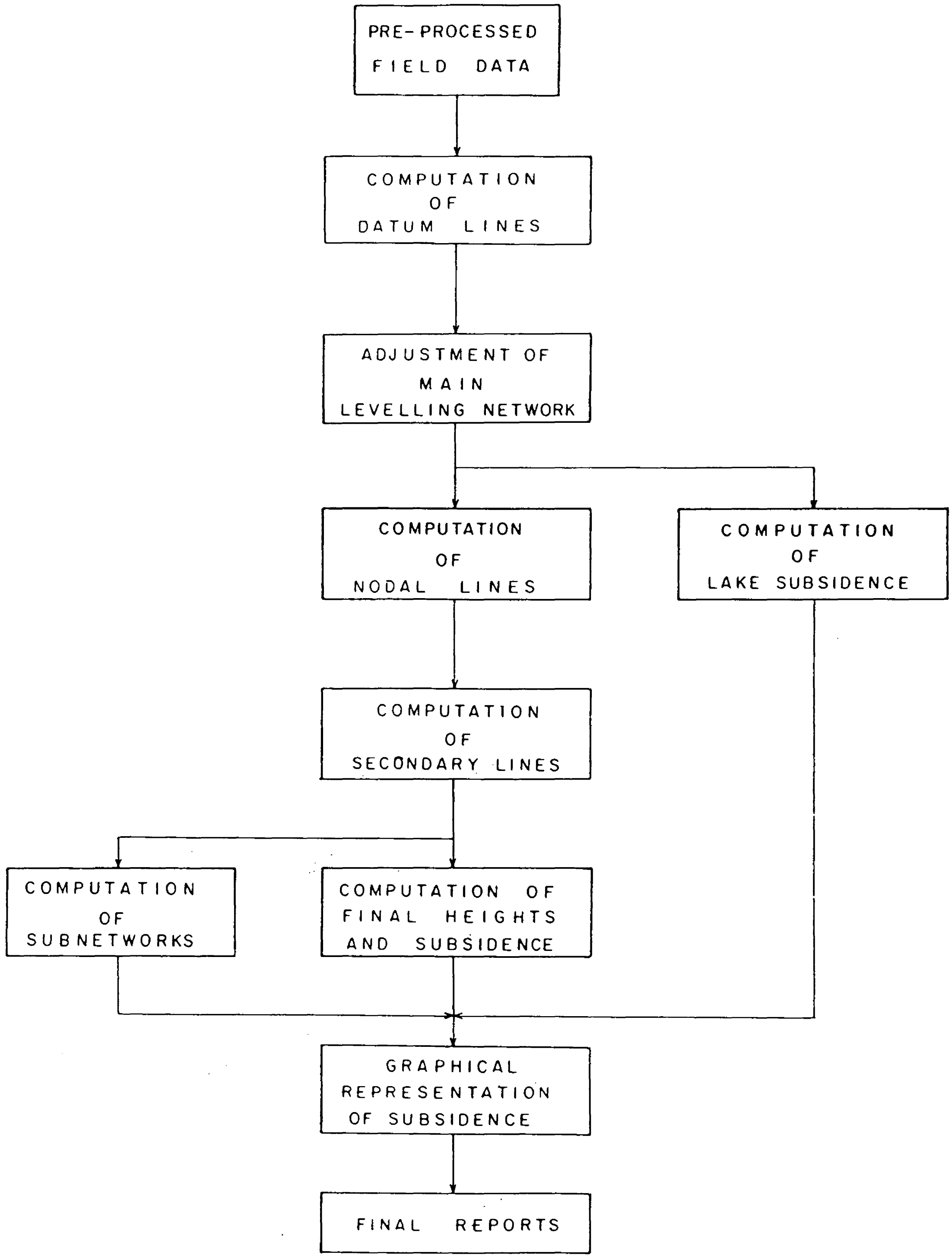

Figure 3.3. Computational Sequence Flow Chart

The new elevations are then considered as fixed for the network adjustment. Thus, in each campaign a new datum is created. Table 3.2 shows the elevations of the extreme BM's in each line for several campaigns. Notice the shifts introduced by this procedure especially on BM's 1175 DP and 1329 DP which are supposed to be the most stable points in the network since they are located farthest inland. This shows that practically **no** BM is actually considered stable between campaigns and the absolute elevations of all points in the network will be systematically affected. This is further discussed in section 3.6.3.

Table 3.2. Elevations of Reference Bench Marks [m].

BM	1980	1982	1984	1986	1988
1329 DP	99.512	99.512	99.511	99.510	99.519
1002 DP	59.508	59.510	59.510	59.511	59.507
1175 DP	53.316	53.320	53.316	53.309	53.302
185 DP	31.134	31.131	31.135	31.155	31.176
1326 DP	54.628	54.629	54.628	54.628	54.628
1324 DP	40.364	40.364	40.365	40.366	40.364

3.4.2 Adjustment of main levelling network

The solution for the first order lines is attained through the least squares adjustment of the main levelling network using condition equations. Twelve independent condition equations are formed as shown by the Roman numerals in Figure 3.1. Constraints are enforced through condition equations IX and XII where the elevations for BM's 1002 DP, 185 DP and 1326 DP, which were computed using the aforementioned procedure and are located on each datum line, are to be treated as fixed. A weight corresponding to the inverse of the length [km] is given to each line. The normal equations are solved using the method of correlates and the solution estimated through the application of the Gauss-Doolitle method [Rainsford, 1957]. The whole network is adjusted in a static mode. The estimated elevations have been time tagged (for the last 20 years) to the first of March of the year of the survey campaign, since it is generally the average date of the main network survey. No error analysis is performed apart from the computation of loop misclosures and *a posteriori* variance factor to indicate the global quality of the observations.

3.4.3 Computation of nodal lines

A group of 3 or 4 nodal lines connecting the main network to a particular nodal BM is called a node (Figure 3.2). Each node is computed by simple extrapolation in time of the adjusted main network BM's at each connection to the date of the survey of each line. A weighted elevation for the junction BM is computed, and then each line is adjusted accordingly.

Interpolation of all of the elevations to the reference date of the main network takes place using the subsidence rate obtained from the previous campaign, for each BM along the line. Residuals (weighted minus observed elevation at the junction BM) larger than 2.8 mm $\sqrt{k}$, where k is the distance in kilometres, lead to the rejection of the particular nodal line. Rejected lines are usually remeasured in the field. There are nine node cases in the main network as shown in Figure 3.1.

3.4.4 Computation of secondary lines

Secondary lines are those of the second order accuracy which are connected either to the main reference network or to the nodal lines. The computation follows a very similar procedure as that for the nodal lines. Time extrapolation of the elevations of the two end BM's to the date of the survey of the line is used to compute a height difference discrepancy for each line. Then, using the same rejection criteria as above, the line is either accepted or rejected for remeasurement. Once accepted, the line is adjusted and interpolated in time back to the reference date.

3.4.5 Additional remarks

In 1984, as part of an automation project, the whole procedure described above was programmed into a PDP 1170 minicomputer. The automation project also included field data collection with the HP41CV calculators, transfer of data and pre-processing using an HP85 microcomputer, and transfer from the HP85 to the PDP1170 for final processing. The computational procedure is still slow and tedious due to the inflexibility of the existing software and obsolescence of the methodology. A total of 3 to 4 weeks is normally needed to process the whole data.

3.5 Economic Aspects

Monitoring has always been performed by precision geodetic levelling techniques as described earlier, which is a slow and costly operation.

The major costs involved in the present inland monitoring scheme arise from the three main sources: field levelling work, bench mark maintenance, and supervisory plus data processing activities. This section is intended to develop approximate relationships to estimate the costs of each one of these activities based on previous experience gained by the author as a manager of the last two campaigns (1986 and 1988). The values are by no means exact since approximate cost rates have been used and minor costs have been neglected for simplicity. The costs of post-processing for the elaboration of final contour maps and monumentation reconstruction or replacement are not included.

3.5.1 Cost of levelling

Performance in geodetic levelling is directly related to the field procedure and existing meteorological conditions. High temperature and humidity generally limit the sight lengths and observation hours. Although a prevalent average temperature of 32°C and 80% humidity is encountered in the Costa Bolivar oil fields, an average daily performance of 6 km has been experienced with survey crews consisting of one surveyor and four non-qualified labour workers. Second order procedures can generally be considered the same as first order but single run. Therefore, the general relationship to estimate the cost of levelling per kilometre for day shifts of 8 hours may be as follows:

$$\text{Cost lev/km} = 1.33 \text{ hr/km } (4C_A + C_B + C_I) \qquad (3.1)$$

where C_A and C_B are the costs of non-qualified and technical labour per hour respectively, and C_I is the cost of instrumentation per hour, which includes vehicle and surveying instrumentation. Assigning values to the above variables in Bolivares (Bs) which is the currency in Venezuela, of $C_A = 150$ Bs/hr, $C_B = 200$ Bs/hr and $C_I = 100$ Bs/hr the cost of one kilometre of levelling would be in the order of 1200 Bs/km, which is equivalent to about 100 Canadian dollars per kilometre using the present exchange rate of 14.50 Bs/US$ applicable to oil industry operations and a ratio of 1.20 Cdn.$/US$.

3.5.2 Cost of maintenance

The maintenance of BM's mentioned here consists basically of minor repairs (e.g. painting) and vegetation trimming for each BM prior to the surveys. An average performance of 12 BM's per day per crew of 4 workers has been maintained over the past years. The following relationship can be used to obtain the maintenance cost/BM:

$$\text{Maintenance cost/BM} = 0.67 \text{ BM/hr } (4C_A + C_I) \qquad (3.2)$$

where C_I now includes the cost of vehicle and working tools per hour.

Using the same approximate values as before with C_I again equal to 100 Bs/hr (since it includes cost of materials), the cost of maintenance per BM is computed to be 469 Bs/BM, equivalent to 39 Cdn.$ per BM.

3.5.3 Cost of supervision and data processing

Costs related to data processing and supervision normally involve the performance of two surveying engineers. One dedicated entirely to supervisory duties, planning, logistics and administration, and the other concerned with daily data logging and processing.

Although the cost of the former would generally be higher, an average daily rate $C_P = 2000$ Bs/day for each could be used. This is equivalent to 166 Cdn.$ per day.

3.5.4 Total estimated cost of one campaign

On the basis of the above figures and considering a total of 1624 BM's, 1469 km of levelling lines and 240 days for supervision and data processing, the total cost of one campaign may be established. Note that this total cost includes neither the costs of post processing for the elaboration of final contour maps and monumentation nor the cost of offshore subsidence surveys.

The total cost may be estimated as follows:

Cost of levelling		
1469 km x 1200 Bs/km		1,762,800 Bs
Cost of Maintenance		
1624 BM's x 469 Bs/BM		761,656 Bs
Cost of supervision and data processing		
240 days x 2000 Bs/day		480,000 Bs
	TOTAL COST	3,004,456 Bs

This is equivalent to 248,644.6 Cdn.$ using the same exchange factors as above. Notice that the major cost arises from levelling.

3.6 Accuracy Evaluation

As already mentioned, the described computational technique has not provided sufficient information and flexibility for a proper assessment of the results. Therefore, an independent evaluation of the actual accuracy of the subsidence monitoring scheme has had to be performed by the author. The data of the last three survey campaigns, which took place in 1984, 1986, and 1988 have been used in the accuracy analysis employing the MINQE technique mentioned below.

3.6.1 Description of survey data

As mentioned earlier, survey data of three previous campaigns was available for the accuracy analysis. The same network geometry was kept during campaigns with the exception of a few BM replacements. Athough most of the 1986 and 1988 data was available, only the first order levelling data from the main levelling network and a section of the second order densification data (shown in Figure 3.2) in the Lagunillas basin was selected for testing. For the 1984 campaign, only the first order levelling data of the main network was available. Total height differences of the levelling lines between main junction BM's were taken for the analysis. Table 3.3 shows a summary of the data.

Table 3.3. Statistical Summary of Survey Data.

Description of Data	1984	1986	1988
First order lines	36	49	49
Second order lines	--	26	26
Total lines	36	75	75
Number of bench marks	26	50	50

3.6.2 Accuracy of levelling surveys

Geodetic levelling is affected by two types of errors — random and systematic. Random errors are always present in the measurements and cannot be eliminated. Systematic errors, however, could be eliminated or minimized by proper field procedures and calibration. A concise review of the characteristics and methods to eliminate most of these errors can be found in the manual for geodetic levelling from the U.S. National Geodetic Survey [NOAA, 1981].

Since 1912 with the introduction of the Lallemand formula, many models to combine the effect of random and systematic errors in levelling have been suggested; the main disagreement being on the interpretation of the systematic effect [Wassef, 1974]. On the other hand, correlations within and between neighbouring lines also exist and has been researched by several authors [e.g., Vanícek and Grafarend, 1980]. Some of the suggested models contain parameters which are empirical and too subjective making their evaluation and application rather unrealistic in the situation at the Costa Bolivar. A more rigorous and practical method, recommended by Chen [1983], is to use variance components estimation techniques to evaluate model parameters from field data. The Minimum Norm Quadratic Estimation (MINQE) described in Chen [1983] has been successfuly used by Chen and Chrzanowski [1985] for estimating error model parameters in levelling networks. The MINQE technique is part of the UNB Generalized Method and was used here by the author in the evaluation of the Venezuelan levelling data. The simple model $\sigma^2_\ell = \sigma^2_i k$ (where k is distance in km and σ_i is the standard deviation per kilometre) was used in MINQE to evaluate the components σ_i corresponding to the first and second order data.

The systematic effects were considered minimal since the levelling lines are rather short (~ 10 km) and the area is mostly flat. The estimated variance components in the form of standard deviations together with their corresponding standard errors for all the campaigns are shown in Table 3.4. Of course, in single variance estimation there is no need for sophisticated variance estimation techniques since the *a posteriori* variance factor estimation will be sufficient. However, to separate the variances corresponding to heterogeneous data, it becomes necessary. In the combination of first and second order data for the 1986 and 1988 campaigns, the same model was used but with σ^2_I and σ^2_{II} representing the variances corresponding to the first and second order data respectively. The estimated standard deviations are also shown in Table 3.4.

It is necessary to point out that neither gravity nor any other corrections were applied to the data. Thus, the misclosures are contaminated by the net effect of neglecting gravity corrections, rod calibration errors, residual refraction, and other error sources affecting the measurements. It has been common practice at the Costa Bolivar not to apply any corrections to the levelling data. Thus, the estimated accuracy reflects the real levelling accuracy used by the Maraven computational method.

Table 3.4. Levelling standard deviations as determined by MINQE

EPOCH	Single parameter estimation σ_I (mm)	Multiple parameter estimation σ_I (mm)	σ_{II} (mm)
1988	2.1 ± 1.4 mm	2.2 ± 1.4 mm	4.2 ± 2.9 mm
1986	1.4 ± 0.9 mm	1.4 ± 0.8 mm	1.9 ± 1.6 mm
1984	2.0 ± 1.3 mm	--	--

The results show that the accuracy of levelling is about half that, expected from the specifications for the first and second order levelling respectively. This can be explained by the failure to apply the necessary corrections and the high daily progress observed in the surveys. As a matter of fact, in the 1984 and 1988 campaigns, the survey was not as closely supervised as in 1986 and the daily survey progress, as seen in the field records, was much faster (up to 8 km/day). In those campaigns sight lengths in the order of 100 m were common during early morning hours.

Since the 1986 campaign was rather an exceptional case compared to most campaigns, it can be concluded that the general accuracy (standard deviations) of levelling surveys in the main levelling network is equivalent to about 2 mm$\sqrt{k}$ for the double run levelling lines and 4 mm$\sqrt{k}$ for the single run densification lines.

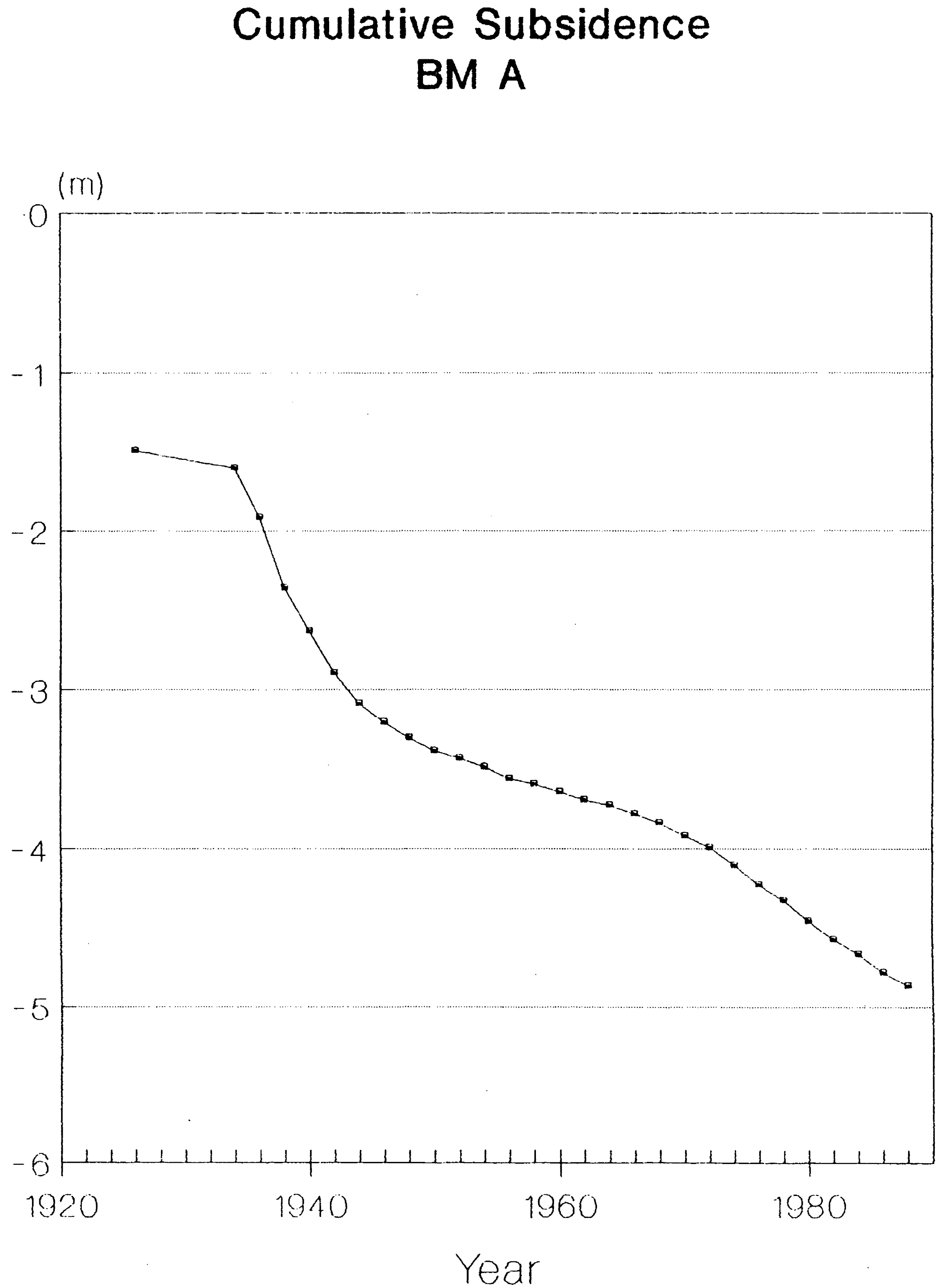

Figure 3.4. Historic Plot of Critical Bench Mark A
(Lagunillas basin)

