The Geodetic Surveying Methods in the Monitoring of Large Dams in Portugal

Dr. João CASACA and Dr. Maria João HENRIQUES, Portugal

Key words: Geodetic Surveys, GPS, Monitoring of Displacements, Safety Control of Dams, Triangulation Networks.

ABSTRACT

Conventional terrestrial geodetic surveying methods, such as precision triangulation, traversing and geometric levelling, have been present at the displacement monitoring systems of the majority of the Portuguese concrete and masonry large dams, since the 1940's. In spite of the widespread use of displacement measurement instruments, such as pendulum and rockmeters, geodetic surveying methods continue to be used in the displacement monitoring systems because they provide: i) An independent control of the other methods; ii) A global relation between the dam, its foundation and the surrounding terrain. The Geodetic Measurement Division (GMD) of the National Laboratory for Civil Engineering, which is involved in dam surveillance activity since the 1950's, is responsible for planning and observation of geodetic surveying systems for more than fifty large dams. The paper presents the experience of the GMD in planning the geodetic surveying systems and in the quality control of the observations, and carries on a prospective view of modern trends, such as motorized tachymeters and GPS relative measurements and permanent stations.

CONTACT

Doctor Dipl. Engineer João Casaca
Department of Dams
National Laboratory for Civil Engineering (NLCE)
Av. do Brasil 101
1700 066 Lisbon
PORTUGAL
E-mail: jcasaca@lnec.pt

Doctor Dipl. Engineer Maria João Henriques
National Laboratory for Civil Engineering (NLCE)
Av. do Brasil 101
1700 066 Lisbon
PORTUGAL
E-mail: mjoao@lnec.pt

The Geodetic Surveying Methods in the Monitoring of Large Dams in Portugal

Dr. João CASACA and Dr. Maria João HENRIQUES, Portugal

1. INTRODUCTION

Engineering works, such as large dams, need a careful safety control along its life span, since its failure might cause great losses in lives and property. The safety control of a large dam lies on the analysis of its structural behaviour, based on monitoring a large set of variables which describe the relations between the actions (gravity, temperature, hydrostatic pressure, etc.) and the corresponding structural responses (stresses, displacements, etc.), taking into account the properties of the materials used in the construction (concrete, embankment, masonry, etc.). The displacements of a discrete and representative set of points on the structure, its foundations and surrounding terrain, are fundamental control variables to structural behaviour analysis of large dams.

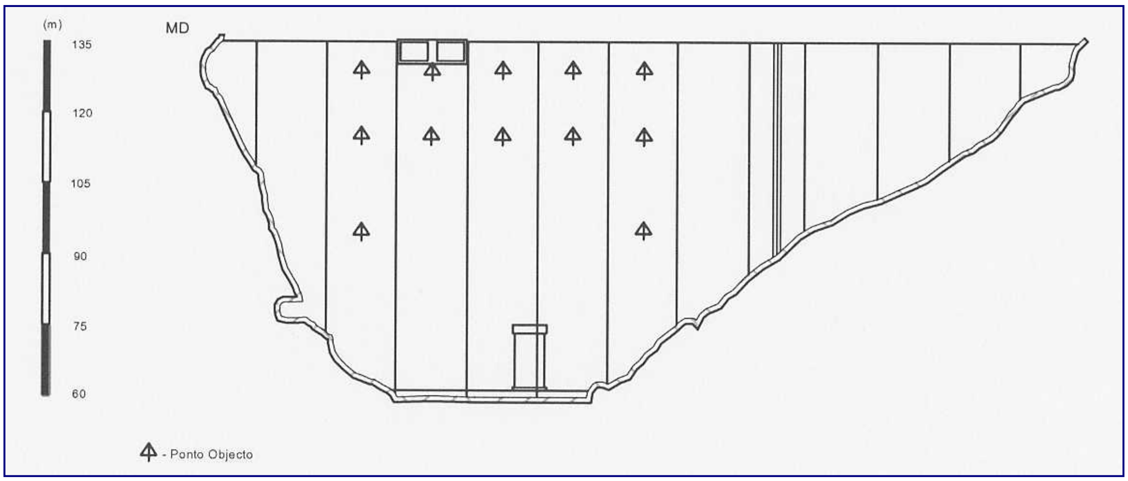

Figure 1 – Object points on the downstream wall of Santa Luzia dam.

