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Using Global Positioning System techniques in landslide monitoring

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Abstract

The precise determination of point coordinates with conventional Global Positioning System (GPS) techniques often required observation times of one to several hours. In the last few years, new GPS methods have been developed (among them, the fast-static and real time kinematic), with higher productivity and good theoretical precision. The main objective of this paper is to ascertain the performance of these methods in landslide monitoring practice. We present, first of all, the basic principles of the GPS, the equipment and working procedures. We discuss afterwards the applicability of the GPS to the monitoring of landslide surface displacements. Compared with the classical surveying methods, the GPS allows a larger coverage and productivity with similar accuracy. Furthermore, it can work in all kinds of weather conditions and a direct line of sight between stations is not required. Finally, we present an example of the performance of the GPS equipment in the landslide of Vallcebre, Eastern Pyrenees (Spain). This landslide has been periodically monitored since 1987 with terrestrial photogrammetry and geodetic measurements [theodolite, electronic distance metres (EDM)]. The movement extends over an area of 0.8 km^2 and has experienced displacements as large as 1.6 m during the period 1996–1997. 14 campaigns, over a period of 26 months, using both static and kinematic GPS methods have been carried out. The GPS measurements have been compared with the results obtained with the EDM, inclinometers and wire extensometers, and checked against fixed stable points. The precision achieved with the GPS measurements is 12 to 16 mm in the horizontal plane and 18 to 24 mm in elevation.

Keywords: Eastern Pyrenees; GPS; Landslides; Monitoring

1. Introduction

The assessment of landslide behaviour is usually undertaken by means of monitoring. Very often, the measurement of superficial displacements is the simplest way to observe the evolution of a landslide and to analyse the kinematics of the movement, the response to the triggering conditions (i.e. rainfall) or the efficiency of corrective measures. In all cases, measurements have to be made efficiently in terms of time and budget.

In the past, a variety of surveying techniques have been used to track the superficial movements of unstable areas (Wilson and Mikkelsen, 1978; Mikkelsen, 1996). Tapes and wire devices have been used to measure changes in distance between points or crack walls (ISRM, 1984; Gulla et al., 1988). Levels, theodolites, electronic distance

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metres (EDM), and *total station* measurements provide both the coordinates and changes of target, control points and landslide features (Ashkenazi et al., 1980). Aerial or terrestrial photogrammetry provide point coordinates, contour maps and cross-sections of the landslides. Photogrammetric compilation enables a quantitative analysis of the change in slope morphology and also the determination of the movement vectors (Ballantyne et al., 1988; Gili and Sendra, 1988; Chandler and Moore, 1989; Oka, 1998).

A general overview of the accuracy of the methods for measuring landslide displacements can be found in Pincent and Blondeau (1978), Krauter (1988), Gili and Corominas (1992) and Mikkelsen (1996). A comprehensive summary of the main methods and their precision is shown in Table 1. These methods should not be considered as excluding alternatives. In practice, two or more among them should be used in a complementary approach.

In the last few years, the Global Positioning System (GPS) has become fully operational. At least four satellites are now available worldwide, day and night. The equipment is more reliable, lighter, cheaper and easier to use. New procedures, methods and software have been developed to assist in the field data capture and in the post-processing. As a consequence, the GPS equipment is progressively more and more used for a wide range of engineering applications, such as the establishment of control points for photogrammetry and remote sensing images, positioning and levelling of boreholes and wells, off-shore surveys, environmental studies and natural resources management. An important development of the GPS techniques in monitoring tasks and geological mapping is expected for the near future (Cooper, 1987; Mikkelsen, 1996; Keaton and de Graff, 1996).

The accuracy required for the measurement of landslide displacements should be, in many cases, at least the order of centimetres. Therefore, the basic question that arises is whether the satellites orbiting 20 200 km above the earth can be used to measure coordinates or displacements of landmark points located at the ground surface with an accuracy of less than a centimetre.

In the early 1980s, GPS precise applications (*static* methods) were progressively conceived and developed. Gervaise et al. (1985) described the use of GPS in a control network for a scientific facility, the CERN Large Electron Positron (LEP) ring near Geneva. After a 3 days campaign, an overall RMS error of about 4 mm was obtained for a set of six baselines ranging from 3 to 13 km (i.e. about 0.5 ppm).

Bock et al. (1986) applied the GPS to a set of baselines from 71 to 313 km, in California. There, a repeatability of 3 to 10 parts in 10^8, or 0.03 to

Table 1
Overview of methods used in measuring surface displacements and their precision[a]

Method	Results	Typical range	Typical precision
Precision tape	Δdistance	<30 m	0.5 mm/30 m
Fixed wire extensometer	Δdistance	<10–80 m	0.3 mm/30 m
Rod for crack opening	Δdistance	<5 m	0.5 mm
Offsets from baseline	ΔH, ΔV	<100 m	0.5–3 mm
Surveying triangulation	ΔX, ΔY, ΔZ	<300–1000 m	5–10 mm
Surveying traverses	ΔX, ΔY, ΔZ	Variable	5–10 mm
Geometrical levelling	ΔZ	Variable	2–5 mm/km
Precise geometrical levelling	ΔZ	Variable	0.2–1 mm/km
Electronic distance measurement (EDM)	Δdistance	Variable (usual 1–14 km)	1–5 mm + 1–5 ppm
Terrestrial photogrammetry	ΔX, ΔY, ΔZ	Ideally <100 m	20 mm from 100 m
Aerial photogrammetry	ΔX, ΔY, ΔZ	$H_{flight}<500$ m	10 cm
Clinometer	$\Delta\alpha$	$\pm 10°$	0.01–0.1°
GPS	ΔX, ΔY, ΔZ	Variable (usual <20 km)	5–10 mm + 1–2 ppm

[a] Note: 1 ppm means one part per million, or one additional millimetre per kilometre of measured line.

0.1 ppm, was achieved. Comparing the results with those of very long baseline interferometry (VLBI), the accuracy was established as 0.1 to 0.2 ppm. Grellet et al. (1993) and Goula et al. (1996) built a network of a similar scale. In this case, a set of 24 fixed points was established to measure natural deformation in the range 1 mm/year/10 km for seismic risk assessment in the Eastern Pyrenees.

Even better precision figures may be achieved for longer baselines, such as those used to measure crustal deformation (Larson and Agnew, 1991). Global baselines crossing the earth (about 12 750 km) can be measured with repeatabilities of 5 cm, i.e. 0.004 ppm (Rius et al., 1995).

In the field of geomechanics, the GPS system has been successfully used to establish control networks around open cuts, mining areas and gasfield exploitations (Joass, 1993).

In instrumented landslides located in the Swiss Alps, Bonnard et al. (1996), using several receivers simultaneously, obtained a precision of about 1 cm. This precision is higher than that achieved using ordinary surveying techniques of triangulation. Vaccaro (1998) additionally reports a six times saving in GPS surveying time in relation to the classical survey when monitoring a mudslide.

Other GPS measurements have been undertaken to assist in the study and prediction of earthquake or volcanic eruption in zones with crustal deformation or special tectonic activity (Moss et al., 1997).

GPS has also been used to measure *vertical* movements, which are considered the less precise component for measuring. In Ashkenazi et al. (1994), a GPS network is used to study the vertical land displacements and mean sea level in Western Europe. Krijnen and de Heus (1995) and Augath and Strerath (1995) report the use of GPS with subcentimetre accuracy for subsidence monitoring and coastal level changes.

Most of the previous applications rely on static, or *conventional*, GPS observations, which require logging times from one to several hours for each baseline to achieve high precision. This expenditure of time is too high, and makes infeasible the applicability for many engineering works and ground reconnaissance.

In addition to static GPS methods, the *kinematic* mode has been used to monitor the deformations of a reservoir embankment (Collier, 1993) or the movement of a cable-stayed bridge (Leach and Hyzak, 1992). Furthermore, designs and tests have been made to obtain *continuous* monitoring of offshore platforms, gas field areas (Flouzat et al., 1995) and dams (Hudnut and Behr, 1998), or to derive the possible movements in *real time* (or *near* real time) in order to predict disasters in mining areas or due to gliding slopes or avalanches (Hein and Riedl, 1995).

In the following sections of this paper we give a general overview of the GPS principles and discuss its applicability to landslide monitoring. Emphasis will be put on the capabilities of higher productivity GPS methods developed in the last few years, namely the fast-static (FS) and real time kinematic (RTK). Special attention will be paid to the theoretical or a priori errors and the actual precision achieved on site. Some practical considerations on installation of station marks, procedures of calculation and productivity will also be given. Finally, we will present the performance of the GPS equipment in the measurement of superficial displacements at the landslide of Vallcebre in the Eastern Pyrenees (Spain).

2. GPS: basic principles

The GPS is a radionavigation, timing and positioning system with a wide set of applications: from air, sea and terrestrial navigation, to environmental studies, natural resources management, geographical information system (GIS) data capture, surveying and geodetic global measurements. By tracking the electromagnetic waves that the GPS satellites are sending continuously to the world, the system can obtain the antenna position (longitude, latitude, and height, or X, Y, Z coordinates). We describe here only the basic ideas relevant for the proper understanding of the GPS monitoring procedures. A general overview of the system, its applicability, along with a complete description of the GPS equipment, measurement procedures and basic equations can be found in Cooper (1987, Chapter 3), Logsdon (1992) and Leick (1995). Herring (1996) is recommended as short and clear introductory reading on GPS.

The GPS was conceived in the early 1970s based on previous navigation procedures such as LORAN and TRANSIT DOPPLER (Navy Navigation Satellite System). A few years later, successive satellites were launched. The testing period extended from 1978 until 1994 and the system is now fully operational. It is based on the NAVSTAR constellation of satellites, the property of the Department of Defence (DoD) of the United States of America, and developed for strategic purposes. There is a parallel development from the former Soviet Union, based on the GLONASS constellation, but it is less used as it is still under deployment. Nevertheless, receivers able to track satellites of both constellations already exist. Nowadays, the European navigation and positioning system, called GALILEO, is under design and discussion. It will be inter-operable and complementary with GPS and GLONASS.

Although the GPS was first conceived for military purposes, civilian use of the system is allowed. From the beginning, many civil applications were developed (sea, air and terrestrial navigation, scientific measurements, etc.). As mentioned in a previous section, in the last decade the GPS has been used extensively for geodesy and surveying, modifying and completing the conventional survey procedures.