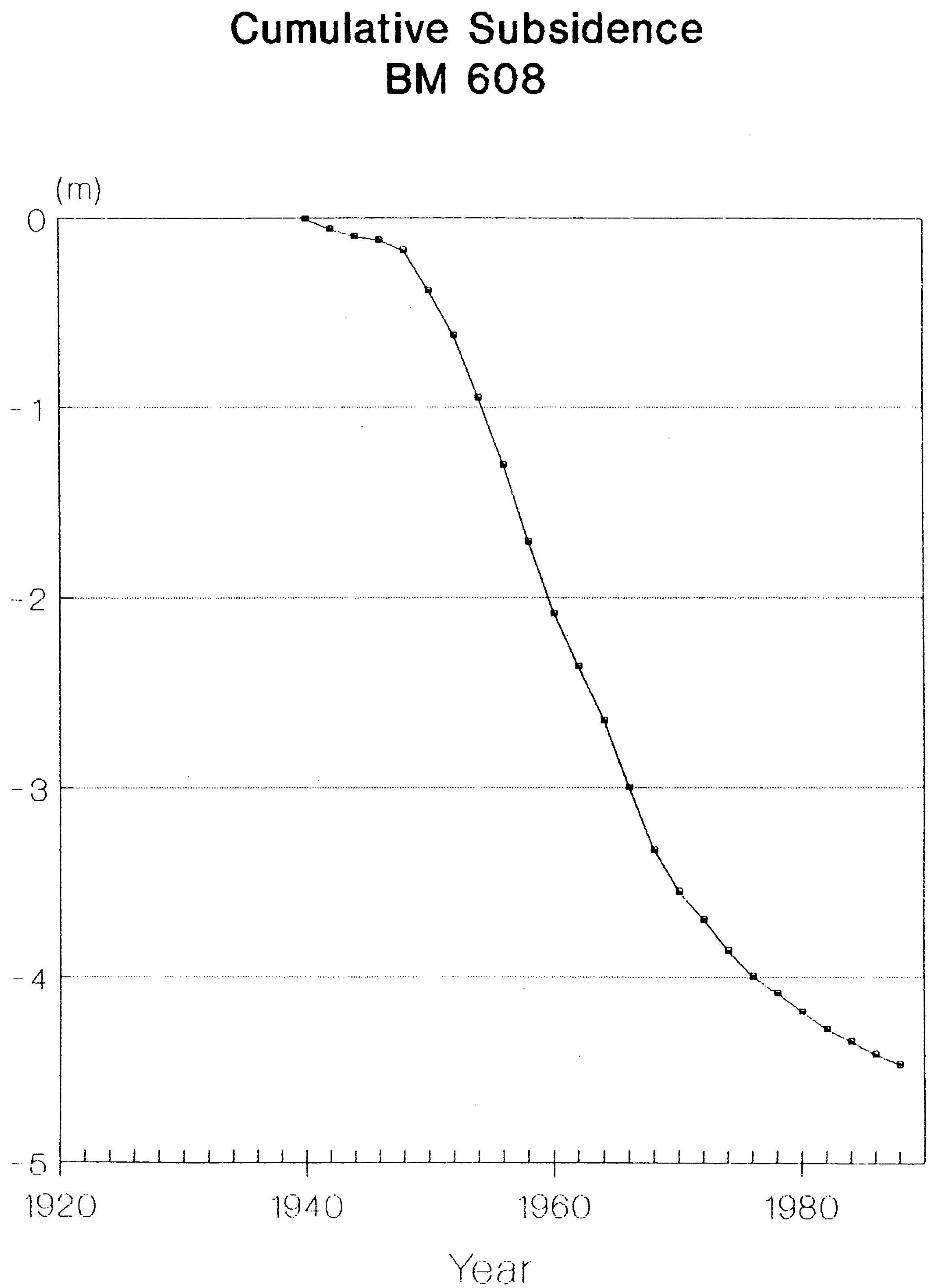

Figure 3.5. Historic Plot of Critical Bench Mark 608
(Bachaquero basin)

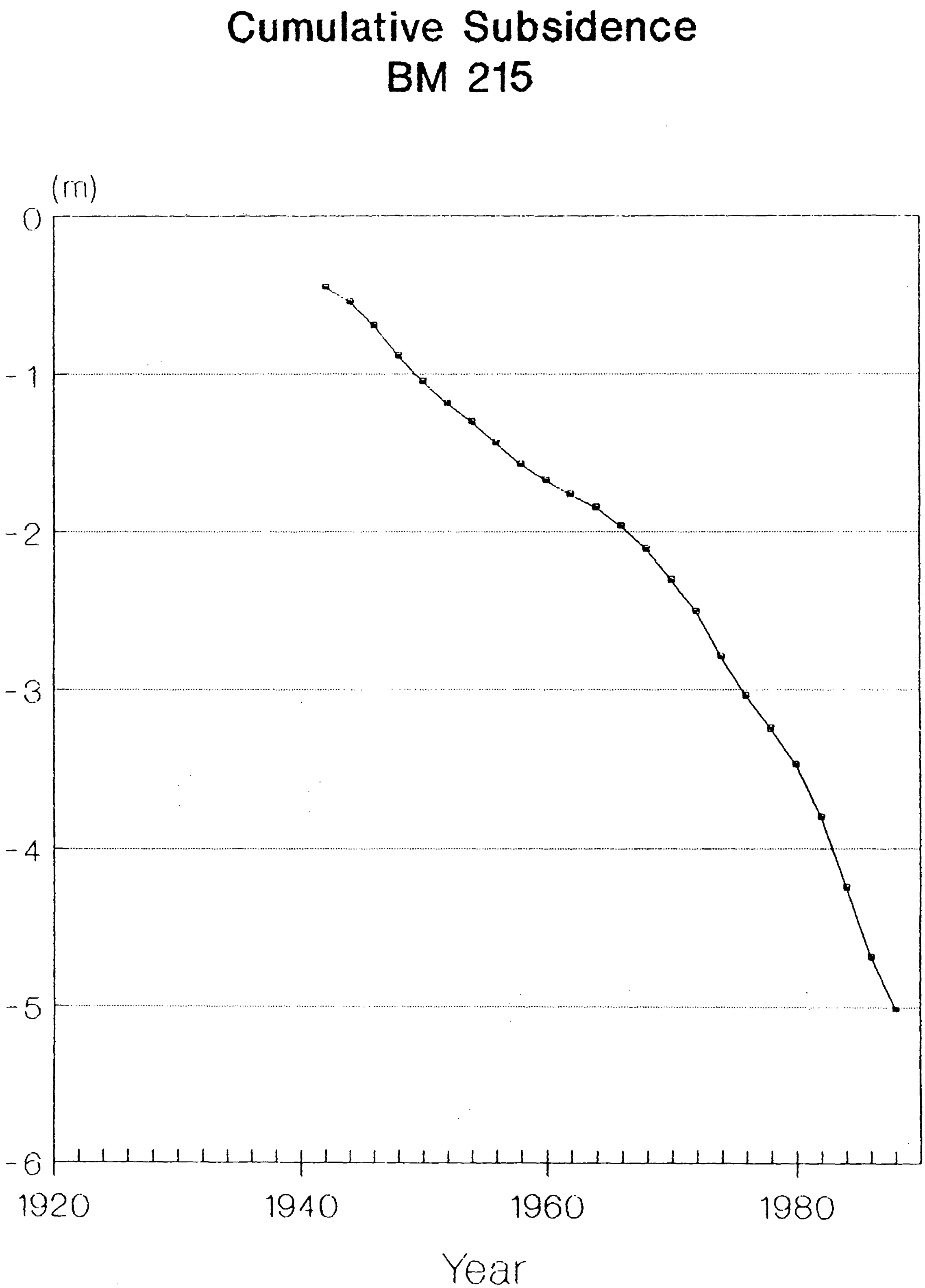

Figure 3.6. Historic Plot of Critical Bench Mark 215
(Lagunillas basin)

3.6.3 Validity of the static network assumption

The dynamic characteristics of the subsidence introduces systematic heterogeneities into the data since the survey is not performed at one instant of time. Although the field surveys are planned in such a way as to minimize this systematic noise, it is necessary to investigate the significance of the static network assumption for the main network adjustment. Although the subsidence in Lagunillas and Bachaquero does not seem to exhibit linear behaviour in time as depicted by historic plots of several BM's located over the major subsidence basins (Figure 3.4, 3.5, 3.6), a short term linear behaviour, up to a decade or so, can be safely considered as valid for the choice of a simple velocity function in a kinematic adjustment of the network. In this investigation the three levelling campaigns were compared using first a separate campaign adjustment in the static mode and then the kinematic modelling approach discussed in Chapter 2.

Parametric adustments with minimal constraints holding Point 1175 DP fixed were performed. The previously estimated variances (from the MINQE method) were used for weighting the observations. The subsidence computed through both methods for the most critical BM's is shown in Table 3.5. No significant difference between both methods is observed, leading to the conclusion that, at the present rate of the subsidence and provided that the surveys in the past were performed simultaneously towards the areas of the maximum subsidence, the assumption of the static adjustment has not introduced significant biases in the subsidence and elevations determination.

3.6.4 Stability of the reference network

A very important aspect of deformation monitoring is the proper assessment of the stability of the reference network. Distorted displacements may lead to erroneous analysis and interpretation of deformations. Over the last few years this topic has been fully investigated. Several methods have been developed within the activity of the FIG committee on the analysis of deformation surveys [Chrzanowski and Chen, 1986]. One of the

Table 3.5. Comparison of Subsidence Determinations.

Bench Mark	Subsidence from Static Adj. (mm) 84/86	86/88	84/88	Subsidence from Kinematic Adj. (mm) 84/86	86/88	84/88
185 DP	+26.4	+28.5	+54.9	+26.3	+28.5	+54.8
1056	-38.4	-45.6	-83.9	-37.9	-45.5	-83.8
329B	-42.6	-34.0	-76.4	-42.5	-34.0	-76.6
-AB	-116.6	-83.9	-200.3	-115.9	-83.9	-199.8
856	-25.5	-31.5	-56.2	-24.3	-31.9	-56.0
846A	-61.6	-44.5	-105.1	-60.1	-45.0	-105.0
639A	-14.8	-14.2	-28.0	-14.0	-14.7	-28.3

methods which is part of the aforementioned UNB Generalized Method is the iterative weighted similarity transformation. The method is meant to yield the "best" relative displacements following an iterative procedure to minimize the first norm of the estimted displacement vector as described in Chen [1983], and Secord [1985]. The method has been applied by the author to analyze the stability of the reference BM's in the monitoring levelling network.

The reference BM's are located along the aforementioned "datum lines" which have been used to constrain the adjustment. The "best" displacements and their significance as obtained from the application of the weighted similarity transformation to different epoch combinations is shown in Table 3.6. Only the extreme BM's on each datum line are listed. A non-iterative procedure especially for levelling networks presented in Chen et al. [1988] has been followed by the author using his own software.

Table 3.6. "Best" Weighted Displacements for Reference Bench Marks

BM	86-88 [mm]	Significance level	84-88 [mm]	Significance level	84-86 [mm]	Significance level
1329 DP	8.9	0.79	6.4	0.40	3.0	0.22
1002 DP	-3.5	0.85	4.1	0.34	13.1	0.86
1324 DP	9.6	0.61	9.4	0.60	4.7	0.29
1326 DP	12.4	0.70	11.2	0.64	3.7	0.22
1175 DP	7.8	0.58	14.8	0.88	12.6	0.73
185 DP	36.3	> 0.99	69.7	> 0.99	39.0	> 0.99

The results show a significant uplift of BM 185 DP which is responsible for the apparent subsidence of reference BM 1175 DP of the Lagunillas datum line when using the earlier described datum lines computation method.

A shift of -7 mm to BM 1175 DP was introduced in the original calculations at Maraven for both the 84-86 and 86-88 comparisons, as revealed by the different elevations estimated for this BM in 1984, 1986 and 1988 (see Table 3.2). However, the author's results which are shown in Table 3.6 do not indicate any significant movement of that BM. The same applies to BM 1329DP which was shown in Table 3.2 as having a movement of +9 mm between campaigns 86-88. The author's calculations show again that its movement is statistically insignificant (see Table 3.6).

It can be concluded that, although the most distant BM's inland which correspond to the ends of the three datum lines seem to be stable, the Maraven computational method introduces systematic shifts to some BM's which are actually stable. This can lead to false elevations and misleading subsidence results. Fortunately, since the Maraven method uses an overconstrained adjustment (section 3.4.2) the smoothing effect that takes place decreases the total effect of the falsely introduced movements. The worst results are expected when the same or similar shifts are introduced at least at two of the reference BM's.

3.6.5 Final accuracy of elevations in single campaigns

Since the Maraven computational scheme does not provide stochastic information to assess the accuracy of the results, an equivalent static parametric adjustment for each campaign was performed. The previously estimated variance components were used and the extreme BM's on each "datum line" (1175 DP, 1329 DP and 1326 DP) were held fixed.

Table 3.7 shows a comparison between the elevation values for the same campaigns obtained at Maraven and the new values obtained by the author. Obviously, the differences show the systematic effect of the datum shifts in the Maraven calculations as discussed in the previous section. The systematic trend is equivalent to -7 mm for the 1986 and 1988 campaigns and to about -3 mm for the 1984 campaign.

A maximum standard error of 7.7 mm which is equivalent to 15 mm at a 95% confidence level was obtained for the adjusted elevations (see Table 3.8). This is, of course, datum dependent as the elevation errors increase with the distance from the constrained points.

It can be concluded that the total uncertainty in the absolute elevations as obtained from the Maraven computational method may reach 15 to 20 mm at the 95% confidence level.

3.6.6 Accuracy of the subsidence determination

The accuracy of the subsidence determination was derived from the separate campaign adjustments. The results of 1984 and 1988 give a maximum standard error of 10.9 mm which is equivalent to 22 mm for absolute subsidence at the 95% confidence level

Table 3.7. Comparison of Elevations.

	Elevations by Maraven			Elevations by Author			Discrepencies (Maraven-Author)		
Bench Mark	Elev. 84 [m]	Elev. 86 [m]	Elev. 88 [m])	Elev. 84 [m]	Elev. 86 [m]	Elev. 88 [m]	Δ[mm] 84	Δ[mm] 86	Δ[mm] 88
1002DP	59.510	59.511	59.507	59.506	59.513	59.501	+4	-2	+6
185DP	31.135	31.155	31.176	31.134	31.165	31.192	+1	-10	-16
744	48.941	48.938	48.936	48.937	48.941	48.932	+4	-3	+4
734	32.704	32.736	32.702	32.703	32.740	32.702	+1	-4	0
1056	31.221	31.183	31.140	31.221	31.188	31.141	0	-5	-1
411	25.056	25.069	25.063	25.054	25.074	25.063	+2	-5	0
387	4.282	4.270	4.270	4.282	4.275	4.271	0	-5	-1
329B	0.817	0.775	0.741	0.817	0.780	0.745	0	-5	-4
1390A	1.314	1.300	1.291	1.315	1.307	1.297	-1	-7	-6
1703	14.897	14.981	14.877	14.899	14.898	14.885	-2	-7	-8
M	17.420	17.414	17.405	17.424	17.423	17.417	-4	-9	-12
184A	27.517	27.519	27.515	27.516	27.529	27.529	+1	-10	-14
1791	10.521	10.511	10.501	10.525	10.519	10.513	-4	-8	-12
-AB	1.095	0.980	0.892	1.098	0.987	0.901	-3	-7	-9
117	2.178	2.168	2.164	2.181	2.174	2.172	-3	-6	-8
46A	5.288	5.274	5.270	5.291	5.280	5.278	-3	-6	-8
856	14.651	14.629	14.594	14.654	14.635	14.601	-3	-6	-7
1725	61.511	61.503	61.505	61.512	61.508	61.511	-1	-5	-6
846A	10.006	9.948	9.900	10.008	9.954	9.906	-2	-6	-6
546A	3.591	3.586	3.573	3.594	3.592	3.581	-3	-6	-8
639A	8.229	8.220	8.202	8.231	8.224	8.207	-2	-4	-5
691B	0.752	0.730	0.721	0.755	0.734	0.726	-3	-4	-5
1324DP	40.365	40.366	40.364	40.367	40.367	40.365	-2	-1	-1

Table 3.8. Summary of Standard Deviations (σ)for Author Elevations.

Bench Mark	σ[mm] 84	σ[mm] 86	σ[mm] 88
1002DP	4.5	3.2	4.5
185DP	4.2	2.9	4.2
744	4.7	3.3	4.7
734	5.4	3.8	5.4
1056	5.7	4.0	5.7
411	6.0	4.2	6.0
387	7.4	5.2	7.4
329B	6.6	4.6	6.6
1390A	6.2	4.4	6.2
1703	5.7	4.0	5.7
M	4.5	3.2	4.5
184A	4.2	2.9	4.2
1791	4.9	3.4	4.9
-AB	6.0	4.2	6.0
117	6.0	4.2	6.0
46A	5.9	4.1	5.9
856	5.9	4.2	5.9
1725	5.3	3.7	5.3
846A	6.1	4.3	6.1
546A	6.1	4.3	6.1
639A	6.1	4.3	6.1
691B	7.7	5.4	7.7
1324DP	3.3	2.3	3.3

Table 3.9. Comparison of Subsidence Results.