Since the beginning of the XXth century, geodetic surveying methods, such as precision triangulation and precision geometric levelling have been widely used to displacement monitoring at large dams. Factors such as the necessity of favourable atmospheric conditions and specialized personnel, the long duration of the measurements and computing procedures led to the introduction, specially at concrete dams, of other displacement monitoring methods such as inverted pendulum and extensometric bars (rockmeters). The modern measurement equipment, such as electronic motorized tachymeters and numerical levels, with automatic reading and recording features, has overcome some of the referred handicaps of the traditional geodetic surveying methods, increasing its speed, efficiency and reliability.

Actually, at concrete dams, geodetic surveying methods, specially planimetric methods, are used as a backup to other horizontal displacement monitoring methods. At embankment dams, which do not have vertical shafts for pendulum, the geodetic surveying methods are the exclusive source of horizontal displacements. The altimetric methods kept its importance to measure absolute vertical displacements on the crest of the dams or to control and relate the information provided by extensometric bars installed in foundation galleries. The great advantage of the geodetic surveying methods consists of providing information on the relation of the dam with its foundation and the surrounding terrain.

2. THE PORTUGUESE CASE

Geodetic methods were used for the first time in Portugal during Santa Luzia dam (Figure 1) first filling, in 1942, and have been applied, since then, to most of the portuguese concrete and masonry large dams. The Geodetic Measurements Division (GMD) of the Dam Department (DD) of the National Laboratory for Civil Engineering (NLCE), which was created in 1955, is responsible for designing the geodetic surveying systems and carrying out measurements of a majority of the portuguese concrete and masonry large dams and of a significant number of embankment dams.

Figure 2 – Concrete Pillar with foundation.

According to the International Committee on Large Dams (ICOLD), a dam is said to be a large dam when its height, from the foundation, is 15 meters or more. Dams which height is in the 5 15 meters range and have a reservoir volume of more than three million cubic meters are also classified as large dams. In Portugal there are 191 large dams, from which 40% are concrete or masonry dams and 60% are fill dams. The Geodetic Measurement Division has been involved in the planning and measurement of the geodetic surveying systems of 63% of the portuguese concrete and masonry large dams and 15% of the fill dams.

The geodetic observation systems of concrete and masonry dams are generally composed of independent planimetric and altimetric systems. The planimetric systems consist usually on small triangulation networks, on the downstream banks, and precision traverses, on the crest or in inspection galleries. The altimetric systems consist on precise geometric levelling lines on the crest or in inspection galleries. In the case of embankment dams, the geodetic observation systems consist on threedimensional networks, which combine tachymetry and geometric levelling (Casaca, 1987b).

The station points of the planimetric systems are concrete pillars with forced centring pieces, thermic isolation and a large concrete foundation on rock (Figure 2). The reference points for the planimetric and altimetric systems are carefully chosen, far from the dam's influence zone, and materialized by concrete pillars well founded on rock. Sometimes the reference points are controlled with inverted pendulum or extensometric bars purposely installed in its neighbourhood.

The standard measurement equipment consists of: i) Automatic optical levels, with parallel plate micrometers and staves with invar scales, for concrete and masonry dams; ii) Precise numerical levels and staves with code bar invar scales, for embankment dams; iii) Precision electronic theodolites equipped with EDM. More recently, motorized tachymeters with precision EDM, have been introduced in precision traversing (Figure 3). The GPS is not yet a standard method for displacement monitoring in Portugal, although several experiments are actually in the planning stage.