2.1. The GPS sectors

Three different levels — called sectors — can be distinguished in the GPS: spatial, control and user sectors. The satellites compose the *spatial sector*. The NAVSTAR constellation has a basic set-up of 24 satellites, deployed in six orbital planes. The orbits have a mean altitude of 20 200 km over the Earth's surface and are approximately circular. Each satellite completes almost two turns around the Earth per day. They send continuously electromagnetic sinusoidal waves to the Earth, in two carrier frequencies within the L band (carrier waves L1 and L2, with wavelengths $\lambda_1 \cong 19$ cm, $\lambda_2 \cong 24.4$ cm). These carrier waves are modulated according to two codes that have an overall appearance of randomly distributed zeroes and ones. For this reason, they are called *pseudo-random codes*. Specifically, the GPS satellites send two codes: the C/A or *coarse acquisition* code, only onto the L1 carrier, and the P or *precise* code, onto L1 and L2. The last one is usually encrypted. The *control sector* is composed of five main control stations, installed more or less along the equator line. They control the orbits, the satellite function, the synchronisation of time — a capital item in GPS calculations — and, eventually, the clock and orbit corrections. The *user sector* is the only part of the equipment owned by an ordinary user. It consists of one or several antennas and receivers. A large variety of GPS devices are available on the market, with prices ranging from 100 US$ — a single OEM board — to 15 000–60 000 US$ — a complete precision set (Oliver, 1996) — obviously with different capabilities and features.

2.2. The GPS observables

The GPS system calculates the user position using a set of values called the *observables*. They consist of logged data derived from the electromagnetic waves received from each satellite. The computations can be made using either the *code* or the *phase* of the carrier, or a combination of both.

- *Code* measurements: the receiver generates a replica of the code that is delayed until it matches the coding on the incoming signal with maximum correlation. The shift that must be introduced is the travel time of the signal between the satellite and the antenna (Δt_i), plus a small delay of the receiver clock (clock bias, δt_i). This time multiplied by the speed of light (C_0) gives the *pseudo-range*, that is, the distance between a particular satellite (in a known orbital position) and the receiver. Repeating the same procedure for each satellite over the user horizon, a set of equations (3D spatial multilateration) may be built and solved to obtain the antenna position (see Fig. 1). At least four satellites are necessary to determine the four unknowns: X, Y, Z and the receiver clock delay. This procedure is very fast, actually it could be considered as 'real time' after a short start-up time. However, the precision is low due to the large distance between the satellites and the antenna, the small magnitude of time increments, and other systematic errors. The precision with this procedure usually ranges

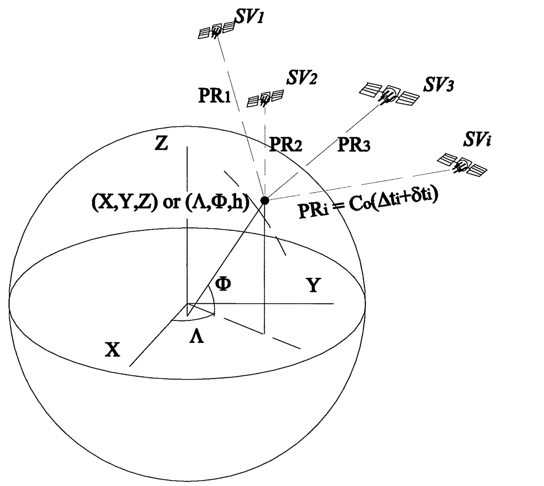


Fig. 1. GPS 3D spatial multilateration of a point based on pseudo-ranges (PR_i) derived from code observation with different satellites (SV_i). (Λ, Φ, h) are respectively longitude, latitude and height of the point above the reference ellipsoid. (X, Y, Z) are the global geocentric cartesian coordinates (see explanation in the text).

between 40 and 100 m for a measurement using a single receiver.

- *Carrier phase* measurements: the receivers read the L1 and L2 carrier, and register the change in phase (phase angle) between the satellite and the receivers with time. The calculations are more involved, with algorithms (single differences, double differences, etc.) extending over observation time, usually from several minutes to several hours. The clue point is the *ambiguity resolution*, that is, the determination of the integer number of complete wavelengths, N, that fits between a given satellite and the antenna (see Fig. 2). This operation can be identified with an initialisation period that has to be spent on some kinematic methods. Often, these calculations are carried out after the fieldwork, during the post-processing, merging the data files from several user receivers. Thus, the results (relative positions, or *baselines*, between antennas) may achieve very high precision, up to a few millimetres.

2.3. GPS field procedures

There are several procedures or methods for data capture and treatment depending on the purpose and precision required for the GPS survey.

- Considering the *number* of receivers involved, the positioning may be absolute (*autonomous*), when using one single unit, and relative (*differential*) when two or more units are used. In all the applications requiring high precision, two or more units (receiver plus antenna) are needed. Relative positions of points in the space are actually obtained with differential GPS. The vector that links two station points is called *baseline* (Fig. 2). A new point is positioned by adding the *baseline* to the coordinates of a previously known one, called *base* point.
- According to the receiver *movement*, the surveys may be *static*, *kinematic*, or a combination of both. Several procedures are available, FS, pseudo-kinematic, stop-and-go, plus others.
- Depending on *when* the solution is obtained, the procedure may be in *real time* (when the position is obtained in the field), or with *post-process*. The latter is used only in certain precision procedures. A special software (with appropriate algorithms, time averaging, filters and network adjustment) is run in order to obtain the final position.

Details about static, FS and RTK methods are included in Section 3.

In order to achieve an optimal productivity and precision, it is very convenient to design carefully the network of control points. For each working session, it is necessary to consider the number and geometry of available satellites, in order to select the best hours and methods to be used. To this purpose, the so-called *dilution of precision* factors (*P*DOP, *G*DOP, etc.) are extremely helpful. These factors are computed from the expected relative geometrical positions between the satellites and the receiver. Their range increases, starting from a value of 1.0 (see Fig. 3). The DOP factor multiplied by the basic precision of the GPS system in use gives the actual precision. Therefore, the lower the DOP factor is, the more precise the GPS observation will be, a priori.

2.4. Errors in the GPS results

The precision of the results is highly dependent on both the equipment and the method used. First

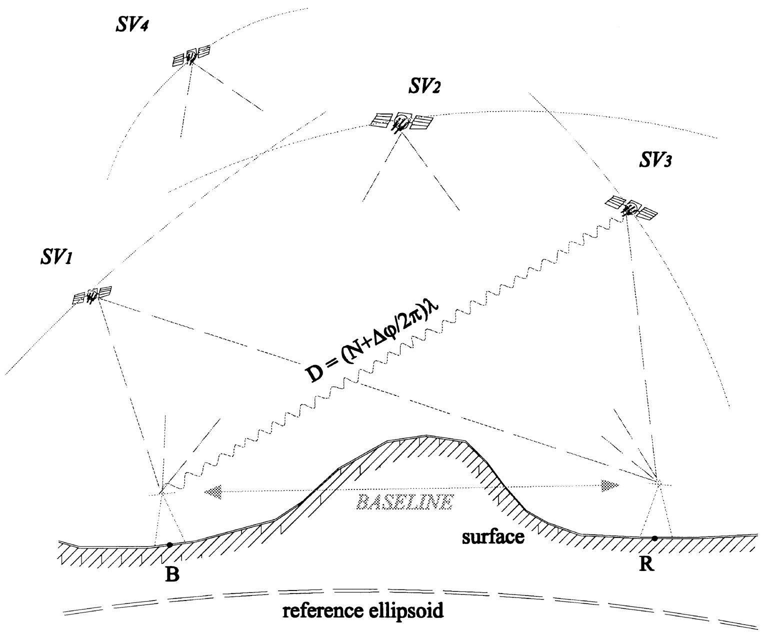


Fig. 2. GPS carrier phase measurements. D is the distance between the antenna and a given satellite, λ the wavelength, φ the phase angle and N an integer number.

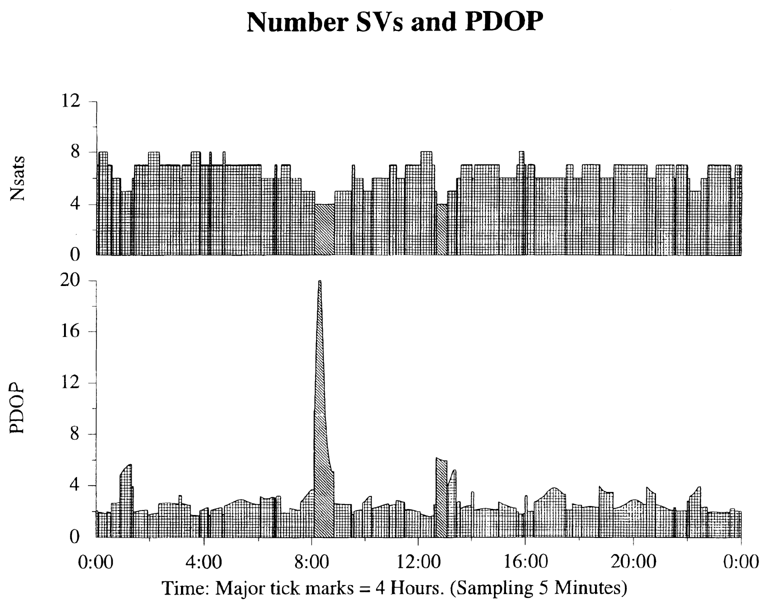


Fig. 3. Example of planning the number and geometry of available satellites in Vallcebre (Eastern Pyrenees, Spain) on February 13, 1997. Top: Total number of satellites above a threshold elevation of 15° plotted against time. Bottom: *P*DOP factor. The values are quite convenient except for 8 to 9 a.m.

of all, we have to avoid *gross* errors such as human mistakes. These errors may be produced, for example, in the process of identification of the station point, setting up the antenna, measuring the antenna height, or by introducing wrong reference point coordinates. Using unsuitable or insufficient data for the precise calculations of the positioning may produce additional errors. This occurs when the signal is lost (satellite occultation, presence of obstacles, fences, forest, buildings and mountains), or unwanted signal reflection occurs with water or walls (*multipath* error). The latter can be minimised with improved antennas and with an appropriate logging and post-processing software.

Other *normal* factors affecting precision or accuracy are: the number and geometry of available satellites (DOP factors); the receiver quality and antenna set-up system; the observation procedure and its duration; the ionosferic and troposferic wave delays; and the precision of orbital parameters used to calculate satellite positions (usually, the broadcast ephemerides). On the other hand, the DoD used to introduce a deliberate perturbation within the system information (satellite clocks and broadcast orbital ephemerides) in order to degrade the accuracy of real time one-receiver positioning. This perturbation is called selective availability (SA).

Finally, post-processing calculations could also affect the accuracy. Ashkenazi and Yau (1986) showed that some discrepancies in the results may appear when processing the same GPS data with different algorithms or software packages. This includes the computation options and parameters, and the coordinate *transformations* (if used): (a) from WGS84 coordinates (the ellipsoid or *datum* upon which the GPS system relies) to local cartographic cartesian coordinates (i.e. UTM); (b) height *transformation* from ellipsoidal altitude to orthometric height (from local sea level *datum*).