	Maraven Values [mm]		Author Values [mm]		Discrepancies (Maraven-Author)	
Bench Mark	84-86	86-88	84-86	86-88	Δ[mm] 84-86	Δ[mm] 86-88
1002DP	+1	-4	-7	-12	+8	+8
185DP	+20	+21	+31	+27	-11	-6
744	-3	-2	+4	-9	-7	+7
734	+32	-34	+37	-38	-5	+4
1056	-38	-43	-33	-47	-5	+4
411	+13	-6	+24	-11	-11	+5
387	-12	0	-7	-4	-5	+4
329	-42	-34	-37	-35	-5	+1
1390	-14	-9	-8	-10	-6	+1
1703	-6	-14	-1	-13	-5	-1
M	-6	-9	-1	-6	-5	-3
184A	+2	-4	-13	0	+15	-4
1791	-10	-10	-6	-6	-4	-4
-AB	-115	-88	-111	-86	-4	-2
117	-10	-4	-7	-2	-3	-2
46A	-14	-4	-11	-2	-2	-2
856	-21	-36	-19	-34	-2	-2
1725	-8	+2	-4	+3	-4	-1
846A	-58	-48	-54	-48	-4	0
546A	-5	-13	-2	-11	-3	-2
639A	-9	-18	-7	-17	-2	-1
691B	-22	-9	-21	-8	-1	-1
1324DP	1	-2	0	-2	+1	0

Table 3.10. Summary of Standard Deviations for the Author Computed Subsidence Values.

Bench Mark	σ[mm] 84-86	σ[mm] 84-88	σ[mm] 86-88
1002DP	5.5	6.4	5.5
185DP	5.1	5.9	5.1
744	5.7	6.6	5.7
734	6.6	7.6	6.6
1056	6.9	8.1	6.9
411	7.3	8.5	7.3
387	9.0	10.5	9.0
329B	8.0	9.3	8.0
1390A	7.6	8.8	7.6
1703	7.0	8.1	7.0
M	5.5	6.4	5.5
184A	5.1	5.9	5.1
1791	6.0	6.9	6.0
-AB	7.3	8.5	7.3
117	7.3	8.5	7.3
46A	7.2	8.3	7.2
856	7.2	8.3	7.2
1725	6.5	7.5	6.5
846A	7.5	8.6	7.5
546A	7.5	8.6	7.5
639A	7.5	8.6	7.5
691B	9.4	10.9	9.4
1324DP	4.0	4.7	4.0

(see Table 3.10). Table 3.9 shows again the influence of the aforementioned systematic effect in the order of -4 mm between the 1984-1986 campaigns when the author's computations are compared with the Maraven data.

For the 1986-1988 campaigns the effect varies from +8 mm near bench mark 1329DP to about -2 mm on the points near Bachaquero. This may be due to the positive shift introduced by the Maraven method in the fixed point 1329 DP (Table 3.2) for the 1988 campaign computation. In conclusion, the total uncertainty in the Maraven calculated subsidence estimates reaches 20 to 30 mm at the 95% confidence level.

3.6.7 Final accuracy evaluation of the subsidence using the UNB Generalized Method

One further step into the analysis of the subsidence computation arises from the application of the UNB Generalized Method through the least squares fitting of a selected deformation model to the observed displacements. In Section 3.6.4, the weighted similarity transformation was used to determine the "best" displacements out of the original datum dependent displacements estimated from two separate static adjustments showing only the reference BM's. For further analysis, the estimated "best" displacements between the 1984 and 1988 campaigns are listed in Table 3.11. On the basis of the observed displacements and their associated confidence levels, single point displacements and a stable block of reference points could be identified.

As discussed in Chapter 2, points showing significant movements could be modelled as separate individual blocks and stable points (i.e. points that do not show significant movement) could be modelled together as a stable block. Once the deformation trend is identified the original displacements together with their variance-covariance matrix are used in the model fitting process.

Rigid body displacement models similar to equation (2.7) may be written as:

$w_j(x, y) = 0$ and

$w_k(x, y) = a_k$

where j represents the block of all stable points and k represents non-stable points treated as separate rigid blocks with individual rigid body displacement a_k with respect to the stable block. Thus the general model could be expressed as

$$\mathbf{d} + \delta = \mathbf{Bc}$$

where **d** is the vector of subsidence values estimated from the minimally constrained adjustments of both epochs;

c is the vector of unknown parameters;

B is the design matrix of the deformation model formed by rows of zeroes for the stable points and unit elements in the columns corresponding to the parameters of the unstable points; and

δ is the vector of residuals.

A comprehensive explanation of the estimation of **c** may be encountered in Chen et al. [1988]. The estimated parameters between the 1988 and 1984 campaigns together with their corresponding standard deviations and significance levels for some selected BM's are shown in Table 3.12. The global test to verify the appropriateness of the above model at a 0.95 confidence level passes, i.e., the inequality $\hat{\sigma}^2_0 / \hat{\sigma}^{*2}_0 < F_{(df1,\, df2;\, \alpha)}$ holds true, where $\hat{\sigma}^2_0$ is the resulting 'a posteriori' variance factor from the model fitting solution, $\hat{\sigma}^{*2}_0$ is the pooled variance factor [Chen et. al. 1988] and df_1 and df_2 are the corresponding degrees of freedom.

There is a clear indication, when comparing to Table 3.10, that the accuracy of the subsidence estimation can be improved by implementing the Generalized Method technique through further modelling the subsidence. The previous subsidence values and their accuracies would then be significantly improved. A maximum standard deviation error of 5.5 mm is shown in Table 3.12. The application of this methodology will also remove most systematic errors arising from the aforementioned shifts.

Table 3.11. "Best" Weighted Displacements (88-84)

BM	84-88 [mm]	Significance level	BM	84-88 [mm]	significance level
1175DP	14.8	0.88	184A	25.7	> 0.99
1329DP	6.4	0.40	1791	0.0	0.0
1326DP	11.2	0.64	-AB	-185.5	> 0.99
1002DP	4.1	0.34	117	2.1	0.21
185DP	69.7	> 0.99	46A	-0.9	0.09
744	4.1	0.35	856	-41.5	> 0.99
734	8.7	0.68	1725	10.3	0.73
1056	-69.1	> 0.99	846A	-90.3	> 0.99
411	18.0	0.94	546A	-1.4	> 0.13
387	-0.4	0.03	580A*	-769.3	> 0.99
329B	-61.6	> 0.99	639A	-13.3	0.79
1390A	-7.2	0.64	691B	-16.9	0.83
1703	-3.0	0.34	1324DP	9.4	0.60
M	5.1	0.87			

*reconstructed in 1986

Table 3.12. Estimated Model Parameters (88-84).

Bench Mark	a_k (mm)	σ_{a_k} (mm)	Significance Level	Remarks
185 DP	62.6	3.3	> 0.99	Global test on
1056 DP	-74.6	3.9	> 0.99	Deformation
411	10.5	3.7	.99	Model passes
329 B	-58.6	4.8	> 0.99	
184 A	-18.9	3.1	> 0.99	1.08 < 2.17
-AB	-183.5	5.5	> 0.99	($F_{17,20;0.95}$)
856	-42.2	5.3	> 0.99	
846A	-87.7	5.0	> 0.99	
580A*	-757.0	4.9	> 0.99	

* re-constructed in 1986

4. ACCURACY OF GPS DERIVED HEIGHT DIFFERENCES

There are two basic factors affecting the accuracy of GPS observations. These are the range error and the geometry of the satellites. According to Mertikas et al. [1986], the range error is expressed by the User Equivalent Range Error (UERE) and the geometry by the Geometric Dilution Of Precision (GDOP), which for the vertical direction is referred to as VDOP. The UERE represents the overall effect of all the observational errors arising from orbit uncertainties, signal propagation errors and receiver related errors. The effect of the UERE (σ_ρ) and the VDOP may be combined to yield the total error in a derived height (σ_h) through the simple relationship

$$\sigma_h = \text{VDOP}\ \sigma_\rho\,. \tag{4.1}$$

Thus, in order to analyse the accuracy of the GPS derived height difference observations, a brief discussion of the geometry and the most significant observational errors is given in this chapter. Results from three test GPS campaigns performed in Venezuela between April 1987 and April 1988 are also presented. Finally, brief discussions on accuracy expectations and cost evaluation for the future Costa Bolivar GPS campaigns are also included.

Differential GPS positioning and successful ambiguity resolution has been assumed throughout the discussion.

4.1 Satellite Geometry

The effect of satellite geometry is generally represented by the GDOP which is a scalar measure of the overall geometrical strength of an immediate point positioning solution. Although the main concern herein is relative positioning, and since the baselines are short, the average GDOP within the observation period has been assumed to give still a valid measure of the geometrical strength of the solution and will be used under this context throughout the thesis. Wells et al. [1986] point out that for long base lines in the order of thousands of kilometres this does not hold. The GDOP is computed from the square root of the trace of the cofactor matrix obtained in a position fix using pseudoranges to at least four observed satellites. This is equivalent to a distance resection solution with unit weights. Thus,

$$\mathrm{GDOP} = \sqrt{q_\phi^2 + q_\lambda^2 + q_h^2 + q_t^2} \qquad (4.2)$$

where $q^2{}_\phi$, $q^2{}_\lambda$, $q^2{}_h$ and $q^2{}_t$ represent the co-factors of the latitude, longitude, height and time coordinates respectively, obtained from the cofactor matrix of the estimated position parameters. The value of the GDOP varies with time and user location since it depends on the movement of the satellites and satellite coverage.

The selection of different components in (4.2) leads to other geometrical scalars such as PDOP, HDOP, or VDOP, for three dimensional, horizontal and vertical positioning respectively. For instance, the VDOP is obtained from

$$\mathrm{VDOP} = q_h \ . \qquad (4.3)$$

Up to the present time (January 1989) with the available prototype constellation, the geometry of the satellites has been rather poor in some parts of the world. VDOP values in the order of 4.5 to 5.0, which for high accuracy requirements may be considered as large, were common in the Venezuelan GPS test campaigns to be discussed later.

For the future 24 satellite constellation, significant improvements in VDOP values are expected, especially near the equator where the satellite distribution will be the most uniform. Santerre [1988] shows that the best satellite coverage will be obtained at low latitudes. VDOP values smaller than 3 may be expected [Milliken and Zoller, 1980].

4.2 Observational Errors

4.2.1 Orbit related errors

The orbit related errors are induced by inaccuracies in the measured or predicted orbit of the satellites. The uncertainty in the broadcast ephemeris is considered to be in the order of 20 to 25 m and for the precise ephemeris, as provided by the U.S. Department of Defense (DoD), in the order of 5 to 10 m [Beutler et al., 1986].

The effect is reduced by relative positioning and can be improved by more accurate ephemeris models among other techniques. There are three general uncertainties related to the orbit -- the radial, the out of plane, and the along track biases. According to Beutler et al. [1987b], the along track biases affect more significantly the height components than the radial biases. The error introduced in the height difference ($e_{\Delta h}$) is said to be equal to

$$e_{\Delta h} = \cos\,(Az_s - Az_b)\,\frac{\Delta s}{\rho}\,b \quad , \qquad (4.4)$$

where Δs is the magnitude of the along track bias,

Az_b is the azimuth of the baseline,

Az_s is the azimuth of the orbital plane of a particular satellite being tracked,

ρ is the range to the satellite, and

b is the length of the baseline.

The maximum error is expected when the baseline orientation coincides with the orbital plane orientation. The error is proportional to length of the baseline. A maximum scale error of 1 to 2 ppm is normally expected when using the broadcast ephemerides. For precise work, the use of precise ephemerides will be more appropriate.

4.2.2 Tropospheric effect

The tropospheric effect is probably the major limitation of GPS in deformation monitoring applications, especially in vertical deformation studies. The effect consists of a delay in the satellite signal as it propagates through the innermost 80 km of the atmosphere. The total refractive effect of the troposphere can be separated into two main effects — the effect of the dry and the wet components. The dry component is responsible for 90% of the total refractivity and can be modelled from surface meterological data with an accuracy of about 1% [Hopfield, 1969], which is equivalent to a range error of about 1 to 6 cm. The wet component is responsible for the remaining 10% but may be modelled to about 10% to 20%, due to the variable water vapor distribution in space and time [Lachapelle et al. 1988]. The effect is reduced in relative positioning particularly for short baselines, provided similar conditions prevail at both ends of the line. According to Beutler et al. [1987b], the effect has a large influence on the height component with an amplification factor of 1/cos z (where z is the maximum zenith angle to the satellite) with respect to the range bias, the effect becoming larger at low elevation angles. Using a simulated continuous satellite distribution, Geiger [1988] estimated an average amplification factor of 3.

Most of the problems encountered with tropospheric modelling are due to inaccuracies in the standard meteorological equipment and local microclimate effects which do not reflect the upper atmospheric conditions at each station. As a result, biased corrections may be expected. To illustrate this further, a table of zenith range errors arising from errors in metereological data using Hopfield's model has been taken from Chrzanowski et al. [1988] and is shown in Table 4.1.

Table 4.1. The zenith range error due to errors in meteorological data (taken from Chrzanowski et al. 1988).

Temperature (T(°C)	Pressure p(mb)	Humidity H(%)	dρ/dp [mm/mb]	dρ/dT [mm/°C]	dρ/dH [mm/%]
0	1000	50	2.3	2.3	0.8
15	1000	50	2.3	5.0	1.7
30	1000	50	2.3	9.8	3.8
0	1000	100	2.3	4.5	0.8
15	1000	100	2.3	9.9	1.7
30	1000	100	2.3	19.6	3.8

Notice the large range errors introduced by the errors in temperature and humidity under very hot and humid conditions. This is an indication of the sensitivity of the solution to the tropospheric effect in tropical climates.

To reduce this effect in small networks, it is common not to use the surface metereological data at each receiver site directly but to average the data and use a local atmosphere model for each session or for the whole campaign. This may be valid for networks with small height differences but for mountainous areas it may yield biased results [Gurtner et al., 1987].

In precise applications the use of balloon or helicopter data collected above each GPS site may be utilized. Pedroza [1988] gives details on tests conducted with this technique during the third GPS campaign at the Costa Bolivar oil fields using instrumentation designed and constructed at the University of New Brunswick. To determine the water vapour pressure in the signal path water vapour radiometry (WVR) has been used, but presently the instrumentation is very expensive and difficult to handle and calibrate [Lachapelle et al., 1988]. Another alternative to tropospheric modelling is to estimate a tropospheric scale factor directly in the adjustment [Santerre, 1988] but it appears to be highly correlated with the height component.

More research on the influence of the troposphere is needed especially at low latitudes where the tropospheric effect is more critical due to the high relative humidity and high temperatures usually encountered.

4.2.3 Ionospheric effect

The ionospheric effect consists of the propagation delay of the satellite signals due to interaction with the charged ions present in the upper atmosphere (from about 80 to 1000 km altitude). The relative effect (Δe) corresponds to a scale error which is a function of the total electron content (TEC), frequency of the carrier and base line length (b). Beutler et al. [1987b] present a formula to quantify the effect on the L1 carrier as:

$$\frac{\Delta e}{b} = -0.7 \times 10^{-17} \, \mathrm{TEC}. \qquad (4.5)$$

The TEC depends directly on solar flux. Thus, it varies with respect to time of the year, time of the day, latitude and direction to the satellite. The highest TEC distributions are found near the equator and in the Auroral regions. Beutler et al. [1987b] estimated values with the L1 carrier varying between 0.35 ppm at night to about 3.5 ppm in the early local afternoon hours for a user location at ± 20° latitude. The effect is corrected accurately using dual frequency observations but a remainder may be left in areas or at times of high TEC [Wells, et al., 1986]. The dual frequency correction increases the noise level of the observations by a factor of 3.3 [Kleusberg, 1986]. In single frequency observations empirical formulae may be used but the level of accuracy of the correction is rather poor (50% - 75%). When the baselines are short the effect is expected to cancel by the differencing, if one assumes that the signal propagates through a homogeneous ionospheric layer. However, this may not generally be the case since local irregularities may affect the signals to each receiver differently. Thus, the integer nature of the differenced ambiguities is corrupted and the data processing becomes difficult.

Another critical effect of ionospheric irregularities, specifically the so-called ionospheric scintillations [Lachapelle et al. 1988] is the fading of the signals. This may cause loss of phase lock and a large number of cycle slips in the data. Lachapelle et al. [1988] point out that multiplexing and sequential channel receivers may be more affected than multichannel receivers due to the less favourable signal to noise ratio in these receivers. These irregularities increase with higher TEC. An 11 year solar flux cycle showed a minimum by the middle of 1986 and is expected to reach a maximum by 1991, so that these problems will become more common in the near future. The use of dual frequency multichannel receivers will be definitely necessary in future high precision GPS surveys in order to correct for the expected larger ionospheric effect, and to collect data of better quality from all possible satellites without loss of geometric strength in the solution.

4.2.4 Carrier signal multipath and antenna imaging

This effect may be considered also as a propagation error since it arises from the interference of reflected signals with the direct signals from the satellites at the receiver antenna. Its effect cannot be quantified since it depends on the location of the reflective environment. It is of cyclic nature so it tends to randomize with longer observation periods depending on the antenna-reflector distance, satellite elevation, orientation of the reflecting object [Tranquilla, 1988] and satellite distribution. It is also frequency dependent. In severe cases, it gives rise to cycle slips. The effect may be minimized by proper antenna design, longer observation periods, satellite selection, and by avoiding reflecting surfaces near the antenna sites. Absorbing sheets or large ground planes to screen the reflected signals may also be used but with the risk of affecting the phase centre characteristics of the antenna [Wells and Tranquilla, 1986].

Another effect also caused by nearby reflecting objects is the problem of antenna imaging. An image of the antenna is created in the reflector and a combined radiation pattern of the real antenna and its image (or images) is formed with seriously degraded phase front characteristics. This causes rapid phase centre variations with observation angle [Tranquilla, 1986]. A similar remedy as for multipath may be used.

4.2.5 Antenna phase center variation

This error is very much dependent on antenna design and direction of the incoming signal. Its effect can be corrected by calibration. Examples of antenna calibrations are given by Sims [1985] and Tranquilla [1988]. The calibration is performed by mapping the antenna phase pattern through a range of azimuth and elevation angles. Typical phase centre variations of 2-10 cm depending on antenna type have been reported by Wells and Tranquilla [1986]. Geiger [1988] estimates an error in absolute height of 3 cm independent of direction to the incoming signals for crossed dipole antenna types.

For monitoring applications the antenna must be calibrated because this effect may introduce large biases in the estimated positions especially if different antenna types are used in different campaigns.

4.2.6 Other error sources

Additional error sources such as receiver and satellite clock errors are eliminated to a great extent by the linear combination of observations (single, double and triple differences). Errors due to receiver noise are considered, generally, to be in the order of 1% of the signal wavelength [Wells ed. 1986]. Therefore, for the case of carrier phase observations with an average wavelength of $\lambda = 20$ cm. This error is equivalent to about 2 mm.

Errors introduced during the data processing may also arise when the station used as fixed in the data processing is shifted from its true geocentric position. The effect according to Eckels [1987] is equivalent to that induced by orbit uncertainties. Chrzanowski et. al. [1988] report changes in height differences larger than one decimetre when comparing the results obtained from a simulated shift of 100 m in the east-west direction, with a similar shift in the north-south direction, when processing a 12.1 km baseline (azimuth 45°-27°) from the Costa Bolivar campaigns.