Figure 3 – Motorized tachymeter (Leica – TCA2003).

3. MONITORING DISPLACEMENTS WITH THE GPS

The horizontal uncertainty of the GPS may reach standard deviations in the range between 1mm and 2mm, provided the measurements are made with special antennae, in static mode and postprocessment with precision ephemmeris. The vertical uncertainty of the GPS is two to three times larger than the horizontal uncertainty, and turns the GPS unsuitable for vertical displacement monitoring at concrete dams.

The use of the GPS to displacement monitoring at large dams has two important drawbacks: the necessity of placing the antenna on the object points and the necessity of a large conical sector of visibility over the horizon. At concrete arch dams, the object points are not accessible at all. Object points inside inspection galleries are out of reach. Only object points in the crest will be able to be positioned in good conditions. The use of GPS is more interesting in the case of the embankment dams, where the object points, materialized by small concrete pillars, are generally accessible and the tolerances for the errors are larger.

The great advantage of GPS is to provide connection of the object points on the dam and its foundation to reference points located far away with no need of mutual visibility nor ancillary intermediate points. Another important feature of the GPS is that it may be used in combination with other methods such as triangulation and traversing.

Another important feature of the GPS is the possibility of the installation of a few permanent stations, at well protected places, to almost real time monitoring of displacements. The measurements may be processed in kinetic, fast static or static modes, with increasing accuracy. The GPS permanent stations at large dams provide a very efficient safety control tool, whenever the risk justifies its installation.

4. THE FUNCTIONAL MODEL

The functional model relates displacements of the object, ancillary and reference points to the observable variables (horizontal and vertical angles, orthometric height differences, cartographic distances, cartographic coordinates measured with the GPS, etc.) which are often called the observables.

The functional model is a set of linearized observation equations which results from the application of Taylor's theorem (taylorization) to the non linear relations between displacements and variations of the observables, according to the method of the variation of coordinates (Casaca, 2000).

The observation equation, correspondent to an oriented horizontal angle A_{OST}, with origin (O), station (S) and target point (T), is:

$$dA_{OST} \approx \frac{N_{ST}}{c_{ST}^2} dE_T - \frac{E_{ST}}{c_{ST}^2} dN_T + \left[\frac{N_{SO}}{c_{SO}^2} - \frac{N_{ST}}{c_{ST}^2} \right] dE_S +$$

$$+ \left[\frac{E_{ST}}{c_{ST}^2} - \frac{E_{SO}}{c_{SO}^2} \right] dN_S - \frac{N_{SO}}{c_{SO}^2} dE_O + \frac{E_{SO}}{c_{SO}^2} dN_O$$

where: i) E and N stand for Eastings and Northings, relative to a conformal cartographic projection, such as the Gauss Krüger or the Lambert (conical) projections; ii) c stands for the cartographic distance between points; iii) E_{ST}, etc., are the coordinate differences $E_T - E_S$.

The observation equations, correspondent to the cartographic distance c_{ST} between the station (S) and the target point (T), is:

$$dc_{ST} \approx \frac{E_{ST}}{c_{ST}} dE_T + \frac{N_{ST}}{c_{ST}} dN_T - \frac{E_{ST}}{c_{ST}} dE_T - \frac{N_{ST}}{c_{ST}} dN_S$$

Given approximate coordinates of the origin, the station and the target points, the observation equations relate small variations of the coordinates (displacements) of the points (dE_T, etc.) to the resulting variation of the observables: horizontal angles dA_{OST}, cartographic distances dc_{ST}, etc.. An adequate set of observation equations leads to a linear system of equations which can be solved in order to obtain the displacements if the variations of the observables are known.

In vector notation the system of (m) observation equations may be represented as a vector equation $A(m,n)\ \Delta X(n,1) = \Delta Y(m,1)$, where A is the matrix of the coefficients of the system, ΔX the vector of the unknown displacements and ΔY the vector of the variations of the observables between two epochs. The matrix A, which elements may be computed with approximate coordinates, even before any field work, is called the first order design matrix (FODM).