Usually, atmospheric delays, orbital parameters, SA and coordinate transformation are the most important factors in terms of positioning error. Fortunately, the effect of these factors can be avoided or reduced when measuring *displacements* with GPS. Using two or more receivers — relative positioning — at points located, for instance, up to 20 km, these errors have a similar influence on both baseline ends, and cancel.

As a simplified approach, in Table 2 three levels of precision and uses are shown.

3. Monitoring of displacements with the GPS: general considerations

The GPS system has become a valuable complement or extension to the conventional surveying methods (theodolite, tapes, bars and EDM, *total stations*, etc.). The procedure is based on the measurement of either the coordinates, distances, or angles of a set of targets from fixed points taken as reference or base points. The difference between the present values and the initial coordinates gives the movement of the target.

Field surveys are usually carried out with a given frequency (i.e. monthly, weekly). The results are discontinuous over time, and related to the cumulative movements of surface points. It is also possible to automate the procedure for the continuous monitoring of the displacements (computer operated servo-total station, permanent GPS stations).

3.1. Advantages and drawbacks

Some considerations should be taken into account before using the GPS. In spite of the improvements made in recent years, the equipment is still somewhat complex. Conceptually, it is convenient to know how the system operates. From an operational point of view, the devices have many options and several field procedures, and a specific software is needed for the pre- and post-processing of field sessions.

Nowadays, the cost of precision GPS equipment is in the same range as that of a good surveying total station. If we also take into account the productivity and accuracy, GPS techniques are similar and often better than geodetic surveying, precise levelling, satellite laser ranging (SLR) or VLBI (Augath and Strerath, 1995). The advantage over conventional surveying increases as the working area enlarges.

A possible constraint, common to classical sur-

Table 2
Main uses of GPS according to the required precision

Use	Measurement basis and no. of units	Typical precision
Geodesy and surveying	Phase (two or more units)	5–10 mm + 1–2 ppm
Low detail surveys, precise navigation, GIS	Pseudo-range (two or more units)	0.8–5 m
Navigation	Pseudo-range (one user unit)	C/A code: SA off, 40 m; SA on, 100 m P code (militar): 1–10 m

veying methods, is operator access to the targets. Accessibility varies at each particular site, but is often restricted in mountain areas. On the other hand, forests may make difficult the line of sight. In classical surveying, it is necessary to establish good visual lines from base or reference points to targets. This is especially difficult in landslide areas in which fixed or stable points cannot be placed in the neighbourhood of the landslide site. Thus, it is common to set up the surveying instruments on the opposite valley slopes, often stable, but located far from the landslide site, thus reducing the precision of the measurements. Furthermore, to achieve precise measurements, one must have special atmospheric conditions, such as absence of rain or smog, proper illumination, lack of air vibration that produces fuzzy images in the telescope. These requirements are specifically important for angular measurements. Conversely, GPS does not require direct line of sight between stations. In most of the GPS procedures it is possible to compute precise baselines, their extreme stations being at opposite sides of a hill or building. The antennas, however, must have good sky visibility, to receive the satellite signals without interference. Ideally, obstacles should not appear 15° above the horizon. Moreover, the GPS can work regardless of weather conditions, and may be used with rain, mist or fog, strong sunshine, or at night.

Other advantages of the GPS system are: (i) GPS can easily cover larger areas than surveying methods, with centimetre level precision; (ii) the results between consecutive surveys are coherent — the procedures, data capture and computation are always the same, regardless of the operator and the particular GPS equipment; (iii) after a due training period, mentioned above, the equipment is easy to operate. The operator does not have to be an experienced surveyor. Therefore, he/she may be a scientist devoted to the engineering geology aspects of the landslide.

3.2. Special considerations for landslide monitoring

Some hints for monitoring landslides with GPS over long time periods are as follows.

- The selected targets must reflect the mean behaviour of the surrounding area. It is not desirable to choose points that may exhibit their own movements, such as large boulders or electric and telephone poles. However, sometimes it is unavoidable.
- Tree canopy, buildings and other obstacles that could restrict the reception of the satellite signal should be avoided.
- To mark the station points, it is advisable to use discreet and sturdy signal systems, such as rock engraving or steel pins, rather than big geodetic-type concrete cylinders. The latter are expensive and usually have a lack of strong foundations. Consequently, they are more prone to natural disturbing actions and to vandalism.
- It is convenient to include within the control network several fixed points outside the landslide area. The successive measurements of these points, that should indicate a more or less constant position, will show the actual reliability of the monitoring set-up, including base point stability.
- To strengthen the results, it will also be helpful to use different methods and repeat the observations. For instance, if we use polar *radiation* of baselines to position the targets from the base point A, we could repeat the tour radiating from another base point B (*double-radiation*). Alternatively, we can use a *reoccupation* method, which consists, essentially, of repeating

the observation at a different hour or day, preferably with a different satellite arrangement.

3.3. Field methods to use in landslide monitoring

As will be presented later, a combination of classical methods and GPS may be envisaged for each practical case. This scheme will increase the precision, reliability and productivity requirements. For instance, several stations may be positioned with GPS, fairly distributed in large areas of a landslide. In a second stage, many details can be calculated through polar radiation of coordinates with a total station around these known points. In this sense, GPS measurements may be used to check the stability of theoretically fixed control monuments used in classical surveying. For example, master fixed stations in the Panama Canal were periodically checked using GPS (Reyes and Fernández, 1996).

Measurement of landslide displacements can be undertaken by means of either static or kinematic methods. The choice depends on the same practical considerations quoted before, namely, the accessibility, number of points, precision, obstacles and distance from point to point. The most productive methods available for determining single points with precision of millimetres or centimetres are FS and RTK, see Fig. 4. These methods were developed in the late 1980s and early 1990s, but have been fully available since 1992 and 1994, respectively.

The FS method is a development of the classical static method, with improved algorithms that speed the procedure (Blewitt et al., 1989; Frei and Beutler, 1990). For instance, measurement of one baseline with six or more satellites available requires 8 min of logging. This time increases to 15 and 20 min with five and four satellites, respectively. No positioning is possible with less than four satellites. In this method a post-processing must be carried out. The data files from different receivers are merged in order to obtain the solution, that is, the baselines between station points.

In the RTK method, the information of code and carrier phase observables received at both extremes of the baseline (*base* station and *rover* station) is merged to compute the precise position

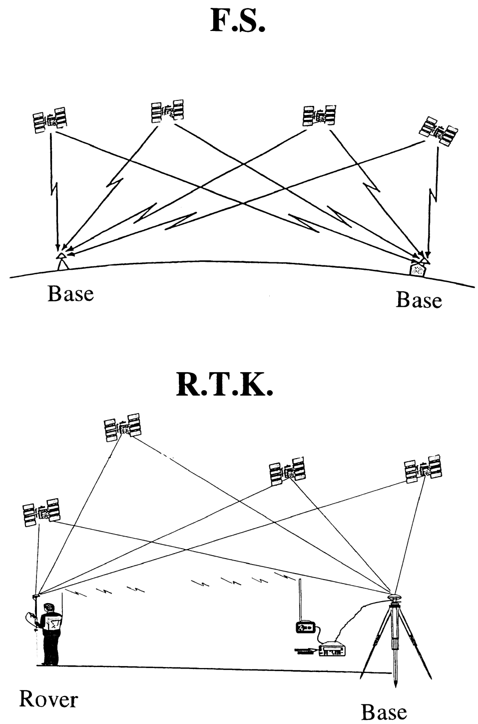


Fig. 4. FS and RTK GPS methods. The station over a known fixed point is referred to as 'base', and the receiver that moves from point to point as 'rover'.

on the spot (Quirion, 1993). The base receiver transmits to the rover a message containing its position, the pseudo-ranges measured through code correlation, and phase measurements of the carrier received from the available satellites. The information recorded directly by the rover is added to calculate its position. Prior to obtaining the first results, it is necessary to spend a few minutes in initialising the system (the so-called *ambiguity resolution*). The RTK calculates new positions from the old ones, through continuous tracking of the satellites in real time. In this procedure, therefore, the post-processing of the field data is not required. Any problem with the equipment can be localised and solved immediately. It works quickly and gives results with precision of one centimetre every second, even in movement.

In RTK, corrections are transmitted to the rover via a local UHF data link; this transmission is quite directional. Therefore, unless repeaters are used, the RTK method needs an almost direct line of sight between base and rover. In general, this method also needs four or more satellites to work. Due to the continuous tracking of satellites, it is very sensitive to the loss of reception and to the quality of the signal. Even short interruptions will produce a *loss of the initialisation*. To recover it, at least five satellites are needed.

The FS is less sensitive to sporadic loss of signal reception, due to the fact that it is static and the observables are usually recorded at intervals of 15 s. Sometimes, FS may be used with the antenna partly covered by a tree canopy, provided that it is not very dense [see Fig. 13(c) later]. This depends on the satellite number and position, but also on the season of the year, due to different foliage density. The presence of trees will increase the logging time, but in some cases it will be impossible to end the session successfully. In these cases, care has to be taken to avoid gross errors in the results due to insufficiency of data.

In Table 3 the main characteristics of the methods are shown, according to Oliver (1996) and the manufacturer's technical notes. The last column includes an estimation of the a priori or theoretical error in the horizontal plane.

3.4. Precision in landslide monitoring with GPS

As can be seen in Figs. 2 and 4, the rover coordinates are calculated from the base point as follows:

$$X_{\text{rover}} = X_{\text{base}} + \Delta X_{\text{baseline}}. \qquad (1)$$

The error associated with the GPS measurement of the baseline between antennas, $e(\Delta X_{\text{baseline}})$, can be evaluated from the technical specifications provided by the receiver's manufacturer. This error can also be estimated from the last column in Table 3. But the antennas are set up over the base and rover point, so we have to take into account the set-up errors at both baseline ends. These set-up errors are often omitted in technical discussions and can be bigger than the first one in certain applications. Therefore, the total error will be the addition of the following terms: base coordinates error, base station set-up error, baseline error, rover station set-up error:

$$e(X_{\text{rover}}) = e(X_{\text{base}}) + e(_{\text{base set-up}}) + e(\Delta X_{\text{baseline}}) + e(_{\text{rover set-up}}). \qquad (2)$$

In case the objective is to calculate movements at a given time, M^t, by differences between successive measurements, then

$$M^t_{\text{rover}} = (X_{\text{rover}})^t - (X_{\text{rover}})^\circ = (X_{\text{base}} + \Delta X_{\text{baseline}})^t - (X_{\text{base}} + \Delta X_{\text{baseline}})^\circ. \qquad (3)$$

If the base coordinates are the same at time 0 and at time t, the former expression is reduced to:

$$M^t_{\text{rover}} = (\Delta X_{\text{baseline}})^t - (\Delta X_{\text{baseline}})^\circ. \qquad (4)$$

Thus, the first term of Eq. (2), $e(X_{\text{base}})$, will have no influence on the computation of the movement between surveys, Eq. (4). In monitoring works, if the coordinates of the reference point, X_{base}, are not exact, this fact has no influence on the computation of the displacement, provided that the same values of X_{base} are always used and that the base points are stable.