The successful application of GPS in deformation surveys will depend to a great extent on the consideration given to some of the aspects discussed at the design and planning stage. A realistic estimate of the overall accuracy is however difficult to make unless real observations under different conditions are performed and analysed. This was attempted at the Costa Bolivar oil fields and is discussed in the next section.

4.3 Costa Bolivar GPS Campaigns

As already mentioned, three campaigns using GPS and levelling simultaneously have been performed in the Costa Bolivar oil fields. The first two campaigns of April 1987 and October 1987, considered as test campaigns, were conducted in the area of Tia Juana-Lagunillas using only 1/3 of the main levelling network, and the third campaign of

April 1988 was treated as the implementation of GPS to replace parts of the primary levelling net previously discussed.

All campaigns were performed using four Wild-Magnavox WM101 single frequency receivers rented from Usher Canada Ltd. The observations for the first and third campaigns were conducted during the night time with a few early morning sessions, and for the second campaign at around local noon. The length of the observation periods at each site were approximately one hour for the first campaign and two and a half hours for the second and third campaigns. Surface metereological readings were taken at half hour intervals and the antennas were oriented in the same direction for all campaigns. The data was processed by Usher Canada Ltd., using single or individual baseline mode of calculation with the program POPS. According to Leeman [1988], the second campaign data was the most troublesome to process since the estimated cycle ambiguities deviated significantly from integer values. This required a major processing effort complemented by experience, which at the same time made the results somewhat subjective. The first campaign also presented serious difficulties in the processing due to similar problems as above and limitations with the amount of data collected per baseline. It is believed that network mode processing of all sessions per campaign should eventually yield better results, mainly due to the improved redundancy which will ensure higher reliability in the solution of the ambiguities.

A summary of the GPS and levelling height differences for each campaign are shown in Appendix I. A total of 9, 13, and 37 baselines were observed during the first, second and third campaigns respectively. The loop misclosures and observed baselines are shown in Figures 4.1, 4.2 and 4.3.

4.3.1 Accuracy evaluation and summary of results

In order to analyse the internal accuracy of the GPS derived height differences the aforementioned MINQE technique was applied by Chrzanowski et al. [1988]. The third

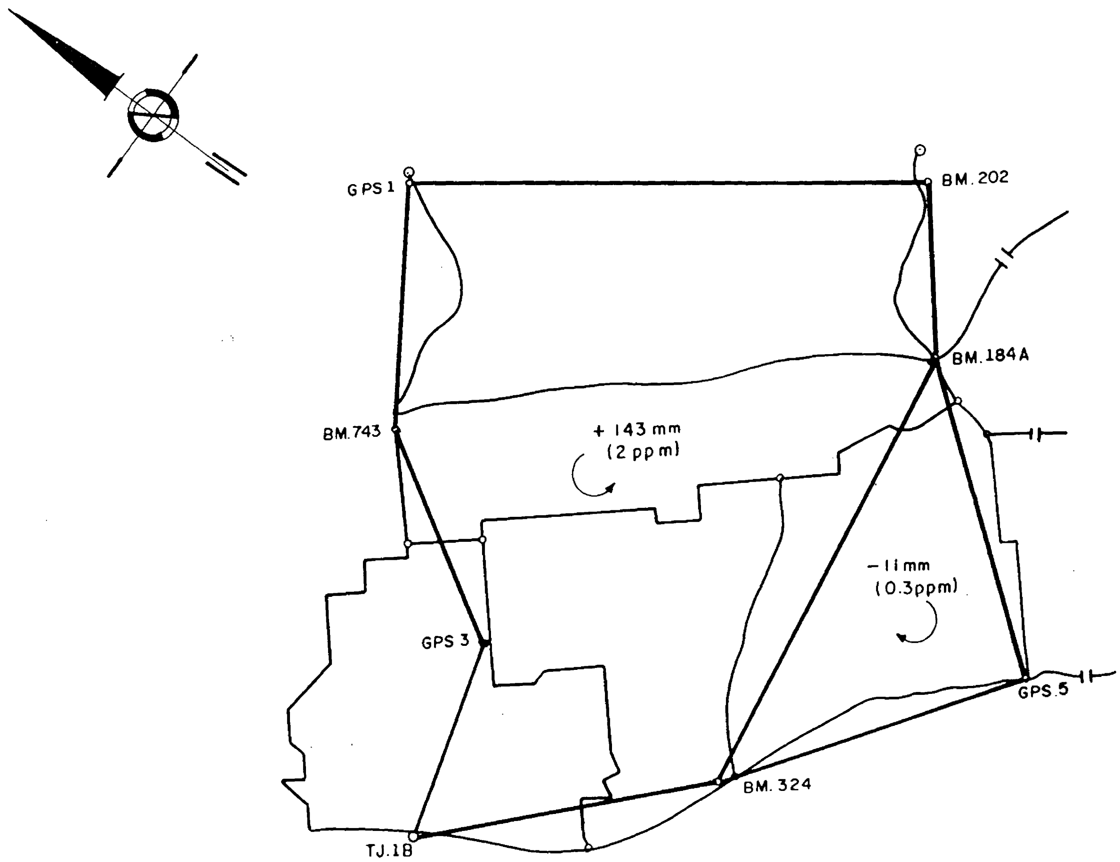

Figure 4.1. GPS Misclosures Campaign No. 1

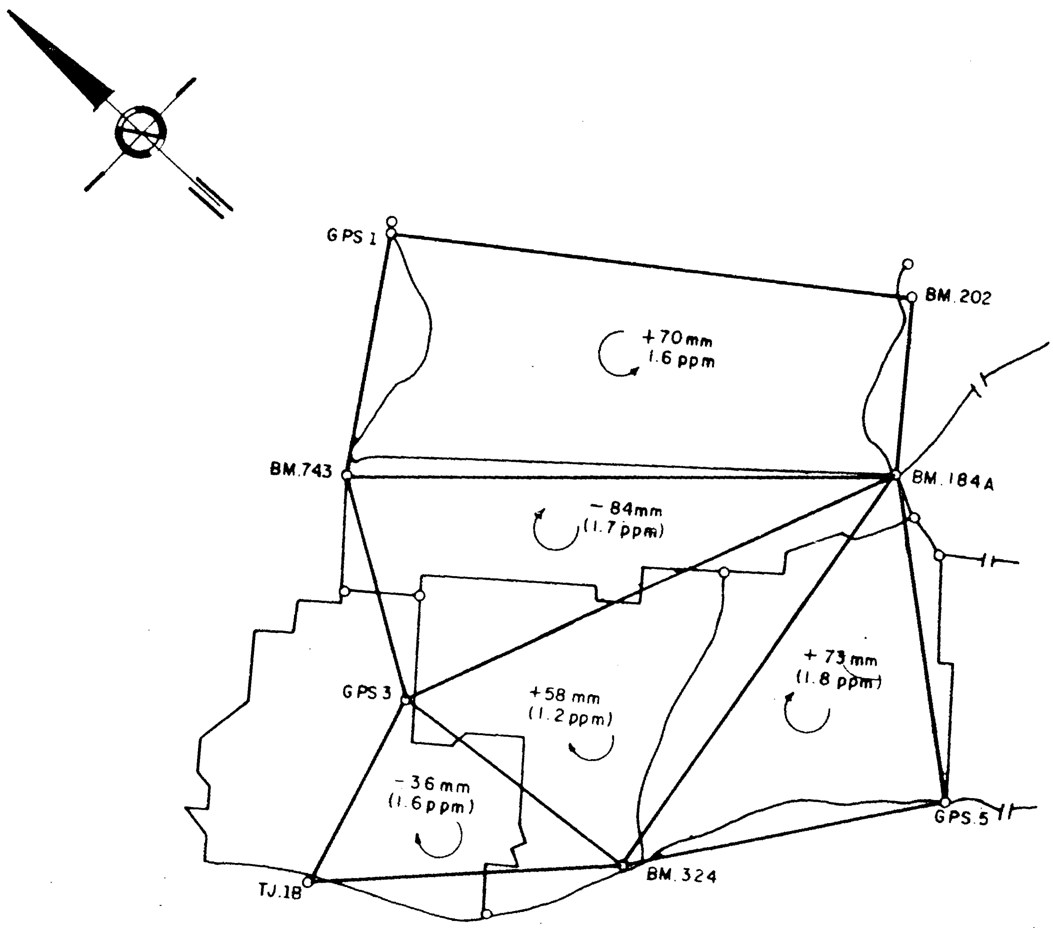

Figure 4.2. GPS Misclosures Campaign No. 2.

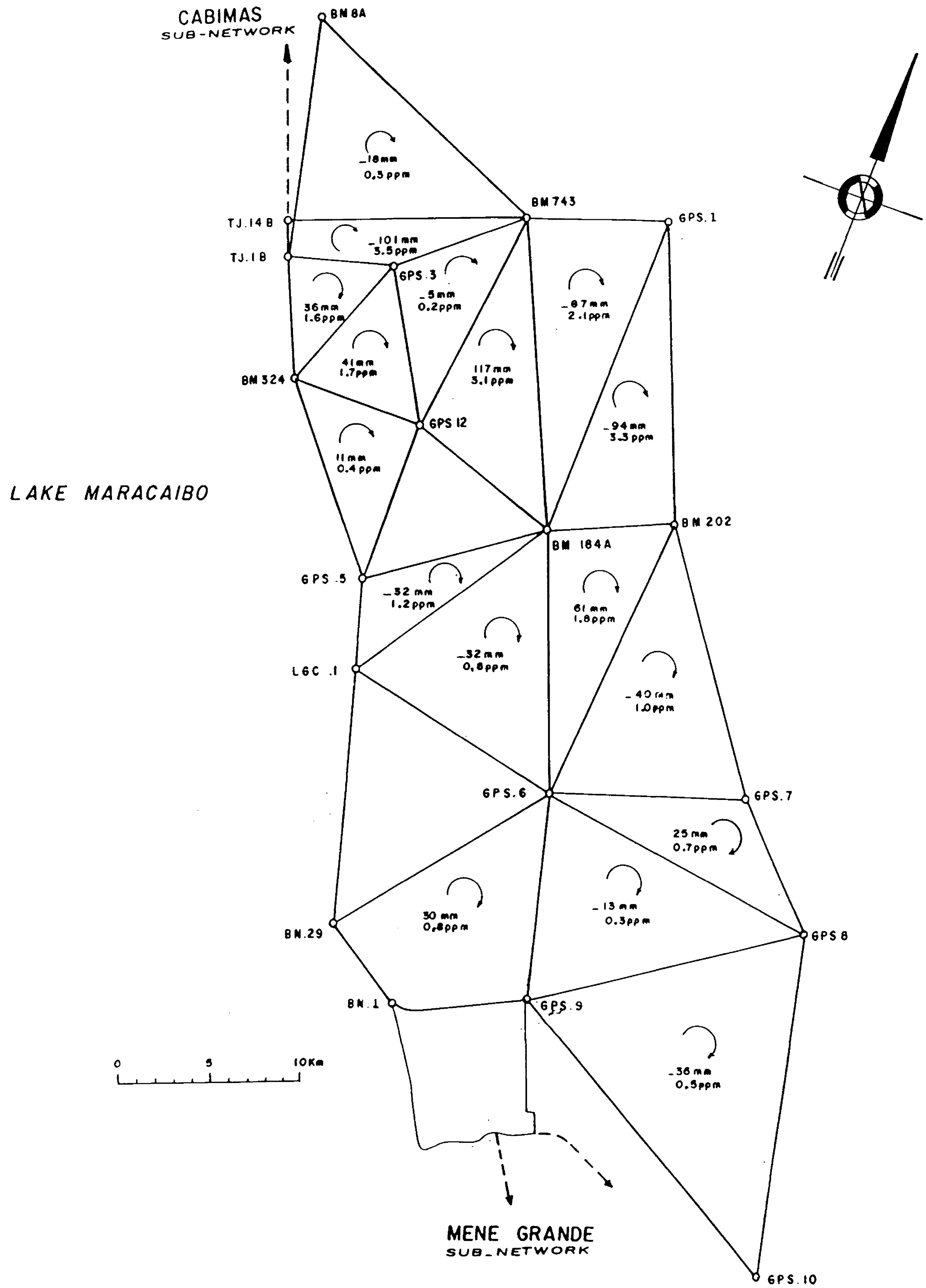

Figure 4.3. GPS Misclosures Campaign No. 3.

campaign was selected due to its higher redundancy in the observations to obtain a statistically more meaningful estimation.

The commonly proposed error model for GPS observations $\sigma^2_{\Delta h} = a^2 + b^2 s^2$ was evaluated. The components a^2 and b^2 were determined using MINQE. The results have shown the component 'b' to be insignificant and 'a' to have a value of 29 mm. This unexpected result (i.e. error independence of baseline length) has been explained to arise from tropospheric effects due to the very hot and humid conditions in the area (32°C and 80% humidity on average). This may lead to the suspicion that at least in small networks the tropospheric effect on the height component is probably independent of baseline length and its influence overruns any distance dependency arising from other factors.

Results of other GPS test surveys conducted at a test network, located in the area of the Mactaquac dam near UNB, to study the feasibility of GPS for deformation monitoring and selection of an adequate receiver model for the Venezuelan campaigns, showed much better accuracies than those obtained in Venezuela. The estimated model components for Trimble and WM101 receivers used in these campaigns taken from Chrzanowski et al. [1988] were:

$a = 10$ mm, $b = 2.2 \times 10^{-6}$ mm for Trimble 4000 SX, and

$a = 7$ mm, $b = 1.4 \times 10^{-6}$ for WM101 (average of two campaigns)

The above tests in Canada were performed in cool climate conditions of late spring and early autumn. It is interesting to note also that the satellite geometry during the UNB campaigns showed VDOP values in the order of 2.5 whereas at the Costa Bolivar campaigns the VDOP values were in the order of 4.5 at best. Therefore, the poor geometry combined with the large tropospheric effects can be blamed for the degraded results obtained in Venezuela.

The comparison of subsidence results obtained from levelling and GPS, under the basic assumption that the geoid has remained stable between campaigns, are shown in Table 4.2. These values were obtained from separate least squares static adjustments of each campaign holding BM 202 as fixed. The comparison is done only on the Tia Juana section of the main network since only one GPS campaign of the whole network has been available.

Table 4.2. Comparison of Subsidence Results - Levelling versus GPS.

Point	Campaign No. 2 - No. 1 [mm]				Campaign No. 3 -No. 1 [mm]			
	GPS	Levelling	Diff.	σ_{DIFF}	GPS	Levelling	Diff.	σ_{DIFF}
GPS1	+1	-8	+9	39	-48	0	-48	39
184A	+14	+5	+9	38	-22	+5	-27	38
743	-16	-7	-9	45	-24	-9	-15	46
GPS3	-33	-15	-18	49	-45	-32	-22	51
324	+11	-5	+16	46	-5	-14	+9	46
TJ1B	-34	-21	-13	52	-86	-44	-42	53
GPS5	-66	-30	-36	49	-23	-54	+31	49

From the results one can notice a reasonable numerical agreement between the first two campaigns and a larger difference, about twice as bad, when comparing campaigns 3 and 1. For most cases the magnitude of the differences are well within their corresponding standard deviations, which is a good indication that the results are not significantly different.

In conclusion, the GPS campaigns in Venezuela have served to indicate the present real accuracy of the GPS height differences under the critical conditions of the tropics and present satellite geometry. The accuracy (at one σ level) was found to be in the order of 29 mm independent of baseline length with VDOP values of around 4.5 to 5. Significant improvements in accuracy are, however, expected in the near future as will be discussed in the next section.

4.4 Future Expectations

The GPS observations are expected to improve significantly in the near future when the full constellation of 24 satellites is operational. The wider and permanent availability of satellites will give enough freedom at the planning stage to minimize the effect of most of the biases and errors discussed above. The most immediate improvements will arise from the better geometry of the satellites, particularly near the equator where, as mentioned already, the satellite distribution will be more uniform than at higher latitudes.

Future values of DOP already indicate an average improvement to about half of the present values. For high precision surveys, optimal windows with minimum GDOP values will be part of the observation plan. In relation to the rest of the biases and errors, major improvements although not quantifiable are also expected.

The permanent availability of satellites will remove constraints on the length of the observation period, so that the economics and the accuracy requirements will dictate the optimal length of time. According to Hothem [1986], a minimum of 120 minutes should be required for precision engineering surveys, but since it will largely depend on local conditions, only experience will have the final word. Longer observation periods tend to randomize the atmospheric effects, orbital biases and the multipath effect.

The larger number of satellites in view, which at certain instances of time may be as high as 12 [Ashjaee et al. 1988], will provide enough satellite redundancy to isolate inaccurate and erroneous satellite observations, leading to much more reliable results. It will also allow the selection of satellites at higher elevation angles (>20°) which will favor the reliability of present tropospheric models and will yield better VDOP values. Furthermore, according to Wells et al. [1986], the overall effect of the orbital biases is expected to decrease with the increase in the number of observed satellites.

With respect to receiver technology, the rapid growth in microprocessor technology has already significantly reduced the size and cost of integrated circuits. One clear example has been the advent of the Ashtech receivers [Ashjaee et al. 1988] with the capability of tracking continuously and simultaneously 12 satellites by means of 12 independent hardware channels fitted in a very compact case of approximately 22 x 12 x 32 centimetres. Thus, all the available information from the satellites can be taken care of, and the need for satellite tracking selectivity or optimization algorithms has vanished. The author believes that all possible data should be collected in the field and the selectivity could be done during the data processing stage. In addition, the quality of the data in the dedicated channel technology will be better since the signal to noise ratio is favoured by continuous tracking.

"All in view," multichannel technology is thought to represent the future trend of receiver manufacturers. Of higher concern are the probable future restrictions in selective availability and corruption of the satellite signals by the DoD. It may cause hardware limitations to some of the presently available receiver types. This is an uncertainty that once defined will clear the view for receiver manufacturers. For static geodetic applications any limitation in the availability of the broadcast ephemeris will be offset by the predicted spread of commercially available precise ephemeris based on VLBI tracking stations. Furthermore, the natural increase of TEC in the atmosphere during the next few years is expected to deteriorate the accuracy. However, the use of dual frequency multichannel receivers and the selection of observation windows during expected minimal ionospheric disturbances will reduce this ionospheric problem. For short baselines under 30 km, as it is the case of the Costa Bolivar GPS network, the effect may be kept low if the above precautions are considered.

For details on future implications on receiver technology and a good review on most receiver types available to September 1988, the reader is referred to McDonald [1988].