An additional problem is that the FODM, even if all possible observables are measured, has a systematic rank deficiency. The solution of this problem lies on the reference points, which displacements are supposed to be either null or measured by independent methods (pendulum, GPS), that provide a set of supplementary equations, the reference equations, that complete the rank of the FODM to the number of columns.

The rank deficiency of the FODM depends of the type of the network (Casaca, 2000). One dimensional networks, have a rank deficiency of one. Two dimensional networks may have different rank deficiencies: the homogeneous horizontal networks, with no distances, have the maximum rank deficiency which is four; if a distance is measured the deficiency reduces to three, etc.. Three dimensional networks have a more complex pattern of rank deficiency, which attains a maximum at seven. It can be stated, in a rough approximation, that: A one dimensional network needs at least one reference point; A two dimensional needs at least two reference points; A three dimensional network needs at least three reference points.

5. THE STOCHASTIC MODEL

The operational mathematical model results from addition of a stochastic model to the functional model, to enable the design and the quality control of the networks that compose the geodetic observation systems.

Figure 4 – Water vapour, resulting from the discharge of the reservoir, affecting the optical path between two station points.

The measurement of the observables (horizontal and vertical angles, distances, height differences, etc.) is affected by observation errors, which arise from several causes, such as the measurement equipment, the operative methods, the atmospheric conditions, the operator's skill, etc.. The relation between the exact value of an observable (μ) e and the measured value (y) may be expressed by (Casaca, 2000):

$$y = \mu + \theta_I + \theta_E + \delta$$

where: i) θ_I stands for systematic errors of instrumental origin, such as the calibration errors; ii) θ_E stands for systematic errors of environmental origin, such as the horizontal and vertical refraction; iii); iii) δ stands for accidental errors. The systematic errors assume the same values when the measurements are carried out in the same instrumental and environmental conditions, unlike the accidental errors, which have a random distribution.

Figure 4, presents an optical sighting from a station point of a triangulation network to a second station point on the opposite bank (within the circle). The water vapour affects the optical path between the two stations (Casaca et alia, 1994) and introduces variations on the horizontal refraction angle (environmental systematic error).

For a given observation equation ($A\ \Delta X = \Delta Y$), the vector of the observables (ΔY) may be regarded as an one sized sample of a multinormal distribution. The stochastic model consists on a null hypothesis (H0) and an alternative hypothesis (HA):

$$H0 \equiv \begin{cases} \Delta Y \in N(\mu, \Sigma) \\ E(\Delta Y) = \mu \\ V(\Delta Y) = \Sigma \end{cases} , \quad HA \equiv \begin{cases} \Delta Y \in N(\mu, \Sigma) \\ E(\Delta Y) = \mu + \theta \\ V(\Delta Y) = \omega^2\, \Sigma \end{cases}$$

where: i) E and V stand for the mathematical expectation and variance operators; ii) $N(\mu, \Sigma)$ stands for the family of the m dimensional normal random vectors with mean vector $\mu(m,1)$ and variance matrix $\Sigma(m,m)$; iii) The m components of the mean vector μ are the unknown exact values of the observables; iv) $\theta(m,1)$ is an unknown vector parameter, the location parameter; v) ω is an unknown positive parameter, the scale parameter.

The location parameter represents uncorrected systematic errors (a non calibrated EDM, etc.). The scale parameter represents a possible degradation of the posterior precision relative to the prior precision. The prior precision is represented by the variance matrix Σ, which is called the second order design matrix (SODM). When a network and its measurement procedures are planned the systematic errors are supposed to be corrected by calibration and adequate measurement procedures, so that the location parameter is null and the scale parameter is one.