Working by differences, several errors that behave as systematic are filtered. Then, as systematic errors are cancelled, instead of *precision*, we can speak properly about *accuracy in the determination of displacements*.

Table 3
Main characteristics of RTK and FS GPS methods compared with static method

Method	Observed time, per point, after initialisation	Post-process	Strength against loss of signal	Typical max. baselines (km)	Typical baseline planimetric error
General static	One to several hours	Yes	Robust	50–100[a]	5–1 mm + 1–0.1 ppm
FS	8–20 min	Yes	Robust	15–20	5 mm + 1 ppm
RTK	1–10 s	No	Sensitive	10	10 mm + 2 ppm

[a] Baselines can easily reach about 50 km. Depending on the observation time, type of post-processing and required precision, it may be extended up to thousands of kilometres.

The remaining error terms, that will be considered random and independent, are:

$$e(M_{\text{rover}})=e(_{\text{base set-up}})^t+e(\Delta X_{\text{baseline}})^t +e(_{\text{rover set-up}})^t-[e(_{\text{base set-up}})^\circ +e(\Delta X_{\text{baseline}})^\circ+e(_{\text{rover set-up}})^\circ]. \qquad (5)$$

The set-up terms depend on the set-up method used. The most accurate is to put the antenna directly over an outcrop of rock or over a concrete cylinder [about 0.1 mm of error; suitable for the base; Fig. 5(a) and (b)]. The simplest set-up method is to use a tripod with optical plummet [1 to 3 mm of error; suitable for base and rover; Fig. 5(c)]. Finally, the telescopic pole or rod with bubble level is suitable for rover points [see Fig. 13(b) later]. The error may be as high as 15 mm. As a general rule, the correct verticalization of the optical plummets and measuring poles have to be checked to discard systematic errors.

To avoid the construction of concrete cylinders, a common set-up may be to locate a tripod on the base point and to use a telescopic pole with bubble level on the rover. The set-up errors may be, for instance, 2 and 10 mm, respectively.

The baseline error depends slightly on the GPS method as shown in Table 3. Therefore, combining the terms in Eqs. (2) and (5) properly (quadratic composition), we will obtain the estimated total error (in terms of RMS) for RTK and FS GPS positioning (Table 4). As can be seen, the error in the Z-axis is slightly greater than the planimetric error.

The values in Table 4 correspond to a typical GPS landslide-monitoring scheme, as the cases presented in the next section. In case more precision is needed, forced setting up in the base and tripod setting up in the rover may be used.

As a real error example, Fig. 6 summarises a set of repeatability tests carried out with both methods in a short baseline. The continuous vertical line is the mean value of 18 measurements with tape [Fig. 6(a)]. 27 baseline lengths obtained with the FS method are shown in Fig. 6(b). The duration of FS sessions was 8, 15 or 20 min, according to the number of satellites. The results are in a ± 10 mm interval, with a mean value only 1 mm apart from the true value. In Fig. 6(c), 24 measure-

Fig. 5. Different antenna set-up in Vallcebre and Pont de Bar (Spain) base points: (a) antenna directly over a rock outcrop; (b) station over a concrete cylinder; (c) set-up with tripod having optical plummet. The GPS receiver, in the yellow case on the ground at the left in (b) and (c), is connected to the antenna through a signal cable. The plane dish around the antenna that can be seen in (a) and (c) is the *ground plane* that reduces the multipath error.

Table 4
Example of estimated planimetric and elevation errors for typical GPS landslide monitoring[a]

Method	Typical $e(X_{rover})$ (mm)	Typical $e(M_{rover})$ (mm)	Typical $e(\Delta Z_{rover})$ (mm)
FS	12	17	26
RTK	16	22	35

[a] $e(X_{rover})$ is the single positioning planimetric expected error [Eq. (2) with $e(X_{base}) \cong 0$]; $e(M_{rover})$ is the expected error in the planimetric (X–Y plane) displacement [Eq. (5)]; $e(\Delta Z_{rover})$ is the expected error in the displacement along the Z-axis. Errors are in terms of RMS. A baseline of 1 km has been used for computations.

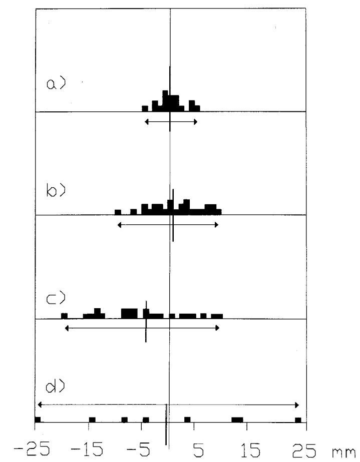


Fig. 6. GPS repeatability test over a baseline of 8 m. The measurements have been done with the same equipment, but by several users and on different days. (a) 18 measurements carried out with tape. (b) 27 baseline lengths obtained with FS method, setting up with tripod and optical plummet. (c) 24 baseline lengths obtained with RTK method. Same set-up method as (b). (d) Eight GPS measurements done with the antenna over a vertical rod with bubble levelled by hand.

ments with RTK method (5 s each, but on different days) have been drawn. The span of the results is ± 15 mm, and the mean value is 5 mm left from the correct one. As can be seen, the FS and RTK measurements are in good accordance. Eight measurements are shown in the plot of Fig. 6(d), and emphasise the importance of good setting up: with a vertical pole and bubble levelled by hand, the error increases to ± 25 mm.

3.5. Productivity

High productivity ratios can be achieved with GPS in general surveying, especially with kinematic methods such as RTK. These ratios may be reduced in landslide monitoring due to the following factors.

- *Difficult access and connectivity between control points.* Sometimes the operator has to walk up and down the hill to reach the points. The size of the landslide site it is also an important factor to be considered.
- *The GPS method used, in relation to the obstacles around the target and precision level to achieve.* The time spent per point ranges between about 2 s (after initialisation) with RTK, to several minutes per point with FS.
- *Number of available receivers (minimum two).* For instance, with three receivers two dispositions may be used: one base and two rovers (double productivity), or two bases and one rover (double radiation in one lap), see Fig. 8.

As an example, in *Vallcebre* landslide, presented in the next section, using both FS and RTK methods with two receivers, in a wide range of situations, travelling across the slope by car and on foot, about 25 points are measured in a single day. The disposition used is a radiation from known base points. The network map of Vallcebre can be seen in Fig. 7. In this case, the productivity of the GPS is very similar to that of classical surveying. The main advantage is an increase in precision and reliability of results under any weather conditions.

In Fig. 8 two other disposition examples are shown. In *Pont de Bar* landslide the disposition used is a double radiation with two bases and one rover. This can be achieved in one lap (almost 1 day for 15 points, using FS method) if three receivers are available, or in two laps (about 2 days) if we have only two.

In a regional program of measurement of subsidence [Fig. 8(b)] four receivers have been used. An outer ring linking four geodetic pillars (B1, B2, B3, B4) around the area has been observed. To gain confidence in this basic ring, it has been connected with other geodetic vertexes quite far away (20–25 km, A2, A3, A4). After that, for

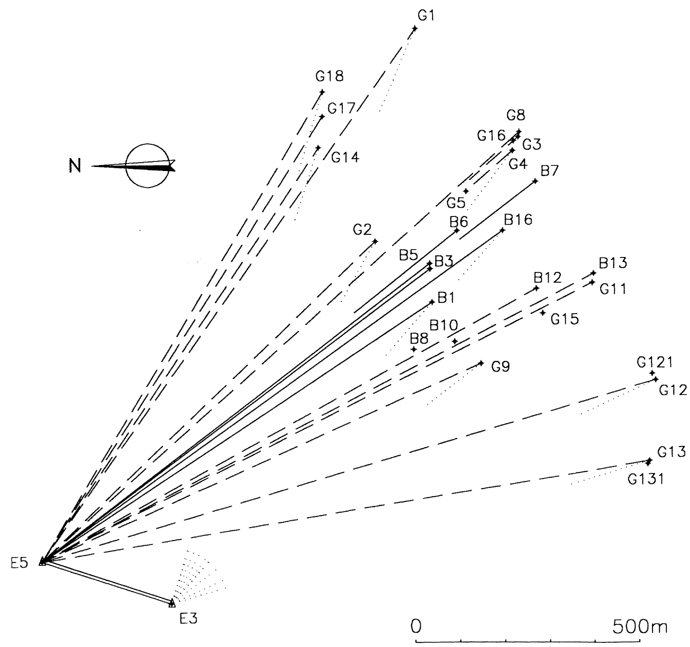


Fig. 7. Vallcebre landslide monitoring network: solid and dashed lines are, respectively, FS and RTK baselines. The dotted lines correspond to an optional double radiation that should be done once a year. For clarity, not all the lines are shown.

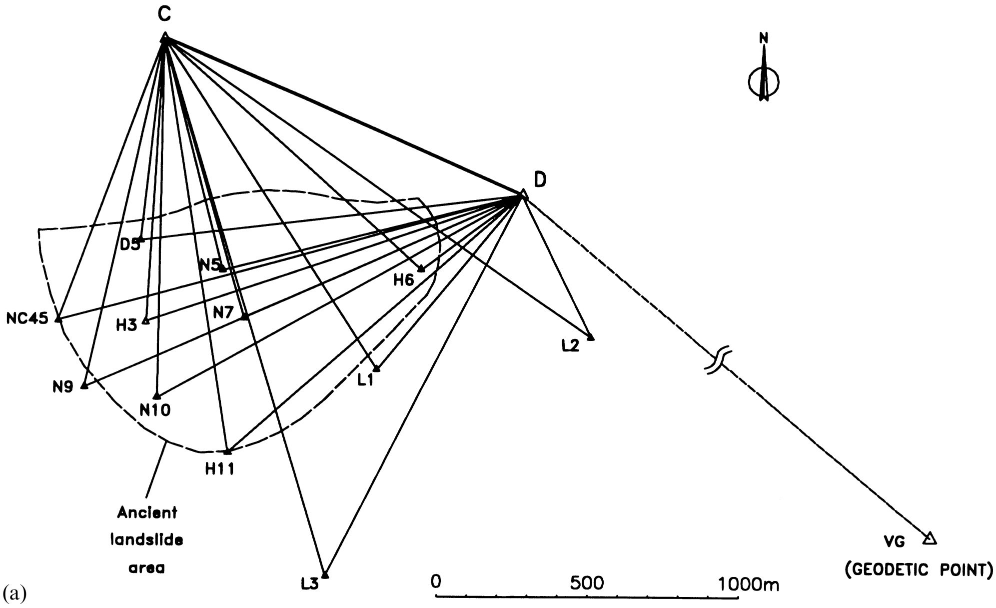
each pair of station points in the area of interest, a logging session has been done. In the figure, as an example, the polygon linking base points B2 and B4 with C11 and C17 is highlighted. Up to six baselines are observed simultaneously each logging session. To achieve better results and redundancy, reoccupation of points (in different order and on another day) has been used. All the baselines have been observed with the FS method: 1 h for the external ring, 15 to 30 min for the logging sessions (dashed line). Due to the distances and difficult access to geodetic points, between 1 and 2 days are necessary to connect all the geodetic vertexes (A and B points). Around 1.5 days are necessary to observe (twice) all quadrilaterals such as the dashed one. In this example, the amount of work, productivity, precision and cost of the GPS approach are clearly better in relation to classical surveying techniques.