As a conclusion it is certain that major improvements in accuracy are about to come as additional satellites will be launched and the full constellation becomes operational. A conservative guess based solely on the expected geometric improvement may be to predict an increase by a factor of at least two compared to present accuracies. Thus, for the case of the Costa Bolivar surveys it is valid to expect future height difference accuracies in the order of 10 to 15 mm. Langley et al. [1986] forecasts future baseline accuracies of the order of 0.05 ppm based on the older configuration of 18 satellites, precise ephemeris, dual frequency observations and WVR. This indicates that our estimates are perhaps too pessimistic.

5. INTEGRATION OF GPS AND LEVELLING

Although GPS observations could be used to completely replace geodetic levelling in subsidence studies, there are cases where the need for detailed information becomes imminent in order to delineate more precisely the subsidence shape and extent. In these cases it may be uneconomical, if not impossible (at least at the present time), to replace geodetic levelling entirely by GPS. Nevertheless, it is feasible to combine both techniques taking advantage of their capabilities for each particular need. For instance, GPS may replace long levelling lines used for connections to stable areas while levelling may be used for densification purposes, especially in urban areas, where the existing structures may critically shield or degrade the quality of the satellite signals. The combination, however, gives rise to height datum problems that have to be alleviated mathematically in order to achieve a homogeneous integration.

This chapter discusses the height incompatibility problem of ellipsoidal versus orthometric heights, and the implications of variations in gravity, a common phenomena in areas where minerals are being extracted. It also presents the designed integration model and accuracy specifications to achieve an optimum combination. Finally, an economic analysis of the combination of GPS and levelling in the Costa Bolivar case and the designed strategy to achieve the integration are discussed.

5.1 Ellipsoidal Versus Orthometric Heights

The combination of geodetic levelling and GPS heights involves problems of height incompatibility. GPS heights are referred to a geocentric ellipsoid (ellipsoidal heights h) and levelling heights are referred to the geoid (orthometric heights H). The two height systems are related through the geoidal height (N). If the value of N can be determined, the transformation may be established through the simple relationship:

$$H = h + N \,. \tag{5.1}$$

Different techniques can be utilized to determine values of N, either on a global or local basis, or combination of the two. Kearsley [1988] provides a brief review on various gravimetric methods presently used for the evaluation of N. The accuracy of N is basically dependent on the distribution, average spacing and accuracy of the input data. Vanícek and Krakiwsky [1982] indicate that for regions with a dense and homogeneous gravity coverage, around the computational point, the geoidal height could be estimated with an accuracy of a few meters. Since levelling involves only height differences between two stations (e.g. p_i and p_j), equation (5.1) can then be rewritten as:

$$H_i - H_j = (h_i - h_j) + (N_i - N_j)$$

or

$$\Delta H_{ji} = \Delta h_{ji} + \Delta N_{ji} \tag{5.2}$$

where ΔN_{ji} is the relative geoidal height between the two points.

Naturally, ΔN can be determined more precisely than N because the dominant sources of errors produce nearly the same effect at both stations ,which tend to cancel in the differencing. The results of published tests in the determination of ΔN, claim agreements between gravimetric and geometric (GPS-levelling heights) methods at the 2 to 3 ppm level. For instance, a GPS survey carried out in the Eifel test network, south of Bonn in the Federal Republic of Germany, yielded a root mean square discrepancy between geometric geoidal heights (derived from GPS-orthometric heights) and gravimetric geoidal heights in the order of 3.3 cm for an average length of 13 km, which is approximately 2.5 ppm [Engelis et al., 1985]. Similarly, Kearsley [1987] reports agreements at the 2 to 3 ppm level. Other tests performed in Canada on the Ottawa and Manitoba networks (Schwarz et al. [1987]) yielded relative agreements in the order of 2 to 4 ppm. Similar results have also been obtained with collocation methods. See Engelis and Rapp, [1984]. All these results are compatible with what is actually attainable with GPS, indicating their validity for GPS height control densification. In subsidence studies, however, the major interest is in changes of elevation between epochs of time, rather than absolute height information. Therefore, the problem of the determination of N or ΔN can be practically eliminated by a method of combining GPS and levelling surveys at a reference epoch of time in order to derive directly the geometric geoidal heights. This is particularly useful in local areas where scarce or no gravity data exists. Furthermore, knowledge of the gravity field and lengthy computer calculations are not necessary. The approach may also provide sufficient geoidal information at discrete points within the area of interest to allow for the mapping of the local geoid, by fitting an appropriate mathematical model to the derived geoidal heights. This way, geoidal heights in the area may be derived directly from the fitted model. This method is referred to in the literature as "geometric geoid modelling" (see King et. al., [1985] and Gilliland, [1986]).

The accuracy of the geometric geoid modelling method depends to a great extent on the goodness of fit of the chosen model and the accuracy of the GPS and orthometric heights involved. Thus, for cases with irregular geoid shapes, the modelling will require the use of complicated mathematical functions and perhaps higher densification of control stations which may turn out to be very costly. For the case of the Costa Bolivar oil fields, a sufficient number of control stations has been available to evaluate the local relative shape of the geoid in the area. Figure 5.1 shows geometric geoidal heights derived, by the author, from separate parametric adjustments performed on the levelling and GPS data of campaign no. 3. The elevation of BM 202 was held fixed in both adjustments. Note that the standard deviations of the adjusted GPS ellipsoidal heights are significantly larger (3 to 4 cm) than some of the N values shown in figure 5.1. However, similar values for N were obtained when the same analysis was performed on the two previous campaigns at common points. Thus, the computed geoidal heights seem to give an indication of a fairly irregular local geoid with approximate deflections of the order of 3" seconds of arc. At first, this seems

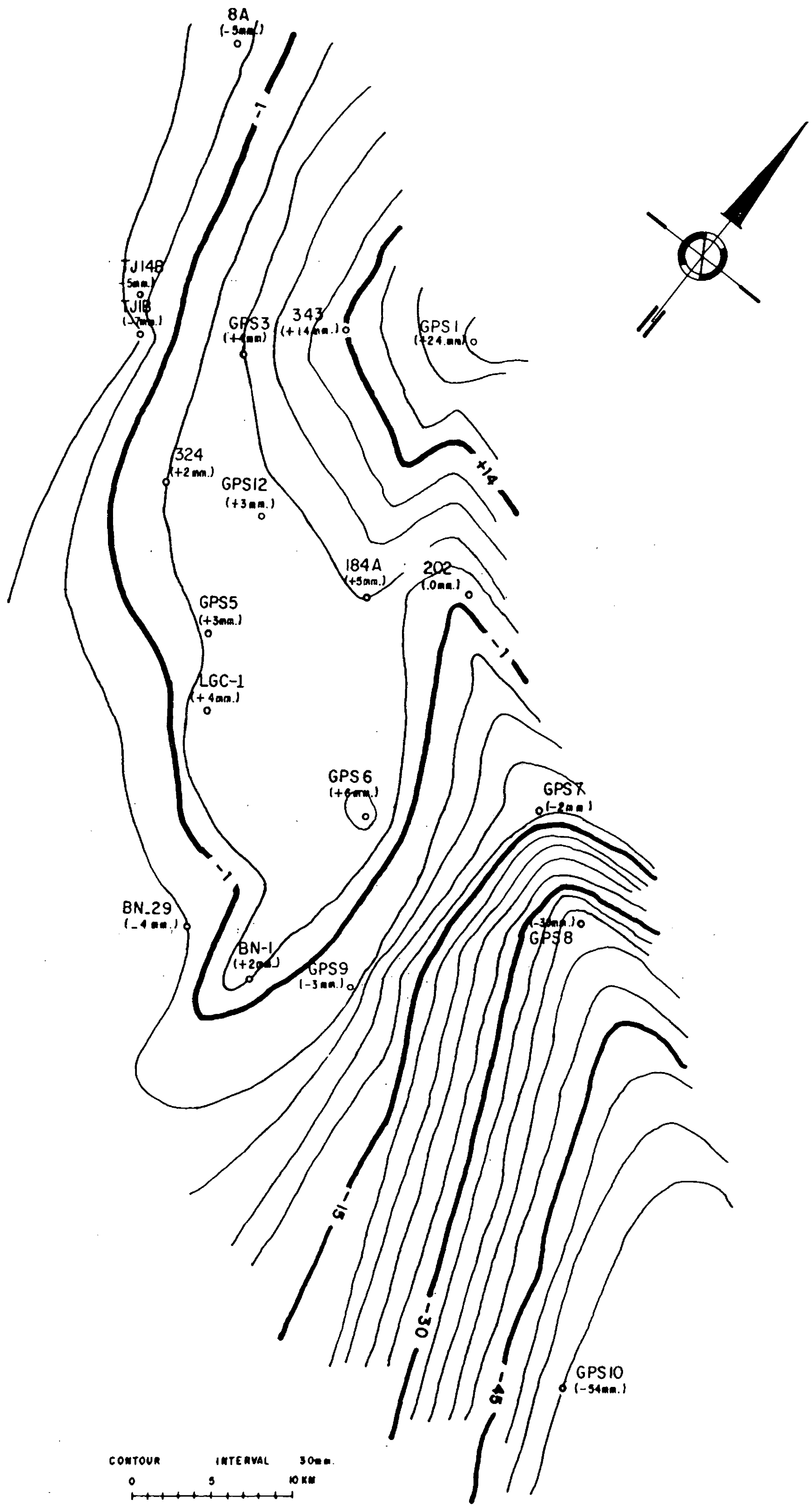

Figure 5.1. Local Geoid (Levelling - GPS) at the Costa Bolivar Network

surprising, if one considers that the topography of the area is generally flat. However, a vast exploitation of oil and water , accompanied by a significant amount of subsidence deformation, has taken place in the whole area within a period of 62 years. For instance, the production records of Maraven S.A.indicate that the total cumulative production of oil and water in the fields of Tia Juana, Lagunillas and Bachaquero up to December 1988 was over 5 billion barrels, equivalent to approximately 7.2×10^{11} kg of mass extracted. Furthermore, the records show that the subsidence has reached, up to March 1988, a maximum value of -5.013 (BM 215B), -4.462 (BM 1242), and -4.470 metres (BM 608C) in the Lagunillas, Tia Juana and Bachaquero fields respectively.

Consequently, for the combination of GPS and levelling in the area, the use of a geometric model to approximate the geoid in the whole area may not fulfill the accuracy requirements due to the smoothing effect and complexity of the fitting. Thus, the safest approach to estimate N or ΔN will be again by the application of a discrete point modelling approach. This implies that any time a new GPS baseline is to be added to the network, a simultaneous GPS and levelling reference survey will have to be performed. This is a rather costly approach but probably the most accurate. At the present time it may be equivalent in accuracy to the gravimetric methods as shown above. However, as the accuracy of relative GPS improves in the future it will most likely become one of the most accurate geoidal height determination methods to be used in precise integrated subsidence monitoring studies. One further problem related to this datum problem will emerge from temporal variations in gravity and their effect on the geoid. This is the topic of the next section.

5.2 The Problem of Gravity Variations

Gravity and the resulting geoid undulations are subject to temporal variations arising from different phenomena such as: water table fluctuations, post-glacial rebound, intraplate tectonics, land subsidence, co-seismic activities, and tidal effect. It has been observed that gravity undergoes changes that range from a few microgals to a few tens of a milligal. The largest values may be expected from seismic activity. Li and Wei [1983] report a trend of gravity variation with an amplitude of 0.1 mGals before the Tangshan earthquake in China. Seasonal ground water fluctuations have been observed to affect gravity in the order of a few tens of microgals or so, as reported by Lambert and Beaumont [1977] for eastern Canada coastal areas. Post-glacial rebound in Fenoscandia has been observed to affect the local gravity at a maximum rate of -1.64 μGals/year [Ekman et al. 1987], and local subsidence has been said to induce variations in the order of 10 μGals/year [Vaníček and Krakiwsky, 1982]. In mining areas subject to subsidence, these variations are usually associated with two main factors; namely, underground mass density variations and subsidence on the surface.

The contribution arising from the surface subsidence could be evaluated from the well known gravity gradient formula given in most geodesy text books [Vanícek and Krakiwsky, 1982; or Torge, 1980], or could be approximated by the commonly known gravity gradients (e.g. the free air gravity gradient or Bouguer gradient, etc.). The actual gravity field gradient is known to vary locally and regionally depending on the topography and the underground density variations. Drewes [1986] mentions a current gradient of dg = -0.2 mGals/m dh determined for the Costa Bolivar oil fields. The same gradient has been used to convert significant gravity variations into height variations in the rifting process in Northern Iceland [Torge and Kanngieser, 1981] and also a local gradient of 0.23 mGal/m has been estimated in the Fenoscandia uplift [Ekman et al., 1987].

The influence of density changes, on the other hand, may be easily evaluated from Newton's law of universal gravitation

$$F = \frac{GMm}{r^2} \tag{5.3}$$

where G is the constant of proportionality equivalent to $6.672 \times 10^{-11} kg^{-1} m^3 s^{-2}$,

M and m represent point masses.

r is the distance between the centres of the two masses, and

F is the force of attraction.

Taking B to be a body of mass M_B, and m_A a point mass at the observing station on the surface of the earth (Figure 5.2), the total component of the force of attraction in the direction of the gravity field (z) could be evaluated by adding the individual components of the force exerted upon m_A by each particular element of mass dM_B. Thus, the formula may be expressed in the form:

$$F_z \doteq [G\rho \iiint_B \frac{\delta M_B}{r^2} \cos\alpha]\, m_A \qquad (5.4)$$

where ρ represents the density of the material in B and is considered as constant through the whole body.

Since m_A has been assumed to be a unit mass, the bracketed term in (5.4) could be considered to be equivalent to the magnitude of the gravitational attraction exerted by M_B upon m_A in the z direction. So, the change in the magnitude of gravity registered at point p_A due to the removal of the body B may be expressed by the formula:

$$\delta g \doteq G\rho \iiint_B \frac{\delta M_B d}{r^3} \qquad (5.5)$$

Equation (5.5) can then be used to evaluate the effect of future oil and water extraction upon the local gravity field in the Costa Bolivar area. Several assumptions have been made to simplify the computation. The first one is that all the reservoirs in the area were represented by three rectangular shapes equivalent in area to the major reservoirs located over Lagunillas, Tia Juana and Bachaquero. The dimensions were approximated, following the zero subsidence isoline in the cumulative subsidence maps issued at Maraven S.A. which is equivalent to the real shape of the reservoir according to Mendoza [1989]. The dimensions and average depths were as follows:

Lagunillas - area 14 x 8 kilometres, depth 700 m;

Tia Juana - area 12 x 8 kilometres, depth 400 m at 20 km from Lagunillas;

Bachaquero - area 14 x 12 kilometres, depth 500 m at 21 km from Lagunillas.

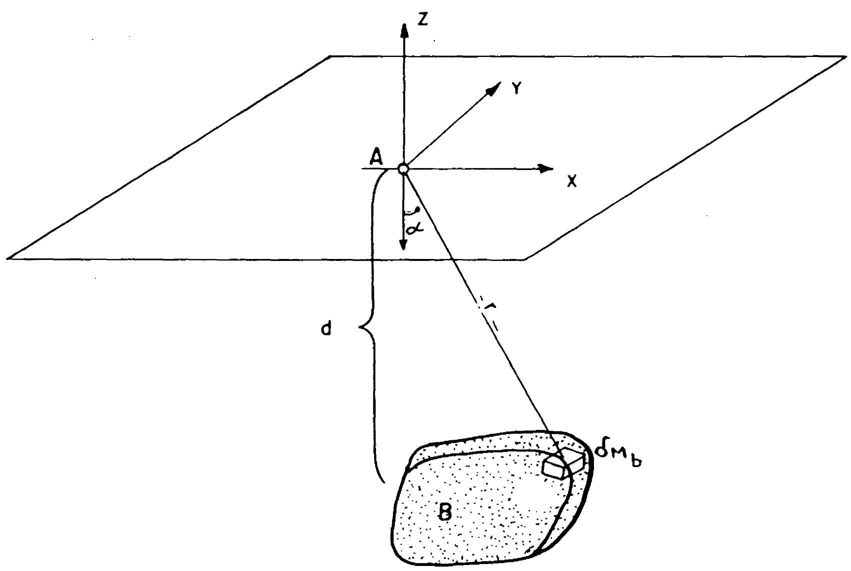

Figure 5.2. Geometry of the Gravitational Attraction of a Body B Upon A.

The theoretical observation point was set at the center of the Lagunillas polder to obtain the maximum variations. The official level of activities regarding future exploitation for the next 5 years (1988-1994) was used as the most reliable source of information to estimate the volume of oil to be extracted from each of the above fields. The computed values were 217.9 million barrels for Lagunillas, 169.7 million barrels for Tia Juana and 198.6 million barrels for Bachaquero. Considering an additional 30% volume of water (which is in average the percentage of water extracted with the oil) and assuming an approximate density of 1000 kg/m^3 for the mixture of heavy oil and water, the total variation of gravity in the next 5 years at Lagunillas, resulting from the removal of fluids, was estimated to be -14μGal. Since subsidence is also taking place at the same time, the empty space is being replaced by rock and one has to estimate also the influence of this effect on gravity. As a first step, the volume occupied by rock, which for practical purposes could be taken as approximately equal to the volume of the subsidence on the surface must be estimated. From the records at Maraven it has been established that the relationship between volume of subsidence versus volume of fluids extracted is in average approximately equal to 0.85. Thus, taking a density of 2600 kg/m^3, as normally used for rock, the change in gravity caused by replacing 85% of the previous volume by rock was estimated to be +25 μGals. Finally, from the algebraic sum of the above values a net change in gravity of +11 μGals in five years was estimated. Assuming a linear trend for the next 10 years (1999) the variation may reach +22 μGal. Gravity measurements have been carried out in the Costa Bolivar area [Drewes, 1978; Drewes et al., 1983; and Benitez et al., 1981]. A gravimetric network located in the northwest extreme of the Tia Juana field has shown maximum variation rates of +25 μGal/year [Drewes et al., 1983]. When considering the subsidence contribution and the local gradient of 0.2 mGal/m, for an average subsidence of 12 cm/year in Tia Juana, the mass variations component is estimated to be equal to +1 μGal/year. This is compatible with a value of +1.3 μGal/year obtained by moving the theoretical observation point, from Lagunillas to the Tia Juana field in the above computations.

Of interest now is to evalute the effect of the estimated gravity variations on the orthometric height differences and the geoid. In subsidence studies and particularly at the Costa Bolivar, relevellings are usually performed following the same route in every epoch. Thus, the influence of the gravity variations on the orthometric height differences as derived by Vaníček et al. [1980], is reflected only in the differential orthometric correction. An estimate for our particular case is obviously negligible since for extreme conditions of a variation of 0.1 mGal and height differences of 1000 metres the differential correction is in the order of one millimetre [Vaníček et al, 1980].