To estimate the unknown vector of the displacements (ΔX), the statistical inference theory recommends the use of the best (minimum variance) linear unbiased estimator (BLUE):

$$\Delta\overline{X} = C^{-1} A^T \Sigma^{-1}\, \Delta Y, \quad C = A^T \Sigma^{-1} A$$

The BLUE is a normal random vector, which mean vector and variance matrix, under H0 and HA, are:

$$H0)\begin{cases} E(\Delta\overline{X}) = \Delta X \\ V(\Delta\overline{X}) = C^{-1} \end{cases} \qquad HA)\begin{cases} E(\Delta\overline{X}) = \Delta X + C^{-1}A^{T}\Sigma^{-1}\theta \\ V(\Delta\overline{X}) = \omega^{2}\, C^{-1} \end{cases}$$

The unknown vector of the errors of the displacements, that results from the BLUE:

$$\varepsilon = \Delta\overline{X} - \Delta X$$

is also a normal random vector, which mean vector and variance matrix, under H0 and HA, are:

$$H0)\begin{cases} E(\varepsilon) = \vec{0} \\ V(\varepsilon) = C^{-1} \end{cases} \qquad HA)\begin{cases} E(\varepsilon) = C^{-1}A^{T}\Sigma^{-1}\theta \\ V(\varepsilon) = \omega^{2}\, C^{-1} \end{cases}$$

The vector of the residuals, which is of prime importance to quality control, is:

$$\Delta = A\,\Delta\overline{X} - \Delta Y = (I - U)\Delta Y, \quad U(m,m) = AC^{-1}A^{T}\Sigma^{-1}$$

where U is a very special idempotent matrix. The vector of the residuals (Δ) is a normal random vector, which mean vector and variance matrix (singular), under H0 and HA, are:

$$H0)\begin{cases} E(\Delta) = \vec{0} \\ V(\Delta) = (I - U)^{T}\Sigma^{-1}(I - U) \end{cases} \qquad HA)\begin{cases} E(\Delta) = (I - U)\theta \\ V(\Delta) = \omega^{2}(I - U)^{T}\Sigma^{-1}(I - U) \end{cases}$$

where Rank(V(Δ)) = (m – n).

6. DESIGN CRITERIA

The most important properties of a network (Casaca, 2000), which are quantifiable by means of its FODM (A) and the SODM (Σ) are: i) Accuracy and precision; ii) Robustness; iii) Economy.

The accuracy and precision are associated to the distribution of the vector of the errors of the displacements. The accuracy is quantified by the effect of the location parameter on the norm of the vector of the errors:

$$\|\varepsilon\| = \left\|C^{-1}A^{T}\Sigma^{-1}\theta\right\|$$

so that, under null hypothesis ($\theta = \vec{0}$), the network is accurate. The precision is described by the variance matrix of the errors (Casaca, 1983):

$$V(\varepsilon) = \omega^2 \, C^{-1}$$

where ($\omega = 1$), under the null hypothesis.

The robustness is the property that characterises the resistance of a network to the effect of a few isolated systematic or gross errors (Casaca, 1987a). In a robust network, those errors will affect much more the vector of the residuals than the vector of the displacements. The robustness is quantified by the local redundancy numbers (Teunissen, 1985), which are de diagonal numbers of the matrix:

$$U = AC^{-1}A^{T}\Sigma^{-1}$$

that assume values between 0 and 1. A robust network must have local redundancy numbers that are homogeneous and close to 1.

The economy is the property that characterizes the cost of installation and measurement of a network. An economic network is a network that attains acceptable levels of accuracy, precision and robustness at a minimum cost. Since the improvement of accuracy, precision and robustness requires more expensive equipment, more skilled personnel, more careful measurement procedures and more redundant measurements, those properties improve at the expense of the economy of a network. Joint optimization of precision, robustness and economy may be carried out by means of risk functions (Casaca et alii, 1993) within the scope of the statistical decision theory.