4. Monitoring of the Vallcebre landslide using GPS

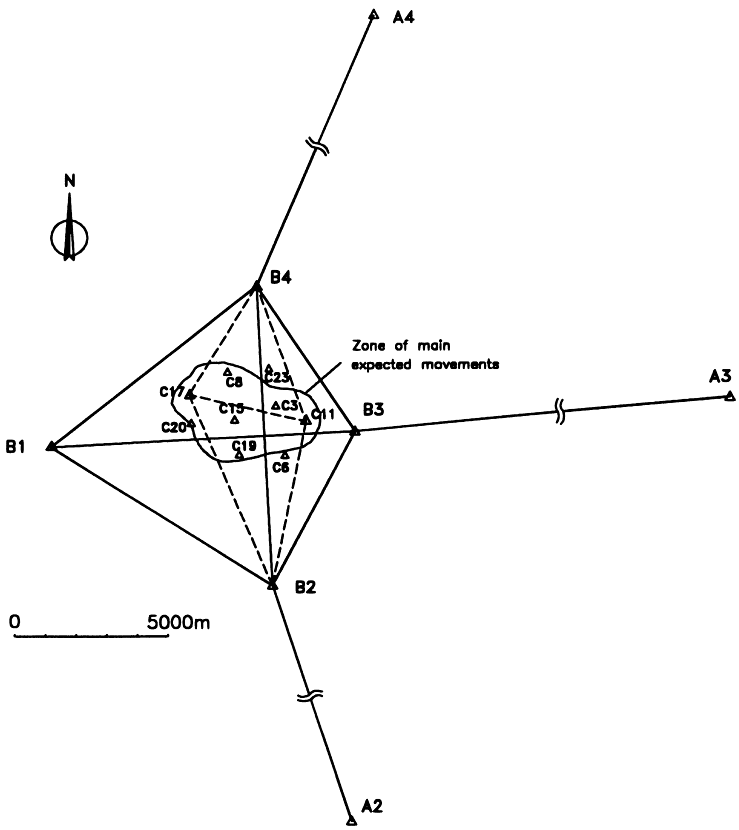
The performance of the GPS in measuring ground displacements has been tested in the active old landslide of Vallcebre. This landslide is located in the Eastern Pyrenees, 140 km north of Barcelona, Spain. The landslide extends on the eastern slope of the torrents of Vallcebre and Llarg (Fig. 9). The mobilised material consists of a set of shale, gypsum and claystone layers gliding over a thick limestone bed. The average slope of the whole landslide is about 10°. The landslide affects an area of 0.8 km^2, which shows superficial cracking and distinct ground displacements.

The landslide of Vallcebre is a translational slide with a stair-shape profile. It is composed of four main morphological units of decreasing thickness towards the landslide toe. Secondary scarps of a few tens of metres high bound the units. At the toe of each scarp, there exists an extension area that originates a graben. Fig. 10 is a geomorphologic sketch of the landslide with the location of the monitored points and boreholes.

Vallcebre landslide has been monitored since 1987 using conventional surveying and photogrammetry (Gili and Corominas, 1992). In 1996, this site was included in the frame of the NEWTECH Project funded by the European Commission. The



(a)



(b)

Fig. 8. (a) Pont de Bar landslide (Eastern Pyrenees, Spain). Network map with base (C and D) and monitoring points. Conceptually, the disposition used is a double radiation, with each point positioned from two base points. (b) Conceptual arrangement of baselines in a regional program of measurement of subsidence (Central Catalonia, Spain). Different points and baselines may be distinguished: far geodetic points (A2, A3, A4); outer ring of geodetic pillars (B1, B2, B3, B4); and 14 monitoring points (C3, C8, etc.). Using four receivers, the basic disposition is similar to the polygon in dashed lines, linking two base points and two subsidence points. The sessions have been extended to other quadrilaterals with different C points. Reoccupation has been used as well.

Fig. 9. General view of Vallcebre landslide from the west.

purpose of the project is to apply and improve new techniques and equipment for landslide monitoring and modelling. Between July 1996 and March 1997, 14 boreholes were drilled in the landslide and equipped with inclinometers, wire extensometers and piezometers. Borehole logs have shown that the thickness of the lowest unit ranges between 14 and 20 m, while the intermediate unit reaches up to 50 m. The failure surface of the landslide was deduced from the results of inclinometer readings. Its average inclination is 10° towards the Vallcebre torrent and it runs roughly parallel to the ground surface. Further details about the site, the fieldwork carried out and the monitoring scheme can be found in Corominas et al. (1999).

The rate of displacement is variable although the most active part is the lower unit, close to the torrent. Data of wire extensometer displacements and of the fluctuation of the groundwater level were systematically recorded and stored in a datalogger. In addition to these observations, we carried out periodic monitoring of the Vallcebre landslide. The measurements were made with a probe inclinometer every two or three weeks until they went out of order, and with GPS. The accuracy of the displacements obtained by GPS at the upper end of nine boreholes has been checked with the results provided by EDM, inclinometers and wire extensometers, and is shown hereafter.

4.1. Previous slope surface measurements

Widespread evidence of activity makes it feasible to start the monitoring of surface displacements as the most convenient approach in terms of investment and disturbance. Most of the slopes around the Vallcebre landslide are unstable as well. Therefore, setting up of the control stations was restricted. The only assumed stable ground is the limestone layer cropping out at the northern edge of the movement.

The first monitoring network was based on

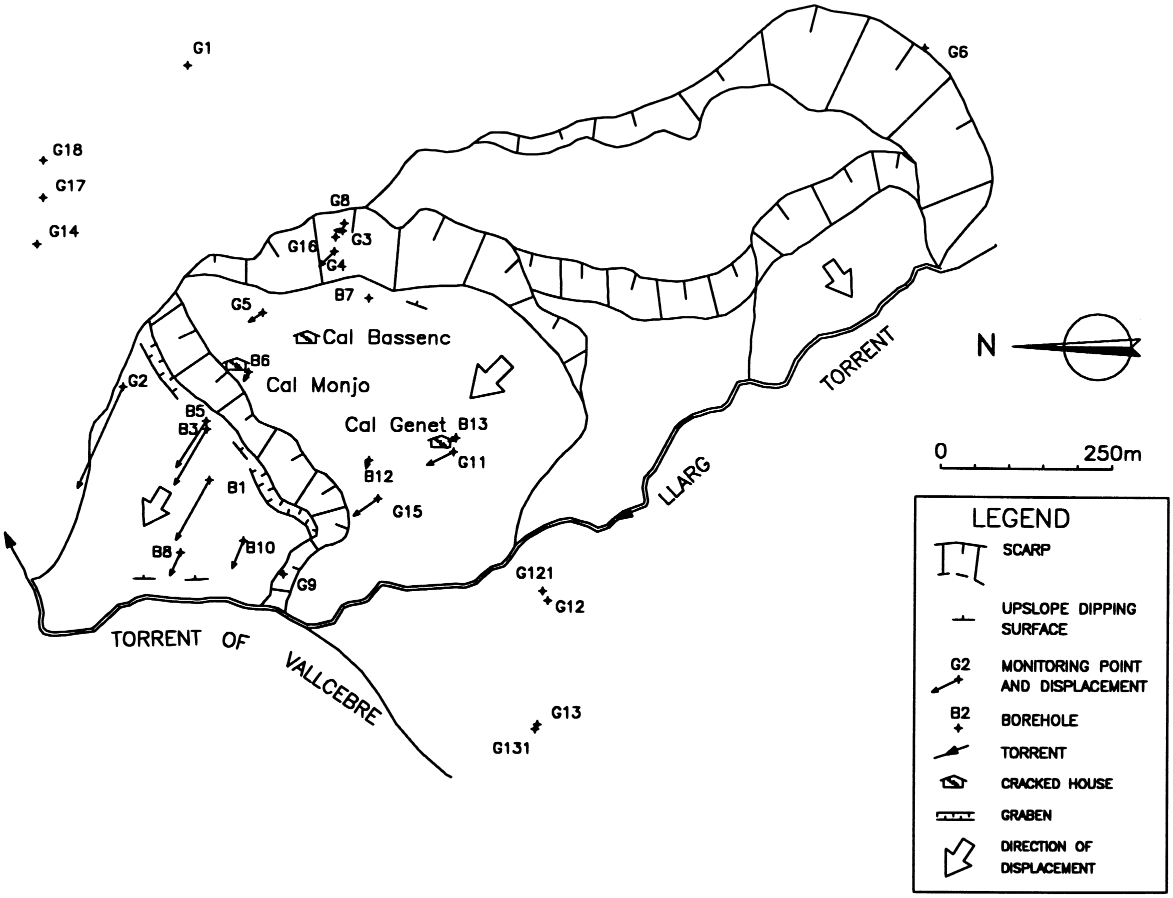


Fig. 10. Geomorphological scheme of Vallcebre landslide. The thickness of the landslide units decreases from the upper scarp to the Torrent of Vallcebre. Base points E3, E4 and E5 are outside this map, about 1 km NW from B1. Displacement vectors between December 1995 and February 1998 are enlarged by a factor of 100.

terrestrial photogrammetry (seven campaigns at the landslide foot between 1987 and 1992) and, since 1988, on geodetic measurements with theodolite and EDM. This stage allowed the identification of the most active sectors within the landslide and a first estimation of the relation between rainfall and surface displacements was envisaged. Details of these previous works can be found in Gili and Sendra (1988), Gili and Sanjuán (1995) and Rius (1996). The results showed that between 1987 and 1994 displacements as large as 8 m were observed at the landslide foot. The rate of movement was strongly dependent on the rainfall. Rates of about 4 m per year were observed at certain points in the rainy years, while almost no displacement occurred during periods of drought. In the other landslide units, the rate of displacement was significantly smaller, in the range 10–15 cm/year. The evolution of a set of targets spread over the landslide, established in January 1994, is shown in Fig. 11.