The influence on the geoid can be evaluated by means of Stoke's formula (taken from Heiskanen and Moritz [1967, p. 95])

$$N = \frac{R}{4\pi\bar{g}} \int_{o}^{2\pi} \int_{o}^{\pi} \Delta g\,(\psi, \alpha) S(\psi) \sin\psi \, d\psi \, d\alpha \qquad (5.6)$$

where Δg is the corresponding gravity anomaly for a particular point on the earth's surface with spherical polar coordinates, ψ (spherical distance) and α (the azimuth).

$\bar{g}$ is the mean gravity value (980.3 Gals),

R is the mean radius of the earth (6371 km) and $S(\psi)$ is the Stokes function.

Following the same methodology used by Vaníček et al. [1980] and Vaníček and Krakiwsky [1982], the author derived (see Appendix II) an expression relating changes in geoidal height as a result of variations in gravity caused by mass density variations only.

The expression is valid for an area within a maximum limit of $\psi < 10°$ and having a conical model behaviour (i.e. decreasing linearly from maximum at $\psi = 0$ to zero at $\psi =$ max). The derived equation is as follows:

$$\delta N \approx 3.7 \text{ mGal}^{-1} \, \delta g_{max} \psi_{max} \qquad (5.7)$$

where δg_{max} is the maximum average change in gravity due to mass density variations only, taken in mGals, and ψ_{max} is the maximum spherical distance from the point of interest taken in radians.

Hence, for the area of the Costa Bolivar with a radius of 50 km ($\psi \approx 0.5°$) and the estimated maximum variation of +11 μGal, the effect on the geoid from (equation 5.7) is estimated to be 0.4 mm in 5 years. This indicates that the gravity variations effect associated with oil extraction at least in the short run can be safely neglected in the integration of GPS and levelling at the Costa Bolivar.

5.3 Integration Model

In Chapter 2 the discussion of subsidence modelling led to the formulation of the general observation equations (2.11) relating the orthometric height differences to the deformation model. When considering GPS height differences, the same equations could be written if only GPS were to be used. However, for the combination with orthometric heights the observation equations have to include the geoidal height parameter (N or ΔN) in order to achieve homogeneity in the system of equations.

Similar to the case of subsidence deformation modelling, the geoidal heights can also be modelled locally by fitting a suitable mathematical function to the differences between GPS ellipsoidal height differences and the orthometric height differences. Thus, the general geoid model as a function of position and time may be expressed as:

$$N(x,y;\, t) = \mathbf{q}(x,y;\, t)\varepsilon \tag{5.8}$$

where N is the geoidal height at point x, y and at time t, $\mathbf{q}$ is a row vector of base functions and ε is the vector of unknown coefficients. The general model encompasses either surface functions or discrete point models.

For further illustration, consider the case of a plane fit to the geoidal height differences (ΔN). If the temporal variations are neglected, one equation of the form

$$\Delta N\,(\Delta x, \Delta y) = a_1\Delta x + a_2\Delta y \tag{5.9}$$

may be written for each particular value of ΔN. At least two equations will be needed to uniquely define the plane. A plane fit will most likely be applicable to very local areas with a smoothly varying geoid. For more complicated geoid shapes a higher degree polynomial or any other suitable function may be necessary.

When combining GPS and levelling, the general model can be integrated directly, in the observation equations and the model fitting done simultaneously in the least squares adjustment. This approach is still valid even if the geoidal model is also time dependent. The reason is that GPS provides a geoid independent measurement of the subsidence deformation as opposed to levelling where there is no distinction between subsidence and temporal geoid variations. Therefore, the combination of both types of data allows to isolate the time dependent parameters of the geoidal model. The general observation equations relating the GPS ellipsoidal height differences to the deformation and geoidal models, for a pair of points p_k and p_ℓ at epochs t_o and t_1, may then be expressed as:

$$\Delta h_{k\ell}(t_o) + \mathbf{v}_{k\ell}(t_o) = H_\ell(t_o) - H_k(t_o) - [\mathbf{q}(x_\ell, y_\ell; t_o) - \mathbf{q}(x_k, y_k; t_o)]\boldsymbol{\varepsilon} \qquad (5.10\mathbf{a})$$

and

$$\begin{aligned}\Delta h_{k\ell}(t_1) + \mathbf{v}_{k\ell}(t_1) = H_\ell(t_o) - H_k(t_o) &- [\mathbf{q}(x_\ell, y_\ell; t_1) - \mathbf{q}(x_k, y_k; t_1)]\boldsymbol{\varepsilon} \\ &+ [\mathbf{b}(x_\ell, y_\ell; t_1 - t_o) - \mathbf{b}(x_k, y_k; t_1 - t_o)\mathbf{c}\,.\end{aligned} \qquad (5.10\mathbf{b})$$

Equations (2.10) are the equivalent observation equations for the orthometric height differences. Note from equations (5.10) and (2.10) that for the solution of a discrete geoidal height model there must be at least one set of observation equations (5.10) for each GPS derived height difference, and a corresponding set of equations (2.10) connecting the extreme points of the GPS baselines.

A (k+1) multiepoch solution, similar to equation (2.12), is also possible, and can be expressed in matrix form as:

$$\begin{pmatrix} \boldsymbol{\ell}(t_o) \\ \boldsymbol{\ell}(t_1) \\ \cdot \\ \cdot \\ \cdot \\ \boldsymbol{\ell}(t_k) \end{pmatrix} + \begin{pmatrix} \mathrm{v}(t_o) \\ \mathrm{v}(t_1) \\ \cdot \\ \cdot \\ \cdot \\ \mathrm{v}(t_k) \end{pmatrix} = \begin{pmatrix} \mathbf{A}_o & \mathrm{Q}_o & 0 \\ \mathbf{A}_1 & \mathrm{Q}_1 & \widetilde{\mathbf{B}}_1 \\ \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot \\ \mathbf{A}_k & \mathrm{Q}_k & \widetilde{\mathbf{B}}_k \end{pmatrix} \begin{pmatrix} \boldsymbol{\xi} \\ \boldsymbol{\varepsilon} \\ \mathbf{c} \end{pmatrix} \qquad (5.11)$$

where $\ell(t_i)$ is the vector of observed height differences for different campaigns t_i $(i = 0, k)$, $v\,(t_i)$ is the vector of residuals, ξ is the vector of unknown constants (heights or height differences), ε and $\mathbf{c}$ are vectors of unknown coefficients, Q_i is the design matrix constructed from geoidal model (5.8) for the observations at epoch t_i and $\tilde{B}_i$ is already defined in section 2.3. For futher illustration, a short example for the case of a plane fit to the geoid and a point velocity deformation model is shown in Appendix III.

In Chapter 2 the need for a discrete point deformation model for the Costa Bolivar subsidence studies lead to the selection of the constant velocity model equation (2.7) for all subsidence modelling in the area. It was also discussed, under section 5.1, that the discrete point modelling was probably the best approach to deal with the geoidal height modelling problem at the Costa Bolivar due to the apparent irregularity of the local geoid and the need for accurate results. From the discussion in section 5.2 it became also clear that, at least in the short run, one could neglect any time variations of the geoid (i.e. N = 0) and assume N to be constant. Therefore, the proposed observation equations, in the short run, for the combination of GPS and levelling in the Costa Bolivar may be sumamrized for points p_k, p_j, p_ℓ, p_m and epochs t_o and t_1 as follows:

a) For levelling

$$\Delta H_{kj}(t_o) + v_{kj}(t_o) = H_j(t_o) - H_k(t_o) \qquad (5.12a)$$

$$\Delta H_{kj}(t_1) + v_{kj}(t_1) = H_j(t_o) - H_k(t_o) + (t_1 - t_o)(\dot{H}_j - \dot{H}_k) \qquad (5.12b)$$

b) For GPS

$$\Delta h_{\ell m}(t_o) + v_{\ell m}(t_o) = H_m(t_o) - H_\ell(t_o) - (N_m - N_\ell) \qquad (5.12c)$$

$$\Delta h_{\ell m}(t_1) + v_{\ell m}(t_1) = H_m(t_o) - H_\ell(t_o) - (N_m - N_\ell) + (t_1 - t_o)(\dot{H}_m - \dot{H}_\ell) \qquad (5.12d)$$

From equations (5.12) one may notice that for each additional future campaign the solution must include at least the reference campaign (t_o) where common GPS and levelling surveys took place in order to avoid solvability problems. Indeterminances may also arise when observations in only one epoch of time, corresponding to new or re-built BM's, are included in the observation equations. In this particular case, they should either be removed from the system of equations until an additional campaign is performed, or if the historic behaviour of the area is known, the approximate velocity or a dummy observation could be extrapolated and included in the system of equations. For cases where additional GPS baselines are added in the future, there must be an equivalent levelling survey run simultaneously. Then, both sets of observations will become part of the so-called reference campaign together with their corresponding date of observation. This particular modelling approach has the advantage of being suitable for multiepoch analysis, leading to more rigorous results as more campaigns are processed together. However, one has to consider the limitation of the computer capacity especially when over 1600 BM's are involved in several campaigns.

Another approach, discussed already in Chapter 2, is the observation differences approach. When applied here, the geoidal heights will also disappear from the observation equations and equation (5.12) will reduce to the following:

a) For levelling

$$d\Delta H_{kj}(\Delta t_{1\text{-}0}) + dv_{kj}(\Delta t_{1\text{-}0}) = (t_1\text{-}t_o)(\dot{H}_j\text{-}\dot{H}_k)$$

b) For GPS

$$d\Delta h_{\ell m}(\Delta t_{1\text{-}0}) + dv_{\ell m}(\Delta t_{1\text{-}0}) = (t_1\text{-}t_0)(\dot{H}_m\text{-}\dot{H}_\ell) \ .$$

The advantages of this approach are that the deformation model parameters are the only unknowns in the least squares solution and that common systematic errors may be eliminated in the differencing leading to more accurate results. There is also no need for simultaneous levelling campaigns each time a new GPS line is added, and no need to maintain a common GPS and levelling reference campaign (t_o) in future solutions, as long as the same observables are repeated in the two campaigns being differenced.

The main disadvantages are the limitations to only a pair of campaigns at a time, and the need for the same network geometry in both campaigns in order to take full advantage of the data.

As a conclusion to this section, it is proposed to take advantage of both approaches by using the differencing approach at the beginning when only a few campaigns will be available, and eventually lead into the multi-epoch solution as additional campaigns are added. This will allow also to assess the temporal behaviour of the geoidal heights and probably re-design the adopted model.

A computer program suitable to handle the previously described integration problem has been developed by the author as a contribution to MARAVEN S.A. and is available to any other Venezuelan oil industry operating companies engaged in subsidence monitoring activities. The program served as the basic tool for most computations performed within this research. It was developed in FORTRAN 77 using an IBM 3090 mainframe computer. For further analysis herein the multiepoch modelling approach has been used.

5.4 Design of Integrated Network and Field Surveys

The well-known advantages of GPS such as: all weather capability, three-dimensional information, no need for intervisibility between stations, high accuracy, and its economical benefits over conventional methods were the basic reasons to integrate GPS with levelling in the subsidence monitoring studies of the Costa Bolivar oil fields. It has been realized that, at least at the present time, GPS cannot provide the same accuracy as geodetic levelling. However, the savings added to the capability to generate horizontal control information in the subsidence areas including the coastal dykes, and the feasibility to expand the connections to more stable areas in the future were considered of enough importance to offset the expected small deficiency in accuracy. Consequently, the Maraven levelling scheme, described in Chapter 3, was analysed with Dr. Adam Chrzanowski and Dr. Richard Langley acting as consultants from the University of New Brunswick, Canada, in the design of the integrated GPS and levelling network.

The design included a GPS network to replace the primary levelling frame and the redesign of the densification field surveys. As a result, the GPS network mentioned in Chapter 4 and shown in Figure 4.3 (for campaign No. 3) was proposed, together with several modifications to the traditional monitoring scheme utilized by Maraven.

The GPS network was designed based on the following criteria:

(a) A maximum baseline length condition b < 30 km to allow, in the worst case, for the use of single frequency receivers and enough correlation to minimize common systematic effects in the differencing.

(b) A self-sufficient network geometry (i.e. no configuration defects) with enough redundancy to allow for a separate least squares adjustment, data screening and accuracy evaluation.

(c) The selection of GPS stations at nodal bench marks where maximum horizontal displacements would be expected from the subsidence behaviour in order to add information on the horizontal deformations taking place.

(d) To include in the design, survey monuments which are part of the horizontal deformation monitoring of the dykes.

(e) To use, where possible, existing bench marks directly as GPS stations.

The modifications to the Maraven scheme included:

(a) Elimination of all primary levelling lines connecting to stable areas and adjacent subnetworks. These are shown as interrupted lines in Figure 5.3.

(b) Replacement of all the double run levelling lines by single run levelling lines in the primary network and subnetworks.

(c) Elimination of the overall classification of lines (e.g. nodal lines, secondary lines, etc.) distinguishing only between GPS and levelling lines.

(d) Addition of the date of observation to the set of field observables.

The implementation of the new design required a common reference campaign of simultaneous GPS and levelling surveys to solve for the aforementioned geoidal height

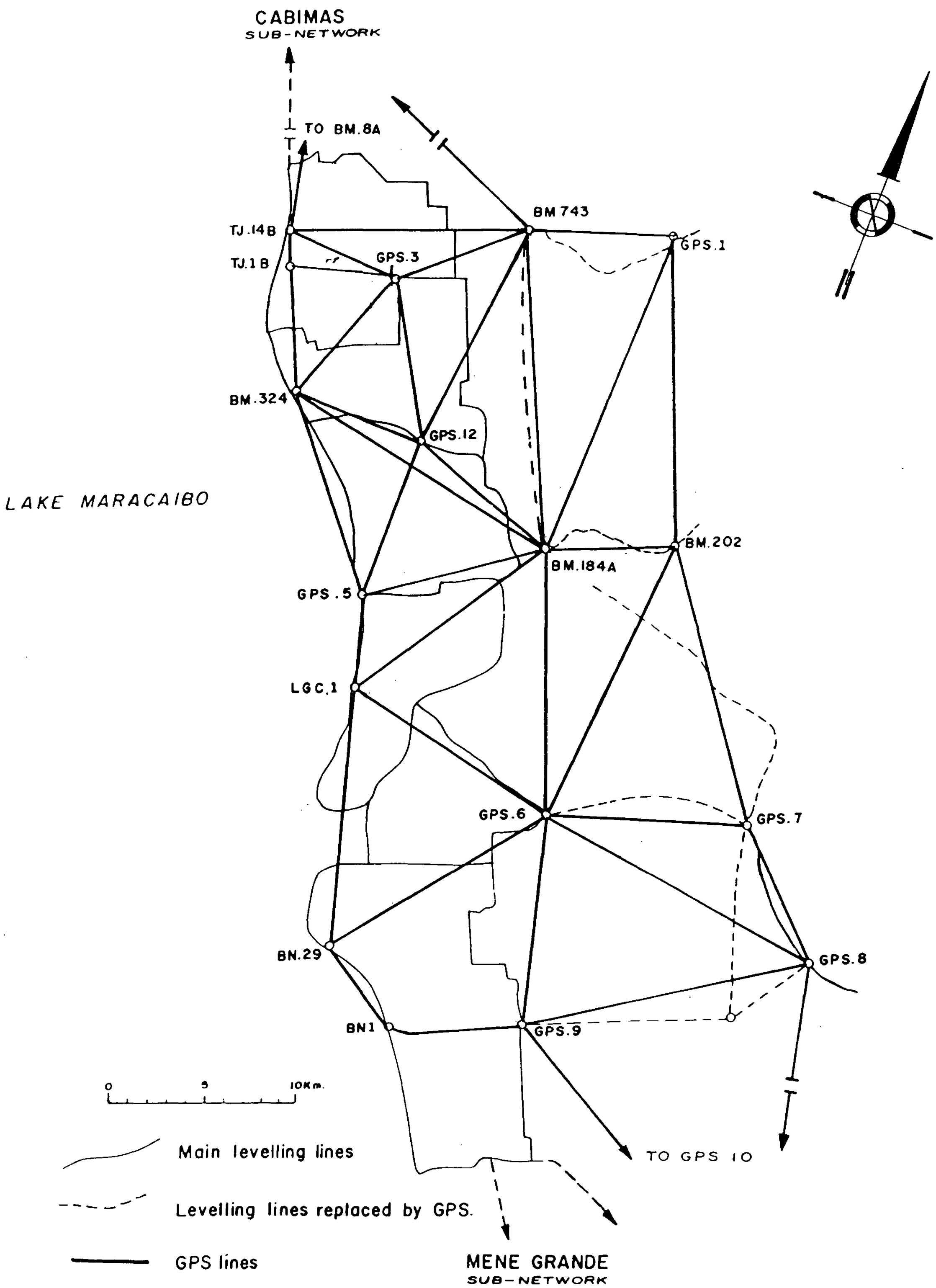

Figure 5.3. Designed Integrated Network.

parameter. This campaign was held in April 1988, as campaign No. 3, but it will be repeated in April 1990, because the accuracy given by GPS (see section 4.3) was lower than expected.

Based on this design and the modelling aspects discussed in section 5.2 the accuracy specifications of GPS for an appropriate combination can now be discussed.

5.5 Accuracy Standards

In order to evaluate the accuracy standards needed to achieve an optimum integration, various preanalyses were conducted, based on the geometry of the integration networks discussed above and the general modelling technique from section 5.3. The aim of the preanalyses was to determine the accuracy of the GPS and redesigned levelling surveys needed to match the accuracy of the subsidence determination as obtained from the Maraven levelling data, in a minimal constrained adjustment. The integration preanalysis, as it will be called from here on, considers the simultaneous solution of a reference campaign together with a subsequent campaign of the redesigned integrated network described in section 5.4. For simplicity, only the original primary net was utilized instead of the full network with densification levelling lines. The expected accuracy of the subsidence determination was evaluated for GPS accuracies of 10, 15, and 30 millimetres and levelling accuracies of 2 mm $\sqrt{k}$ and 4 mm$\sqrt{k}$ per kilometre as obtained in the previous accuracy evaluation. A standard deviation of 2 mm$\sqrt{k}$ was utilized for all the levelling observations in the reference campaign and 4 mm$\sqrt{k}$ for subsequent campaigns. One simulation using 4 mm$\sqrt{k}$ standard deviation in the reference campaign was also performed.