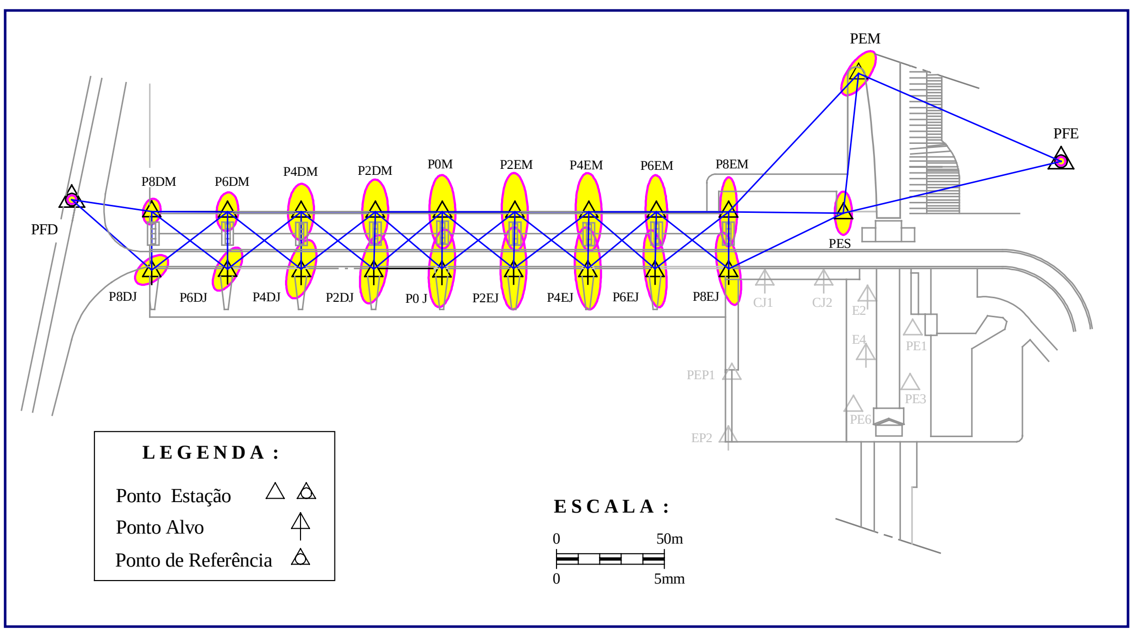

Figure 5 – A heterogeneous (angles and distances) triangulation on the crest of a gravity dam, with the error ellipses for a 0.90 probability level.

The method used in the design of the networks (Casaca, 1988) consists on the choice of an initial configuration, which is improved by an iterative CAD procedure. The FODM of the initial configuration is computed with approximate coordinates extracted from a topographical map. The SODM is postulated as a function of the standard measurement equipment. After analysis of the error ellipses and the local redundancy numbers, the network is accepted or is improved by means of inclusion of new stations and new observables.

Very often, the constraints on the location of the stations and the limitations of the standard equipment do not allow great improvements on the initial configurations. Figure 5 presents a heterogeneous network on the crest of a gravity dam and the error ellipses correspondent to the measurement of angles and distances. The eccentricity of the ellipses is originated by the difference of prior standard deviations assumed for the angles and for the distances. Not very much can be done to decrease de eccentricity of the ellipses else than change the prior relations of the standard deviations.

7. QUALITY CONTROL

The quality control of the measurements is to be carried out by the measurement teams, in the field, before returning to the office. The main tool for this task is a portable computer with network adjustment software, which centralizes the measurements recorded by the PCMCIA of the tachymeters, numerical levels, etc..

The quality control of the measurements is based on the quadratic random variable (v) derived from the vector of the residuals:

$$v = \Delta^T \Sigma^{-1} \Delta$$

which has a chi square type distribution. Under the null hypothesis (H0), v has a central chi square distribution with (m–n) degrees of freedom. Under the alternative hypothesis (HA), v is proportional to a non central chi square distribution, with (m–n) degrees of freedom:

$$\text{H0) } v \in \chi^2 (m-n), \quad \text{HA) } v \in \omega^2 \chi^2 ((m-n); \xi)$$

where m and n are the number of lines and columns of the FODM, and the non centrality parameter (ξ) is a quadratic function of the location vector parameter (θ).