In terms of the precision of the observations, the EDM measurements have proved to be more reliable than angle measurements with theodolites. This is due to the fact that the precise determination of angles needs greater experience and better environmental conditions (line of sight, vibration, etc.), as quoted above. This fact has been outlined in Fig. 12(a).

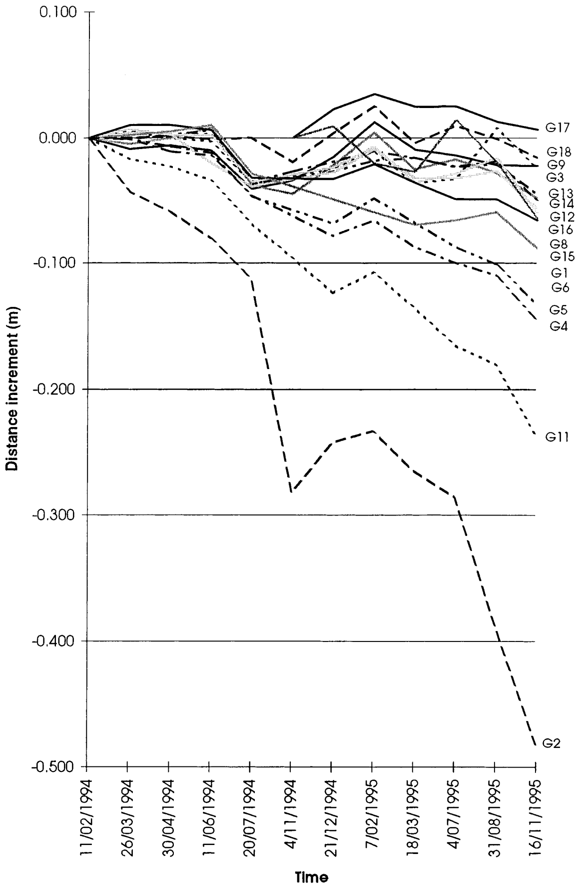


Fig. 11. Changes in the electronic distance measurements of 16 targets spread over the landslide, along 12 campaigns during 1994 and 1995 (Rius, 1996). The highest displacement was measured at point G2, with 48 cm. Average rates of about 0.5 m/year were reached during the autumn season of 1994 and 1995. Some points exhibit apparent uphill movement due to measurement errors.

4.2. The GPS surveys

Theoretically, GPS a priori errors have to be more balanced in the three axes [Fig. 12(b)] compared with theodolite and EDM ellipses. This point was confirmed by the results of several monitoring campaigns, and led us to implement a GPS scheme in Vallcebre.

GPS preliminary tests were made in July 1994. Five baselines, from 800 to 1500 m, were observed with Topcon Geodetic Receivers GP-R1, following the static method (logging sessions of 45 min, recording each 20 s). The results were promising due to easy-to-use, precision and matching with surveying measurements. The comparison of baseline lengths gave typical differences of 15 mm (maximum 26 mm). Then, another test was carried out. A displacement of 10 mm north was imposed on the antenna, and the static observation repeated. The new computations gave a vector movement of $\{\Delta E=0\ \mathrm{mm},\ \Delta N=16\ \mathrm{mm}\}$. All previous results are within the expected errors, taking into account that the EDM measurements were made with a Wild DIOR 3002S, with standard error of $\pm(4\ \mathrm{mm}+3\ \mathrm{ppm})$.

In December 1995, a complete EDM and GPS survey was carried out, to link the measurements taken using classical methods with the first campaign using GPS. The equipment used was a Trimble *GPS Total Station 4000 SSi* model with two dual frequency receivers, antennas and accessories such as internal memory, data link and software (Figs. 5 and 13).

New base points were established (E3, E4 and E5, engraved directly on the stable surface of the limestone layer) for stability and visibility requirements, and most of the old targets were recovered with minor modifications (Gili et al., 1997). As new points have been added since 1996, the monitoring network now has 30 points (Figs. 7 and 10 and Table 5). The first type of point includes three base points and four additional points on the limestone around the sliding zone. These points were the fixed points used to check the GPS accuracy. This network allowed both the measurement of displacements and the comparison with movements obtained with the borehole equipment (inclinometers and wire extensometers).

The set-up in 16 of these points was made with a telescopic pole (up to 2.5 m), while at three points the antenna was put directly over the mark (the corner of the roof of a farm and two iron pins on the poles). A telescopic pole does not give the highest precision, but it is easy to transport and allows quick setting up. In Vallcebre the pole could be maintained vertical with an auxiliary tripod, providing enough precision for the range of expected movements. At important points such as base points and boreholes, we have set up an over tripod with optical plummet (11 points). In

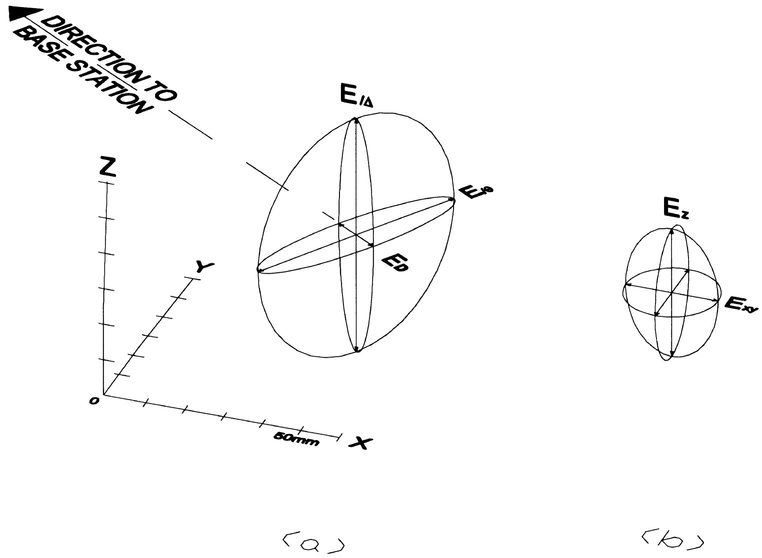


Fig. 12. Comparison of typical positioning uncertainty in Vallcebre: (a) due to distance (E_D) and angle errors ($E_{/\Delta}$ and $E_{/\theta}$); and (b) due to GPS errors (E_{xy}, planimetric, and E_z, in elevation).

Fig. 13, different types of point and set-up may be appreciated.

The GPS methods used were RTK and FS. We try first the RTK method for productivity reasons. However, at certain points, RTK positioning became infeasible due to the satellite position or to difficulties in reception of the signal because of the presence of tree leaves. Then, the FS method was used. This method is also recommended for important determinations such as the first positioning of a new target, or for baselines linking base points. In a normal campaign, about 25 points are measured: four to six with FS, the rest with RTK.

14 campaigns were carried out from December 1995 to February 1998, one survey every 2 months approximately. For a regular campaign the following tasks were accomplished.

- *Preparation tasks*. Prior to survey sessions, we planned the satellite constellation during the day. Thus, we knew in advance the number and geometry of available satellites, the best hours, routes linking stations, and GPS methods to use. This planning was prepared with specific commercial software like Quickplan (Fig. 3), and must be done with recent ephemerides, that is, the foreseen satellite orbits contained in the *almanac* files. Anyway, unexpected deviations from the planned satellite behaviour could not be discarded. Finally, the GPS equipment was checked and the battery units charged.
- *Running the GPS survey*. The first in situ task was setting up the base station. Once the equipment was checked, the rover crew (one or two operators) obtained successively the survey stations, following the planned schedule. After reaching a given point, the GPS antenna was set up (with tripod or telescopic pole), the antenna height was measured, and then the observation procedure started. Usually, the route was arranged in such a way that the RTK measurements were done first, and afterwards the FS sessions. As real initialisation or logging times are hard to anticipate, and also equipment difficulties may arise, the session's schedule was adjusted using two-way radio communication between base and rover crew. Parallel with GPS tasks, general geomorphological changes were recorded and manual readings of automatic logging equipment at boreholes taken. When all the rover points had been visited, all the equipment was torn down. Once a year, double radiation should be done. This implies, essen-