For comparison purposes, the accuracy of the subsidence determination as obtained from levelling only and based, as mentioned already, on a minimally constrained solution (BM 1175DP fixed), was evaluated by a kinematic adjustment of the 1986 and 1988 Maraven levelling data. Similarly, all the integration preanalyses were performed using the same minimal constraint. The results are shown in Table 5.1. The BM's listed correspond

Table 5.1. Preanalyses Results (standard deviations of subsidence in [mm]

*k = distance in km.

BM	SUBSIDENCE ACCURACY LEV. 86-88	$\sigma_{lev_I} = 2\ mm\ \sqrt{k}$* $\sigma_{lev_{II}} = 4\ mm\ \sqrt{k}$ $\sigma_{GPS} = 10\ mm\ \sqrt{k}$	$\sigma_{lev_I} = 2\ mm\ \sqrt{k}$* $\sigma_{lev_{II}} = 4\ mm\ \sqrt{k}$ $\sigma_{GPS} = 15\ mm\ \sqrt{k}$	$\sigma_{lev_I} = 2\ mm\ \sqrt{k}$* $\sigma_{lev_{II}} = 4\ mm\ \sqrt{k}$ $\sigma_{GPS} = 30\ mm\ \sqrt{k}$	$\sigma_{lev_I} = 4\ mm\ \sqrt{k}$ $\sigma_{GPS} = 15\ mm\ \sqrt{k}$
1175DP	0	0	0	0	0
1329DP	14	10	14	28	14
1326DP	15	12	17	33	17
744	12	9	12	23	13
734	12	11	14	23	15
1056	12	11	14	23	15
411	12	12	15	24	16
387	13	11	15	25	16
329B	12	12	15	24	16
1390A	11	10	13	23	14
1703	11	11	14	23	15
M	9	9	12	22	13
184A	8	8	11	22	12
1791	9	10	13	22	14
-AB	11	9	13	23	14
117	12	11	14	23	16
46A	11	11	14	23	16
856	12	8	12	22	12
1725	12	10	15	29	15
846A	12	11	14	23	15
546A	12	11	14	23	16
580A	14	12	16	25	17
639A	13	11	15	25	16
691B	15	15	18	27	20

junction and extreme BM's in the network.

The results show, as expected, a maximum standard error at bench mark 691B which is the farthest away from the constrained point (see Figure 3.2). For the analysis of levelling alone, the standard error is equal to 15 mm and is compatible at a 95% confidence level (29 mm) with the requirements of Maraven, which, according to Murria and Abi Saab [1988], are in the order of 3 cm. When comparing the preanalyses results of the integrated survey with this maximum value, compatible results are found when standard deviations of GPS height differences are 10 mm and of levelling are 2 and 4 mm$\sqrt{k}$.. For the case of GPS height differences with standard deviations of 15 mm, there is a slight deterioration in accuracy which exceeds the requirements of Maraven by 5 mm at the 95% confidence level and may still be considered adequate, when considering that a similar deterioration in accuracy is introduced by the Maraven computational scheme, as discussed in sections 3.4.1 and 3.6.6. For lower accuracies of GPS and levelling the results deteriorate rapidly and tend to be unacceptable.

The results of the simulation using a standard error of 4 mm/$\sqrt{k}$ in the reference campaign and 15 mm for the GPS derived height difference indicate the convenience for having higher levelling accuracy in the reference campaign. This is justifiable also from the point of view of the accuracy of the geoidal heights parameters which are to be derived from the simultaneous GPS and levelling survey in the reference campaign.

As a conclusion, the accuracy standards for GPS and levelling surveys at the Costa Bolivar subsidence studies are as follows: The standard deviations of GPS derived height differences should be between 10 and 15 mm, and the standard deviations for levelling should be equal or smaller than 2 mm$\sqrt{k}$ for the survey of the primary network in the reference campaign and 4 mm$\sqrt{k}$ for the densification lines in the same reference campaign and for all levelling surveys in subsequent campaigns.

5.6 Strategy for the Integration

As a result of the whole discussion, a strategy for the integration of GPS and levelling at the Costa Bolivar area has been designed. It focuses separately on the field work and computational aspects regarding the reference and subsequent campaigns.

5.6.1 Field strategy

For the reference campaign (next March 1990), the whole levelling network will be measured using basically the same field procedures and instrumentation presently used by Maraven S.A., adding the date of the observation as a new observation parameter. At the same time the GPS survey of the designed network shown in Figure 5.3 will also be conducted. Nothing specific can be said about instrumentation for the GPS survey other than the fact that multichannel receivers are preferred for maximum satellite tracking and better data quality. Since the campaign will be held in an epoch of nearly maximum TEC in the atmosphere, the possibility of using dual frequency receivers must be seriously considered. A final decision on the model of receivers to be used will be made towards the end of 1989. The antenna phase centre should be mapped since different receivers may be used later on in subsequent campaigns, and a maximum cut off angle of 15° should be observed.

Further specifications regarding field procedures such as length of observation period, time of the observation windows, collection of meteorological data, etc. cannot be established yet, since that depends basically on the local conditions and availability of satellites. Consequently, one has to rely on past experience and the experience of others. Refer, for instance, to Hothem [1986] for practical specifications to be considered in this type of survey. Once the full constellation is in place an optimum set of specifications will certainly have to be established, not only for subsidence surveys, but also for other types of applications.

For subsequent campaigns the levelling field surveys will be modified as discussed already in section 5.4. The new design includes the following variations with respect to the reference campaign:

(a) Elimination of connection lines to stable areas and subnetworks. These are:

From BM 1329DP	To BM 743
From BM 744	To BM 184A
From BM 1175DP	To BM 184A
From BM 184A	To BM 1725
From BM 1725	To BM 856
From BM 1725	To BM 1324
From BM 1327	To BM 639A
From BM 387	To BM VP69A
From BM's 881A and 880A	To BM PC5

(b) Adoption of the same field procedures as used in the survey of secondary lines (section 3.4.4) for the whole network and subnetworks, i.e. equivalent to U.S. second order class II standards, single run.

(c) Change the traditional strategy of levelling simultaneously towards the areas of major subsidence rates to a strategy of rapid circuit closures. This will allow a more rigorous and immediate assessment of the field data specially now that levelling will be performed in one direction only.

(d) The length of the circuits should be kept below 40 km and a maximum tolerance of 8 mm $\sqrt{k}$ (k is the distance in kilometres) will be used to test circuit misclosures. In the case of rejected misclosures, rechecks against the previous year observations and immediate relevellings of suspected sections will be required.

Simultaneous surveys of the GPS network will be required during each campaign adopting the necessary specifications for maximum accuracy. For the case of additional GPS baselines in the future, simultaneous levelling surveys must be conducted between the

GPS stations and ties to the main network for the geoidal height determination. Each GPS station should be provided with a set of references located around the station in order to relocate precisely lost monumentation or monitor very localized movements of the monuments themselves. For the cases of destroyed monuments, the replacement should be located as close as possible to the previous position and should be distinguished by a different name in order to avoid later confusion.

5.6.2 Computational strategy

For the reference campaign a static parametric adjustment of the levelling data will be performed for outlier detection and comparison with the elevations from the previous campaign in order to estimate the subsidence and the official elevations for Maraven. Similarly a parametric adjustment of the GPS derived height differences must be performed for outlier detection, accuracy evaluation, and error model determination using the MINQE technique. In subsequent campaigns, this practice of separate static adjustments will continue for data screening and accuracy evaluation. At the same time, the results of the GPS adjustments may be used in a weighted similarity transformation to assess the stability of the reference deep BM's which are used as constraints in the least squares adjustment.

At the beginning, the observation differences approach may be used in the estimation of the deformation parameters. Later on, as more campaigns are added, the multi-epoch solution will be adopted and to avoid solvability problems at this stage, the reference campaign must be included in all solutions. For the case of additional GPS baselines in the future, the two sets of observations (levelling and GPS) will become part of the reference campaign with their corresponding observation date. For new or replaced monuments with only one observation epoch, one could either neglect them from the solution, if possible, until another observation epoch is available, or interpolate in time and space a dummy observation, for the previous campaign, based on the historic deformation records of the area. When a new observation is available, from future campaigns, the dummy observations may be removed or given zero weights.

Similarly, if a replaced monument happens to be one of the reference GPS stations, the offset with respect to its original position, may be measured directly using the surrounding references and added to the original levelling observation to create the new reference observation for future computations. Otherwise, a simultaneous levelling survey must be performed.

For a multi-epoch solution over a long time span, one must be aware that the chosen models may fail to depict the real behaviour of the deformation at discrete points. Therefore, new parameters may have to be added or new models may have to be included for these particular points. A similar treatment will have to be given to the assumed geoidal model.

The results of the integrated solution will be the velocity of each BM with its corresponding stochastic information which can then be used to estimate the subsidence between any two given reference dates and the official elevation of Maraven for the chosen date (first of March of the campaign year). Further analyses may be performed by the application of the UNB Generalized Method to model the subsidence as described already in section 3.6.7.

5.7 Cost Analysis

5.7.1 Initial investment cost in Maraven operations

In order to evaluate the cost of GPS for the subsidence monitoring application at the Costa Bolivar oil fields, it is necessary to highlight the fact that the subsidence application constitutes only a marginal part of the wide range of applications that GPS will have in the Venezuelan oil industry operations. Therefore, it has been determined that the cost of the initial investment for testing the application of GPS in the area should not be absorbed solely by the subsidence budget but should be shared among the rest of the application budgets within the first year of use. Consequently, assuming a usage factor of 100 days/year (based on the present use of TRANSIT satellites and potential applications) and considering the use in subsidence to be of about 8 days every two years, the application of GPS in subsidence has been estimated to be of the order of 4% of the total use, which is equivalent to a share of 120,000 Bs out of the initial investment. Furthermore, the cost of the reference campaign which will also become an additional initial investment must be estimated and added to the previous amount in order to obtain the total initial investment cost to be amortizable by the subsidence application.

Part of the initial investment is also the cost of instrumentation. In this respect, regarding the uncertainty in the future cost of GPS instrumentation and based also on a theoretical date of purchase of receivers by Maraven (end of 1991), the following assumptions have been made.

a) purchase cost per receiver Cdn. $60,000. (equivalent to 725,000 Bs);

b) depreciation period, 5 years;

c) maintenance cost 20% of the purchase cost.

Making use of the above usage factor of 100 days/year, the cost of instrumentation has been estimated to be 1740 Bs/receiver/day (equivalent to Cdn $144). This rough value will be used in further analyses within this section. Note that these are very rough estimates and by no means should be considered as accurate.

5.7.2 Replacement of levelling by GPS

Of interest now is to estimate the economic feasibility of GPS in the subsidence monitoring application at the Costa Bolivar oil fields. First of all, various assumptions regarding certain aspects of the survey design and field logistics for all campaigns, must also be made and are listed as follows:

a) A total of four receivers will be available for the surveys.

b) Each receiver will be operated by a field crew formed by a technician, a labourer and a vehicle.

c) Each crew will be available only for a total of 8 hours per day which is also assumed to be the optimum window for best results.

d) A total of two field sessions will be observed daily, yielding 6 independent baselines per day.

e) A surveying engineer is expected to be in charge of the data processing and supervision of the project. Baseline mode of processing has been assumed together with a conservative daily progress of three processed baselines.

f) Each baseline is meant to be observed only once unless it is flagged as faulty or unacceptable. Then, it will be repeated. A 20% repetition rate has been assumed.

The GPS network (Figure 4.3) consists of 37 baselines. By adding the 20% of repetitions, a total of 45 baselines to be measured is obtained. Using four receivers and a daily progress of 6 baselines/day gives a total of 8 days to survey the whole network. From similar relationships as those used in section 3.5.1, the cost of a GPS crew per day (8 hours) will be given by the simple equation

$$\text{GPS crew/day} = (c_A + c_B + c_I) \times 8 .$$

Using the same cost rates as in 3.5.1 the total cost of the GPS crew will be 3600 Bs/day, which is equivalent to Cdn $298./day.

Processing and supervision will take a total of 16 man days at a rate of 2000 Bs/day. The total cost of one GPS campaign is then distributed as follows:

Cost Distribution GPS Campaign

Field Work		
4 GPS crews x 8 days x 3600 Bs/day		115,200.00 Bs
4 GPS receivers x 8 days x 1740 Bs/day		55,680.00 Bs
	Subtotal	170,880.00 Bs

Data Processing and Supervision

16 days x 2000 Bs/day		32,000.00 Bs
	Subtotal	32,000.00 Bs
	TOTAL COST	202,880.00 Bs

which is equivalent to Cdn $16790.00.

For the cost of the reference campaign to be held in March 1990, assuming the receivers will not be purchased then, a rental rate of Cdn $360./receiver/day or equivalent to 4350 Bs/receiver/day has been considered appropriate. Furthermore, since all day coverage will not be possible, an estimated daily progress of 3 baselines/day using four receivers may also be used. On this basis the cost of the reference campaign will be estimated as follows:

Cost of GPS Reference Campaign (1990)

Field Work

4 GPS crews x 16 days x 3600 Bs/day		230,400.00 Bs
4 GPS receivers x 16 days x 4350 Bs/day		278,400.00 Bs
	Subtotal	508,800.00 Bs

Data Processing and Supervision

16 days x 2000 Bs/day		32,000.00 Bs
	Subtotal	32,000.00 Bs
	TOTAL COST	540,800.00 Bs

which is equivalent to Cdn $44800.00.

Finally, the cost of the redesigned levelling survey must be estimated. From Table 3.1 one can easily infer that 236.2 km pertaining to connecting lines are eliminated together with one half of the total number of kilometres corresponding to the first order levelling lines (i.e. 389.5 km). As a result 250 bench marks corresponding to the connecting lines have been also eliminated in the new design. The number of days estimated for data processing has been cut down to 180 since the new procedure and computational scheme has been optimized. On this basis, the cost of the redesigned levelling survey is as follows:

Cost of Redesigned Levelling Survey

Field Work		
Cost of Levelling		
843.3 km x 1200 Bs/km		1,011,960 Bs
Cost of Maintenance		
1374 BM's x 469 Bs/BM		644,406 Bs
	Subtotal	1,656,366 Bs
Data Processing and Supervision		
180 days x 2000 Bs/day		360,000 Bs
	Subtotal	360,000 Bs
	TOTAL COST	2,016,366 Bs

which is equivalent to Cdn.$166,871.70.

The total cost of an integrated GPS - levelling campaign is then 2,219,246 Bs or Cdn.$183,661 which, when compared to the original cost estimate of section 3.5.4 (3,004,456 Bs), yields a difference of 785,210 Bs or a 26% savings over the original cost. Considering a total investment cost of 660,800 Bs (i.e. initial investment + cost of reference campaign) one can easily infer that for the campaign of 1992, there will be still a savings of 124,410 Bs, equivalent to 4% of the total cost. This means that in the campaign of 1992, the initial investment cost for the subsidence applications will be completely amortized and thereafter a net savings of 26% will be obtained. Note that the major contribution in this cost analysis arises from the redesigned monitoring scheme, where savings in levelling are involved, and not from the costs of the GPS campaigns. Therefore, the inaccuracies regarding the assumptions that lead to estimate the cost of the GPS campaigns do not affect significantly the whole cost analysis. This can be easily seen by taking for instance a 50% improvement in the above cost estimates for GPS. The results indicate only a 3% effect on the overall savings estimate. On the other hand, considering the possibility that as the accuracy of GPS improves in the future, one could lower the accuracy standards for levelling so that the present field daily progress could increase, say from 6 to 7 km/day. The new estimates show a significant increase from 26% to 38% in the overall estimated savings. This demonstrates the economic potential of GPS in this particular application. The estimates so far encompass only the inland subsidence monitoring scheme. Additional savings may be expected in the offshore subsidence monitoring methodology by using the "stop and go" technique [Remondi, 1985].

6. CONCLUSIONS AND RECOMMENDATIONS

(a) The subsidence monitoring studies at the Costa Bolivar required the selection of a kinematic deformation model. The selection of the model was bound by the irregular shape of the subsidence basins in the area, and the need for utmost accuracy. As a result, a discrete point modelling approach was selected, within which a general point velocity model was justified at an initial stage. The possibility of future modifications to the initially proposed model must be seriously considered within a multi-epoch solution extending beyond a time span of 10 years.

(b) The integration of GPS and levelling required the homogenization of the field observations by removing the datum incompatibility problem. It was shown theoretically that, at least in the short run, it may be safe to neglect any temporal variations of the geoid, so that the parameters in the geoidal model can be assumed as constant within the integrated solution. The apparent irregular shape of the geoid in the area and the need for high accuracy lead again to select the discrete modelling approach. Thus, depending on whether a multi-epoch or an observation differences approach to the least squares solution is used, a point constant geoid height parameter may be explicitly included in the observation equations or excluded completely if observation differences are taken. It is recommended to maintain a close check on the behaviour of the estimated geoidal heights for the points where common GPS and levelling observations are to be collected in each campaign in order to confirm the above findings.

(c) The tools of the UNB Generalized Method proved also their validity in the accuracy evaluation and analysis of the levelling and GPS data of the Costa Bolivar oil fields. The general accuracy (standard deviation) of the levelling surveys in the primary levelling network was estimated through the MINQE technique to be 2 mm$\sqrt{k}$ for the double run main levelling lines and 4 mm$\sqrt{k}$ for the single run densification lines.

This worse than expected accuracy is most likely due to deviations from the field standards and failure to apply corrections for gravity and other possible effects. It is recommended to increase the field supervision in order to enforce the close observation of the standard procedures. The performance of a gravity survey in the area to provide the corresponding corrections must be considered.

(d) The computational procedure followed by Maraven in the computation of the datum lines was proven, by the application of the weighted similarity transformation, to introduce systematic shifts in the elevations of the stable reference bench marks which are used as constraints in the network adjustment. This has resulted in an accuracy deterioration of about 20% in the three most recent campaigns. The systematic effect, when added to the random error component, results in a total uncertainty of 20 to 30 mm at the 95% confidence level for the subsidence determination, and of 15 to 20 mm at the 95% confidence level for the absolute elevations. These values, although adequate for purposes of Maraven, are worse than expected. The designed methodology will certainly eliminate this source of error by the application of an appropriate point stability analysis.

(e) The standard error of the GPS derived height difference was evaluated to be equal to 29 mm, independent of baseline length when using the baseline mode processing technique. The poor geometry reflected in VDOP values of the order of 4.5 to 5.0, combined with tropospheric effects, are most likely to be the reasons for the results being poorer than expected. A very valid suggestion may be to adopt the network adjustment mode for future GPS data processing. Significant improvements in accuracy may be expected. The future offers a much better scenario regarding the accuracy of GPS observations. A conservative prediction based solely on the expected geometric improvement indicates an increase by a factor of at least two times the present accuracies which, when translated to the Costa Bolivar area, means improvements to about 15 mm. In the evaluation of the accuracy standards, however, this level of accuracy was shown to cause a deficiency in the accuracy of the subsidence of the order of 17% (5 mm at a 95% confidence level) when compared to levelling. Nevertheless, if considering the limitation of the Maraven computational approach discussed above, the deficiency in accuracy introduced by the relative GPS accuracy of 15 mm will be offset by the introduction of the new computational scheme. Therefore, it is confidently expected that the accuracy of the integration approach designed here will be in the near future, as the full GPS constellation becomes operational, equivalent to the level of accuracy presently obtained for the subsidence at the Costa Bolivar oil fields.