The network is designed under the null hypothesis. After the measurement of the network, a statistical test of the null hypothesis against the alternative hypothesis is carried out. If the null hypothesis is accepted, the measurement team returns to the office. If the null hypothesis is rejected, the measurement team checks and repeats measurements until the null hypothesis is accepted.

The variable of the test (v) is compared to the acceptance (RA) and critical (RC) regions:

RA = [0, q],   RC =] q, +∞ [

where q is the 0.95 quantile of the central chi square distribution with (m–n) degrees of freedom. If v falls into RA, the null hypothesis is accepted. If v falls into RC, the null hypothesis is rejected. The probability of rejecting wrongly the null hypothesis (the significance level of the test), is 0.05, according to the probability level associated to the quantile q. The probability of accepting wrongly the null hypothesis (the power function of the test) is an increasing function of the norm of the location parameter and of the scale parameter.

At robust networks, the efficiency of the quality control may be improved, using the variance component estimation theory, computing posterior standard deviations for each station and testing them against the prior values used at the design stage (Casaca et alia, 1985, 1988).

8. CONCLUSION

Geodetic surveying methods are expected to collaborate in large dam safety control activities for many years to come. At the present, the most important problem, regarding the application of the geodetic surveying methods to displacement monitoring, at large dams, is the improvement of the integration of the GPS with the conventional surveying methods.

At embankment dams the GPS is able to offer a competitive alternative to the conventional surveying methods. However, at concrete and masonry dams, where the tolerances for the errors are smaller, the GPS has a prospectively less important potential, specially to vertical control.

REFERENCES

Casaca, J. (1983), A Precision Criterion for Geodetic Network Design. Memória Nº 615, Ed. LNEC, Lisboa.

Casaca, J. et Henriques, M. J. (1985), Variance Componente Theory and its Application to Network Analysis. Memória Nº 653, Ed. LNEC, Lisboa.

Casaca, J. (1987a), A Reliability Criterion for Geodetic Network Design. Memória Nº 682, Ed. LNEC, Lisboa.

Casaca, J. (1987b), Redes Geodésicas Locais para a Determinação de Deslocamentos em Barragens de Aterro. Geotecnia, Nº 50.

Casaca, J. (1988), Computer Aided Design of Local Threedimensional Networks with Heterogeneous Observables. Revista do IGC, Nº 7.

Casaca, J. et Henriques, M. J. (1988), Variance Component Estimation and Reliability at Local Heterogeneous Networks. Proceedings of the IAG International Symposium on Integrated Geodesy, Sopron.

Casaca, J.; Henriques, M. J. et Matos, J. L. (1993), "Risk Analysis of Local Heterogeneous Networks" in "Applications of Geodesy to Engineering". IAG Symposia 108 (pp. 74 81), Springer, Berlin.

Casaca, J. et Henriques, M. J. (1994), Numerical Modelling of Atmospheric Refraction. Proceedings of the "International Symposium on Deformations", Istanbul.

Casaca, J. (2000), O Método da Variação de Coordenadas na Observação Geodésica de Barragens. LNEC, Série ICT, ITB 21, Lisboa.
Teunissen, P. (1985), “Quality Control in Geodetic Networks” in “Optimization and Design of Geodetic Networks”. Springer Verlag, Berlin.

BIOGRAPHICAL NOTES

Doctor Dipl. Engineer **João Casaca** is a Senior Research Officer of National Laboratory for Civil Engineering (NLCE), Head of the Geodetic Measurement Division of the Dams Department of NLCE. João Casaca is also invited Professor of Topographical Sciences at Lisbon Technical University.

Doctor Dipl. Engineer **Maria João Henriques** is a Research Officer of NLCE, working at the Geodetic Measurement Division.