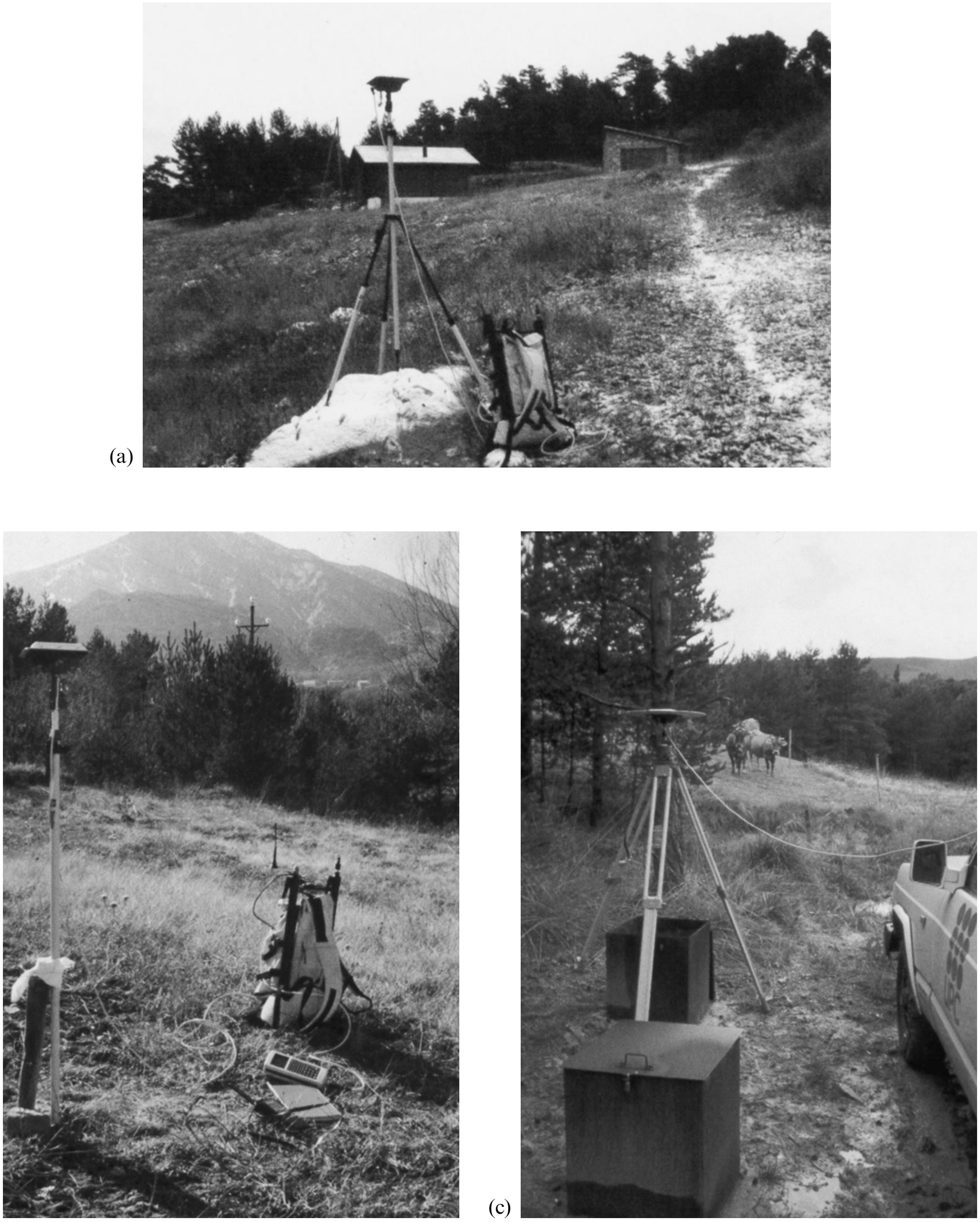


Fig. 13. Different set-up methods in Vallcebre monitoring points. (a) Telescopic pole with auxiliary tripod at point G1, an iron pin in a rock block. (b) At point G15, a wood stake in soil, the telescopic pole could be maintained vertical by hand for RTK positioning. The GPS receiver is in the yellow backpack and is controlled by means of the data collector TDC1 beside the notebook and the walkie-talkie. At the backpack top, the small black antenna of the UHF data link receiver can be identified. (c) Set-up with tripod having optical plummet over an inclinometer borehole, B1.

Table 5
Type of GPS points in Vallcebre network

Type of marking	No. of points
Point engraved in limestone layer	7
Point engraved in rock blocks	9
Steel rods (0.5 m) in soil	1
Wood stake in soil	1
Poles	2
Roof of buildings	1
Top of boreholes casing (usually inclinometer)	9

tially, repeating all the survey using another base point and scheduling, requiring an additional field day.

- *Post-processing*. This consisted of data reduction, checking, adjustment and representation of results. These tasks were computer-aided and thus the logged data had to be downloaded from the internal memory within the GPS receivers (FS) or from a data collector (RTK) into a personal computer. Downloading was accomplished with the help of specific communications software, that in our case is a module of the main processing package, the standard programs supplied by Trimble (GPSurvey and Trimmap). After 10 to 30 min of computation, the baselines and the new points were obtained in the WGS84 system (longitude, latitude, and height). Then, a local transformation and the UTM map projection were applied to convert them into UTM cartesian coordinates. The difference of present values from initial campaign coordinates gives the apparent displacement of rover points. The local transformation is only approximate, but we always use exactly the same values. As has been mentioned in Section 2, the possible error has a similar influence on every campaign, and the computed displacements will not be affected. Other process control parameters were: 15° elevation mask, Hopfield simple model for tropospheric correction, ionosphere correction based on double frequency observations, and broadcast ephemerides. Finally, the displacements were plotted on adequate graphs.

4.3. Results and analysis

The results obtained with GPS are drawn on ΔX–ΔY axes (Fig. 14). Plots of both ΔZ (settlement) and ΔD (distance variation) versus time were obtained as well (Fig. 15). In Fig. 10, the displacement vectors at each GPS point between December 1995 and February 1998 have been added to the Vallcebre map.

All these graphs are very appropriate to identify active zones and accelerations through time (usually in relation to heavy rainfall). Although a complete Vallcebre landslide interpretation is beyond the scope of this paper, we mention that the GPS results, along with other techniques, confirm that there exist different activity zones within the landslide. The most active part is the

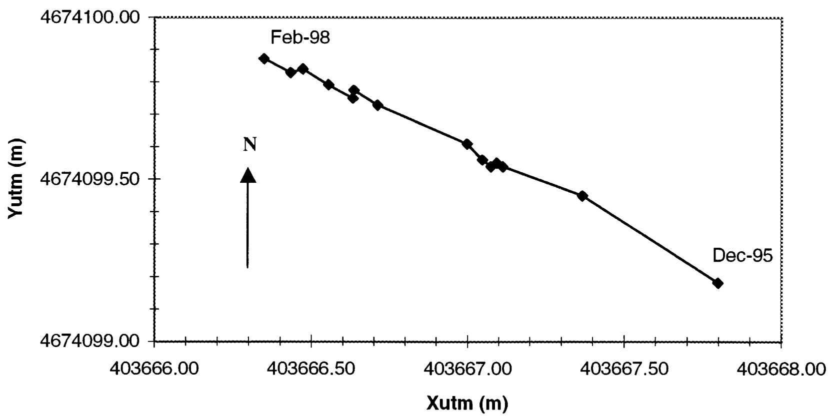


Fig. 14. Horizontal displacement of point G2, between December 1995 and February 1998.

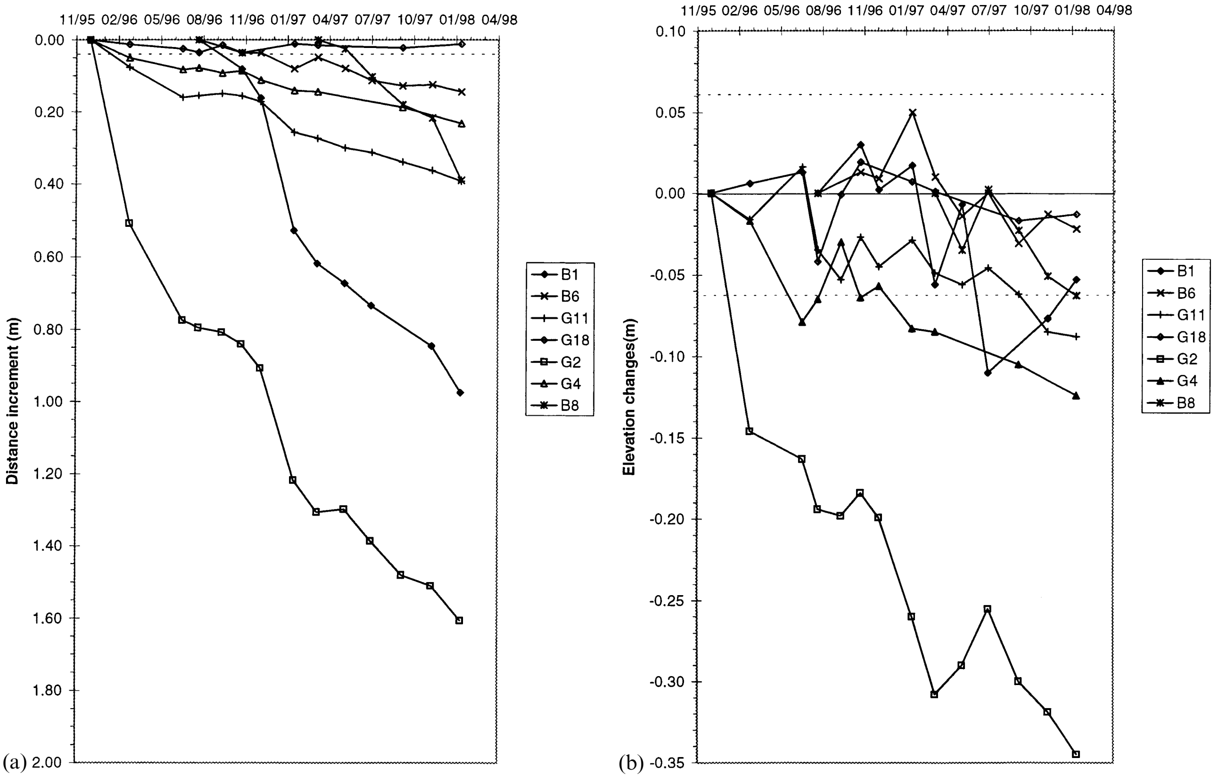


Fig. 15. Vallcebre landslide. (a) Changes in distance from initial position of seven representative points along 14 campaigns between December 1995 and February 1998. (b) Changes in elevation for the same points and period. A maximum error fringe has been added to both plots to highlight the tolerance threshold.

area close to the Vallcebre Torrent (foot of the slope, points G2, B1, B3, B5, B8, B10), where the torrent is continuously eroding the toe of the slope. During the winter of 1996–1997, rates of displacement up to 13 cm/month have been recorded in this zone (Fig. 15), but rainfall can generalize the movement to 3 cm/week (Ledesma et al., 1997). With this lower unit, the average velocity of displacement during 1997 is 3 cm/month. These results have been checked in several ways to analyse their precision. First of all, the plots show a more defined trend compared with those shown in Fig. 11. In Fig. 16 we can detect the different apparent behaviour of point G2, for instance, when monitored with classical surveying and with GPS. At control points located on top of an inclinometric tube, the GPS results were compared with inclinometric readings. Fig. 17 shows both measurements. The precision of inclinometric measurements is higher, but the GPS values fit well within their own error bar.

The GPS measurements have also been used to

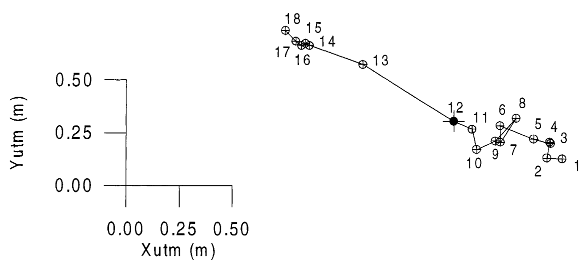


Fig. 16. Different apparent behaviour of Vallcebre point G2, when monitored with classical surveying and with GPS: 1 to 12, classical surveying, between January 1994 and November 1995, with a Wild T1000 theodolite and a Wild DIOR 3002S EDM; 12 to 18, GPS campaigns, between December 1995 and December 1996, with two Trimble 4000 Ssi receivers and accessories.

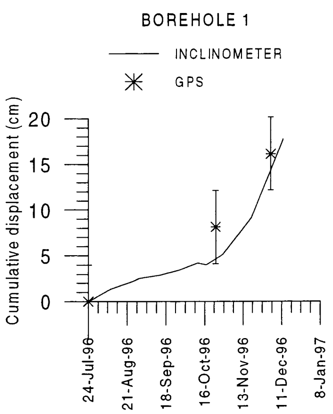
check wire extensometer displacements at several boreholes (Corominas et al., 1999). After calibrating the equation parameters of the wire, the results show an excellent match. Compared with the GPS, the extensometric technique has the advantage of being an automatic and continuous measuring system. Instead, measuring with GPS, in addition to distance increment, we could obtain the direction of the movement and the ΔZ.