(f) The economic analysis has shown that the application of the designed integration approach will bring savings in the order of 26% in the total cost of one inland campaign. These savings, however, do not pose any sacrifice in accuracy, provided the accuracy of the GPS height differences reaches at least the expected standard error of 15 mm. For the future, if the accuracy of GPS surveys becomes even higher, further economic benefits may be expected by tolerating lower order levelling surveys which will certainly speed up the daily field progress. Savings of the order of 38% may be expected if the average daily field progress increases from 6 to 7 km/day. Additionally, if the kinematic methods "stop and go" become compatible in accuracy with the adopted levelling standards, further economic benefits will arise from the possible replacement of levelling by GPS.

(g) With respect to the field surveys, it is recommended to design an algorithm based on the average estimated velocity of each BM and the observed height differences in previous years in order to have a field blunder check that would indicate the immediate repetition of sections which are out of tolerance. This is particularly valid in the new design where only single run levelling lines are to be observed.

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APPENDIX I

(GPS and Levelling Data)

Table. I.1. Derived GPS and Field Levelling
Data from Campaign No. 1.

FROM	TO	ΔH*(lev) (m)	ΔH (GPS) (m)	DATE
GPS1	743	-44.110	-44.002	31/03/87
743	184A	-26.364	--	1/4/87
GPS1	202	--	-44.054	1/4/87
GPS3	324	-12.934	--	5/4/87
743	GPS3	-39.465	-39.358	31/03/87
GPS3	TJ1B	-10.945	-10.857	31/03/87
202	184A	-26.259	-26.307	1/4/87
184A	GPS5	-25.267	-25.307	1/4/87
184A	324	-26.041	-26.041	3/4/87
324	TJ1B	1.991	2.042	3/4/87
GPS5	324	-0.764	-0.745	3/4/87

* reduced to the GPS date.

Table I.2. Derived GPS and Field Levelling
Data From Campaign No. 2

FROM	TO	ΔH* (Lev) (m)	ΔH (GPS) (m)	DATE
GPS1	743	-44.109	-44.016	29/10/87
GPS1	202	--	-44.058	1/11/87
743	184A	-26.349	-26.268	1/11/87
202	184A	-26.254	-26.296	29/10/87
743	GPS3	-39.475	-39.366	2/11/87
184A	GPS5	-25.305	-25.346	28/10/87
184A	BM324	-26.045	-26.046	30/10/87
184A	GPS3	-13.128	-13.182	2/11/87
GPS3	TJ1B	-10.951	-10.874	30/10/87
GPS3	324	-12.927	-12.922	2/11/87
GPS5	324	-0.741	-0.627	28/10/87
324	TJ1B	1.975	2.012	30/10/87

*Reduced to the GPS date

Table I.3. Derived GPS and Field Levelling
Data From Campaign No. 3 (Tia Juana section only)

FROM	TO	ΔH* (lev) (m)	ΔH (GPS) (m)	DATE
GPS1	202	-44.211	-44.015	17/04/88
GPS1	743	-44.119	-43.970	17/04/88
743	184A	-26.346	-26.246	20/04/88
202	184A	-26.254	-26.338	17/04/88
743	GPS3	-39.490	-39.416	19/04/88
184A	GPS5	-25.328	-25.290	28/04/88
184A	324	-26.060	-26.004	28/04/88
GPS5	324	-0.724	-0.710	28/04/88
GPS3	324	-12.916	-12.900	23/04/88
GPS3	TJ1B	-10.957	-10.937	19/04/88
324	TJ1B	1.960	2.000	23/04/88
TJ1B	TJ14B	-0.979	-0.966	29/04/88

*Reduced to the GPS Date.

APPENDIX II

(Formula Derivation)

DERIVATION OF THE GEOID VARIATIONS FORMULA AS A FUNCTION OF GRAVITY VARIATIONS

The principles and methodology used within this derivation have been taken from Vaníček et al. [1980] and Vaníček and Krakiwsky [1982].

Consider the average gravity anomaly $\Delta\,\bar{g}$ within a spherical distance ψ to be obtained from

$$\Delta\bar{g}(\psi) = \frac{1}{2\pi}\int_{o}^{2\pi} \Delta g(\psi, \alpha)\, d\alpha \qquad \text{(II.1)}$$

By substitution of Δg by $\Delta\,\bar{g}$ in Stokes formula (5.9) one integral vanishes and the Stokes formula becomes

$$N = \frac{R}{2g}\int_{o}^{\pi} \Delta\bar{g}(\psi)\, S(\psi) \sin\psi\, d\psi \qquad \text{(II.2)}$$

For small changes in the geoidal height as a function of changes in the gravity anomaly the above expression becomes

$$\delta N \approx \frac{R}{2g}\int_{o}^{\pi} \delta\Delta\bar{g}(\psi) S(\psi) \sin\psi\, d\psi \qquad \text{(II.3)}$$

Making use of the integration by parts technique whereby $\mu = \delta\,\bar{\Delta}g(\psi)$ and $v = \int S(\psi)\sin\psi\, d\psi$ and considering only the changes to occur within a small area $\psi < 10°$, 'v' may be approximated by 2.3 ψ (ψ in radians), based on tabulated values given in Lambert and Darling [1936].

Since $\delta\Delta\,\bar{g}(\psi)$ corresponds to the mean change of the gravity anomaly within the spherical distance ψ_{max} so that $\delta\Delta\,\bar{g}(\psi) = 0$ for $\psi > \psi_{max}$ the first term in the integration by parts vanishes and one obtains

$$\delta N \approx -\frac{R}{2g}\int_{o}^{\psi_{max}} \frac{\partial\delta\Delta\bar{g}(\psi)}{\partial\psi}\, 2.3\psi\, d\psi \qquad \text{(II.4)}$$

If variations in the gravity anomaly are taken as a function of mass displacements only, $\delta\Delta\,\bar{g}$ and (II.4) becomes

$$\delta N \approx -7.4 \int_{o}^{\psi_{max}} \frac{\psi \partial \delta \bar{g}(\psi)}{\partial \psi} d\psi \ . \qquad \text{(II.5)}$$

Assuming the gravity varies according to a conical model (shown in Figure II.1) which can be represented mathematically by

$$\delta \bar{g}(\psi) = \delta g_{max} - \delta g_{max} \frac{\psi}{\psi_{max}} \qquad \text{(II.6)}$$

(The gravity variation decreasing linearly, from a maximum at $\psi = 0$ to a minimum at ψ_{max}).

then

$$\frac{\partial \delta \bar{g}(\psi)}{\partial \psi} = \frac{\delta g_{max}}{\psi_{max}} \ . \qquad \text{(II.7)}$$

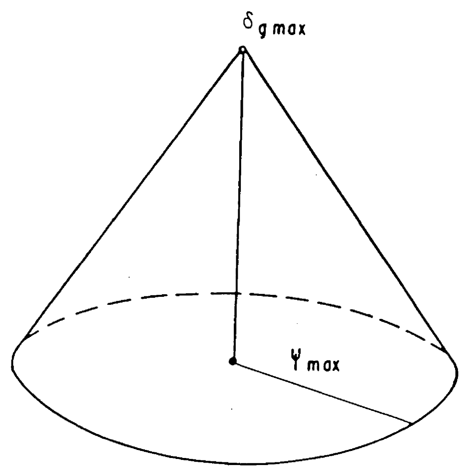

Figure II.1. Conical Model.

Then by substitution of equation (II.7) into equation (II.5) one obtains

$$\delta N \approx -7.4 \int_{o}^{\psi_{max}} \psi \frac{\delta g_{max} d\psi}{\psi_{max}} \qquad \text{(II.8)}$$

which, after the evaluation of the integral, becomes

$$\delta N \approx -7.4 \frac{\psi^2}{2} \frac{\delta g_{max}}{\psi_{max}} \Bigg|_{o}^{\psi_{max}} \tag{II.9}$$

and after simplifications yields the final expression relating the gravity variations to the changes in geoidal heights as:

$$\delta N \approx -3.7 \text{ [m mGal}^{-1}\text{]}\ \delta g_{max} \psi_{max} \tag{II.10}$$

where δg_{max} is the change in gravity due to density variations in milligals and ψ_{max} is the radius of the cap in radians.

APPENDIX III

(Integration Example)

Consider two campaigns of the levelling network shown in Figure III.1. The GPS baselines (straight lines) and levelling lines are shown for each campaign with their corresponding epoch of observation t_i which, for practical purposes, may be equivalent to the date of the observation. Thus, t_1 will be day one, t_2 will be day two and so on.

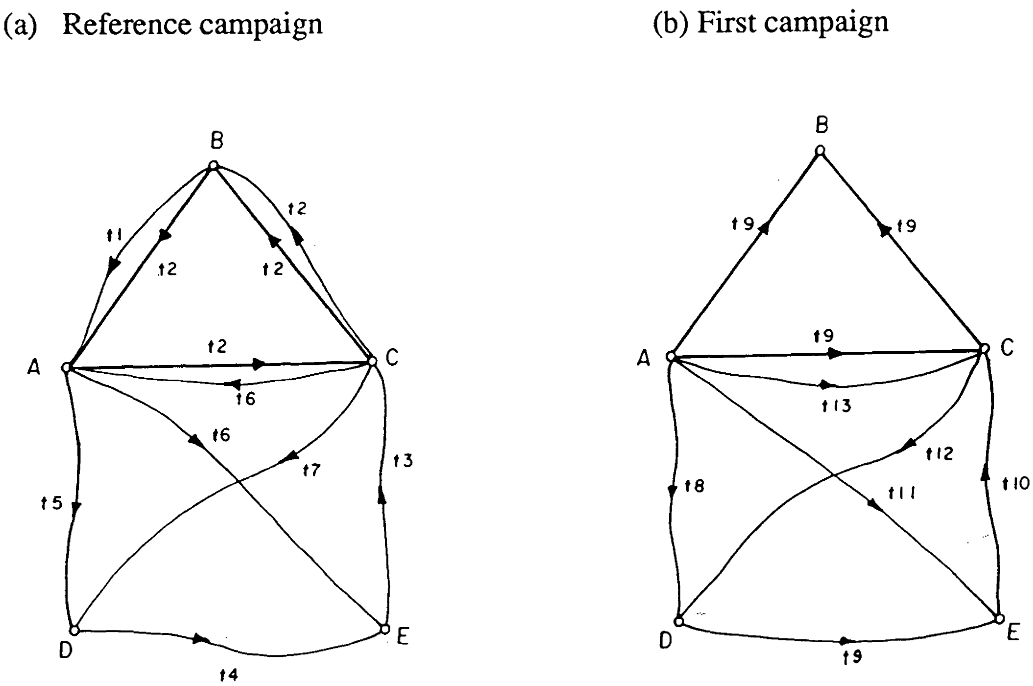

Figure III.1. Sample Levelling Network.

Consider also a local coordinate system x, y with origin at point B and the equation of a plane to approximate the geoidal height differences between two points i and j as:

$$\Delta N_{ij} = (x_j - x_i)a + (y_j - y_i)\mathbf{b} \qquad \text{(III.1)}$$

If a set of homogeneous heights is also desired, the observation equations for the integration using the point velocity model equation (2.7) and the plane model are as follows:

Reference campaign.

levelling:

$$\Delta H_{BA}(t_1) + V_{BA}(t_1) = H_A(t_o) - H_B(t_o) + (t_1-t_o)(\dot{H}_A-\dot{H}_B)$$

$$\Delta H_{AD}(t_5) + V_{AD}(t_5) = H_D(t_o) - H_A(t_o) + (t_5-t_o)(\dot{H}_D-\dot{H}_A)$$

$$\Delta H_{DE}(t_4) + V_{DE}(t_4) = H_E(t_o) - H_D(t_o) + (t_4-t_o)(\dot{H}_E-\dot{H}_D)$$

$$\Delta H_{EC}(t_3) + V_{EC}(t_3) = H_C(t_o) - H_E(t_o) + (t_3-t_o)(\dot{H}_C-\dot{H}_E)$$

$$\Delta H_{CA}(t_6) + V_{CA}(t_6) = H_A(t_o) - H_C(t_o) +(t_6-t_o)(\dot{H}_A-\dot{H}_C)$$

$$\Delta H_{CD}(t_7) + V_{CD}(t_7) = H_D(t_o) - H_C(t_o) + (t_7-t_o)(\dot{H}_D-\dot{H}_C)$$

$$\Delta H_{AE}(t_6) + V_{AE}(t_6) = H_E(t_o) - H_A(t_o) + (t_6-t_o)(\dot{H}_E-\dot{H}_A)$$

$$\Delta H_{CB}(t_2) + V_{CB}(t_2) = H_B(t_o) - H_C(t_o) + (t_2-t_o)(\dot{H}_B-\dot{H}_C)$$

GPS:

$$\Delta h_{BA}(t_2) + V_{hBA}(t_2) = H_A(t_o) - H_B(t_o) + (a\Delta X_{BA} + b\Delta Y_{BA}) +(t_2-t_o)(\dot{H}_A-\dot{H}_B)$$

$$\Delta h_{CB}(t_2) + V_{hCB}(t_2) = H_B(t_o) - H_C(t_o) + (a\Delta X_{CB} + b\Delta Y_{CB}) + (t_2-t_o)(\dot{H}_B-\dot{H}_C)$$

$$\Delta h_{AC}(t_2) + V_{hAC}(t_2) = H_C(t_o) - H_A(t_o) + (a\Delta X_{AC} + b\Delta Y_{AC}) + (t_2-t_o)(\dot{H}_C-\dot{H}_A)$$

<u>Campaign No. 1</u>

Levelling:

$$\Delta H_{AD}(t_8) + V_{AD}(t_8) = H_D(t_o) - H_A(t_o) + (t_8-t_o)(\dot{H}_D - \dot{H}_A)$$

$$\Delta H_{DE}(t_9) + V_{DE}(t_9) = H_E(t_o) - H_D(t_o) + (t_9-t_o)(\dot{H}_E - \dot{H}_D)$$

$$\Delta H_{EC}(t_{10}) + V_{EC}(t_{10}) = H_C(t_o) - H_E(t_o) + (t_{10}-t_o)(\dot{H}_C - \dot{H}_E)$$

$$\Delta H_{AC}(t_{13}) + V_{AC}(t_{13}) = H_C(t_o) - H_A(t_o) + (t_{13}-t_o)(\dot{H}_C - \dot{H}_A)$$

$$\Delta H_{CD}(t_{12}) + V_{CD}(t_{12}) = H_D(t_o) - H_C(t_o) + (t_{12}-t_o)(\dot{H}_D - \dot{H}_C)$$

$$\Delta H_{AE}(t_{11}) + V_{AE}(t_{11}) = H_E(t_o) - H_A(t_o) + (t_{11}-t_o)(\dot{H}_E - \dot{H}_A)$$

GPS:

$$\Delta h_{AB}(t_9) + V_{hAB}(t_9) = H_B(t_o) - H_A(t_o) + (a\Delta X_{AB} + b\Delta Y_{AB}) + (t_9-t_o)(\dot{H}_B - \dot{H}_A)$$

$$\Delta h_{AC}(t_9) + V_{hAC}(t_9) = H_C(t_o) - H_A(t_o) + (a\Delta X_{AC} + b\Delta Y_{AC}) + (t_9-t_o)(\dot{H}_C - \dot{H}_A)$$

$$\Delta h_{CB}(t_9) + V_{hCB}(t_9) = H_B(t_o) - H_C(t_o) + (a\Delta X_{CB} + b\Delta Y_{CB}) + (t_9-t_o)(\dot{H}_B - \dot{H}_C)$$

The matrix equation (5.17) may then be written as:

$$\begin{bmatrix} \Delta H_{BA}(t_1) \\ \Delta H_{AD}(t_5) \\ \Delta H_{DE}(t_4) \\ \vdots \\ \Delta h_{BA}(t_2) \\ \Delta h_{CB}(t_2) \\ \Delta h_{AC}(t_2) \\ \Delta H_{AD}(t_8) \\ \vdots \\ \Delta h_{CB}(t_9) \end{bmatrix} + \begin{bmatrix} V_{BA}(t_1) \\ V_{AD}(t_5) \\ V_{DE}(t_4) \\ \vdots \\ V_{hEA}(t_2) \\ V_{hCB}(t_2) \\ V_{hAC}(t_2) \\ V_{AD}(t_8) \\ \vdots \\ V_{hCB}(t_9) \end{bmatrix} = \begin{bmatrix} A_1 & 0 & B_1 \\ A_2 & 0 & B_2 \\ A_3 & 0 & B_3 \\ \vdots & & \\ A_8 & Q_1 & B_8 \\ A_9 & Q_2 & B_9 \\ A_{10} & Q_3 & B_{10} \\ A_{11} & 0 & B_{11} \\ \vdots & & \\ A_{19} & Q_6 & B_{19} \end{bmatrix} \begin{bmatrix} \xi \\ \varepsilon \\ c \end{bmatrix}$$

where

$$A_1 = [-1 \;\; 1 \;\; 0 \;\; 0 \;\; 0] \qquad Q_1 = [\Delta X_{BA} \;\; \Delta Y_{BA}]$$

$$A_2 = [-1 \;\; 0 \;\; 0 \;\; 1 \;\; 0] \qquad Q_2 = [\Delta X_{CB} \;\; \Delta Y_{CB}]$$

$$\widetilde{B}_1 = [(t_1 - t_o) \quad (t_1 - t_o) \quad 0 \quad 0 \quad 0]$$

$$\widetilde{B}_2 = [-(t_1 - t_o) \quad 0 \quad 0 \quad (t_1 - t_o) \quad 0] \quad \text{and so on.....}$$

and

$$\xi = \begin{bmatrix} H_A(t_o) \\ H_B(t_o) \\ H_C(t_o) \\ H_D(t_o) \\ H_E(t_o) \end{bmatrix} \qquad \varepsilon = \begin{bmatrix} a \\ b \end{bmatrix} \qquad \mathbf{c} = \begin{bmatrix} \dot{H}_A \\ \dot{H}_B \\ \dot{H}_C \\ \dot{H}_D \\ \dot{H}_E \end{bmatrix}$$

The solution can be obtained by simple least squares estimation. To avoid singularity, one point with zero velocity must be held fixed unless the technique of free network solution is used.

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