Finally, at points that are in stable bedrock, the apparent movement gives us a direct check of the global reliability of measurements (Fig. 18).

The analysis of all previous results shows that the practical precision obtained in Vallcebre is within the range from 1 to 2 cm (expressed as RMS error, based on standard deviation), in good accordance with *theoretical* values derived in Table 4. It is also possible to evaluate the errors in terms of *tolerance threshold*, that is, the value not surpassed 99% of times. This error is also known in surveying as *maximum error*, and its value is, roughly, 2.5 times the RMS error. In a tentative way, on the basis of theoretical error considerations and practical results shown in Table 6, maximum error values have been established for Vallcebre GPS measurements. These error values are well suited to plot the error bars (1D), or the ellipse of error (2D) around the computed position, in order to include the 99% confidence area (Fig. 19). These error zones are a big help in the assessment of whether a point is moving or not. Although a certain tendency may be anticipated, it cannot be stated that a point has had a significant movement until its displacement surpasses the confidence zone.

The ellipses of error and the error bars are also helpful in gross error detection. In the Vallcebre project about 300 GPS positionings have been accomplished in 14 campaigns during 26 months. Only one big GPS error has been detected with no apparent explanation or special software warning, at point B1 (7 cm in X–Y plane and 13 cm in Z). On the other hand, eight human mistakes have been detected and corrected with the help of the field notes. These human errors were made in relation to point identification, antenna height measurement, or antenna height input into the receiver.

For a few times and places it has been impossible to position with GPS in a reasonable amount of time. For instance, in campaign number seven, we have not been able to obtain G5 target coordinates, due to the low number of satellites and high *P*DOP at this time. As has been stated, several points are partly under tree canopy. Sometimes, this forces us to extend logging times to get enough satellites or data to position the target. But in borehole B7 almost half the sky to the south is obstructed by foliage. At this borehole, we have frequently had many difficulties that make impossible GPS posi-



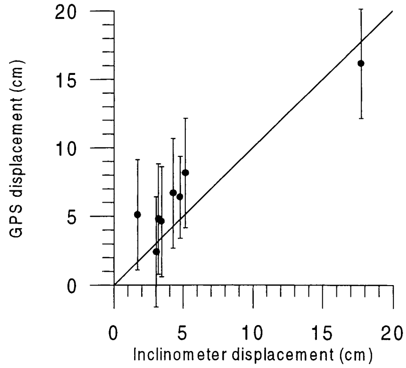


Fig. 17. Vallcebre GPS displacements compared with inclinometric results. (a) Inclinometric readings and GPS results versus time at borehole B1. (b) GPS displacement versus inclinometer displacements for different boreholes (B1, B3, B8, B10, B12 and B13) and dates.

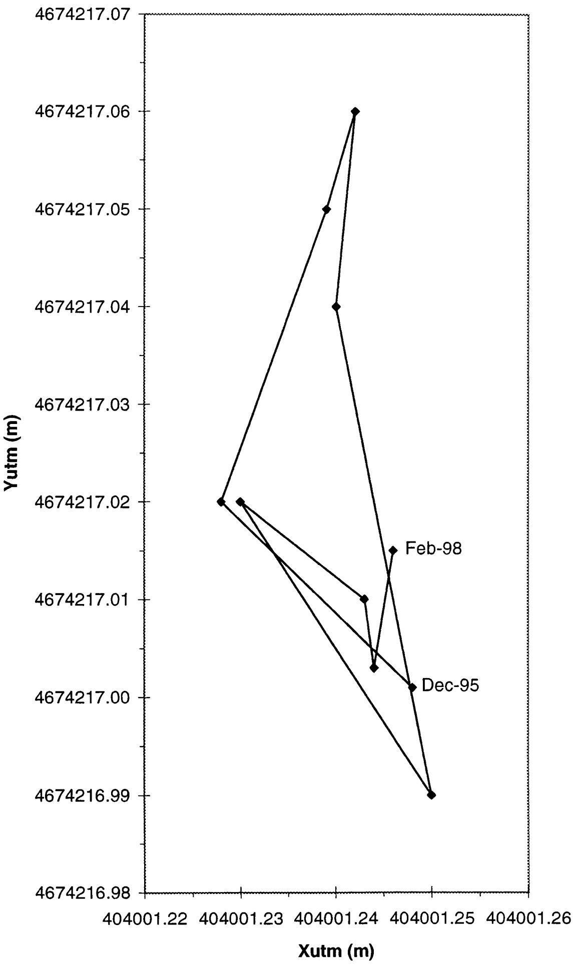


Fig. 18. Vallcebre landslide. Apparent horizontal displacement for fixed point G18, in stable bedrock, between December 1995 and February 1998.

tioning, even after 1 h waiting. Apart from this, using the FS method, the positioning of points partly surrounded by tree canopy has been feasible with fewer problems than expected.

5. Final remarks and conclusions

The GPS system has been fully operational since 1994. It enhances a great variety of engineering surveys, completing the classical surveying techniques. Furthermore, GPS makes feasible a set of completely new applications, even by non-highly specialised users. After a brief training period, the operator may be able to use the equipment properly for specific tasks.

The GPS receivers are now affordable. Nowadays, it is possible to buy a precision GPS equipment at prices in the same range as a good surveying total station. The cost/benefit ratio for GPS monitoring is good as far as large observational areas are concerned. In general, for extensions of about 1 km^2, both precision and productivity are of the same order of magnitude as classical surveying methods. However, precision with GPS is much more homogeneous in the three axes. The advantage of the GPS techniques over the classical surveying methods — especially concerning productivity and accuracy — increases with size of the observed area. In such cases, using GPS represents a big saving.

For an adequate performance of the GPS measurements, the antennas must have good sky visibility (for example, no interference caused by forests or mountains) to receive the satellite signals. However, using the FS method, the positioning of points partly surrounded by tree canopy has often been possible. Conversely, GPS does not require a direct line of sight between the stations. Therefore, baseline points may be at opposite sides of a hill or in different valleys.

In GPS surveys lasting for several years, discreet and sturdy target marks, such as rock engraving or steel pins, are recommended. In landslide monitoring, the inclusion in the network of fixed points outside the moving mass will guarantee the reliability of the set-up and the base point stability. Some kind of redundancy is also advisable.

The use of GPS relative positioning (two or more receivers) allows the cancellation of certain systematic errors. Thus, errors in terms of SA, the precision of orbital parameters, atmospheric delays and the local transformation parameters will not affect significantly the results. Even an imperfection in horizontal or vertical datum (base point coordinates, for instance) will not disturb the measurement of landslide displacements, provided that the same computation procedure and transformation parameters are always used.

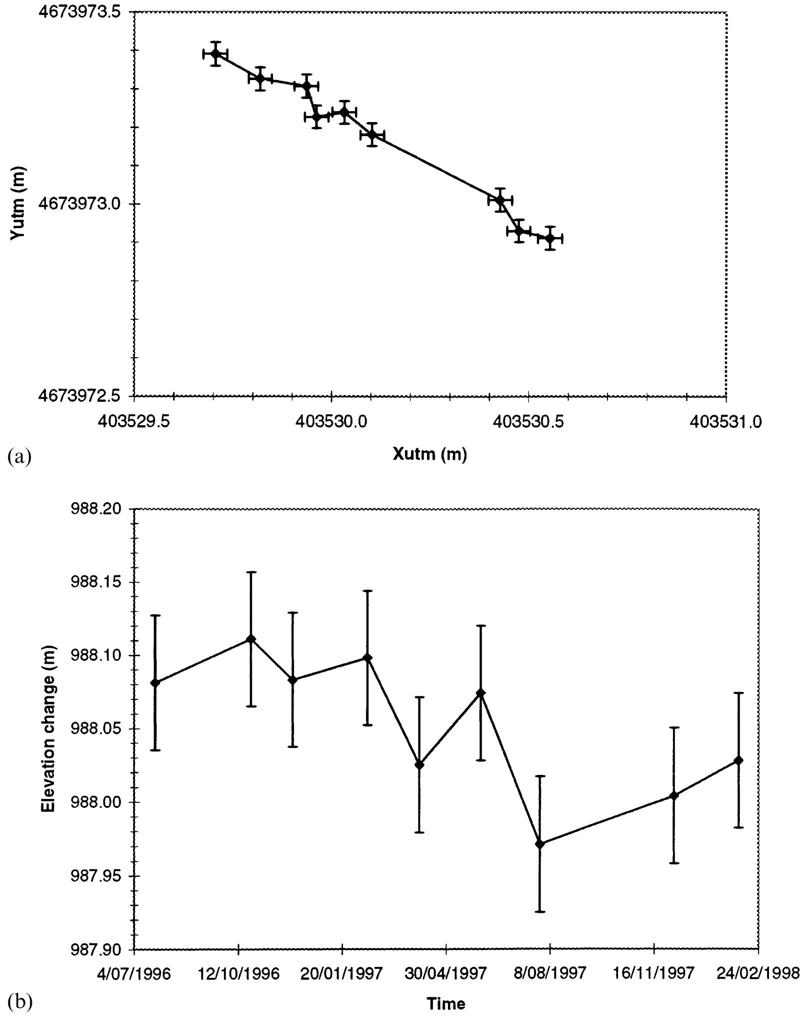


Fig. 19. Vallcebre landslide. Tentative 99% confidence zones around the positions calculated for borehole B1 at different times. (a) 2D error bars around the horizontal position from July 1996 (right) until February 1998 (left). (b) Error bars in the elevation vs. time plot for the same point and period.

Table 6
Tolerance (maximum error) established for Vallcebre GPS single positioning[a]

Method	Planimetric error (mm)	Elevation error (mm)
FS	30	46
RTK	40	62

[a] Errors are in terms of 99% confidence interval. Values adjusted for baselines in the range 1–1.5 km.

In contrast to conventional static GPS, with observation times in the range of hours, recent GPS methods, like FS and RTK, need only several minutes to a few seconds to get the positioning. Throughout the work presented in this article, it has been confirmed that the performance of these methods is appropriate for monitoring landslides and under all weather circumstances (rain, mist, fog, strong sunshine, by night).

The GPS measurements have been checked at several points against inclinometric measurements and fixed stable points. In the Vallcebre landslide, the estimated maximum errors of FS and RTK, obtained from theoretical and practical results, are respectively 30 and 40 mm in the horizontal plane, and 46 and 62 mm in the vertical plane. The same errors, but in terms of RMS, led us to state that, in the Vallcebre set-up, the precision of GPS positioning is 16 mm in the horizontal plane and 24 mm in elevation for the RTK method, and 12 mm in the plane and 18 mm in elevation for the FS method. In other cases, the precision may be increased using a forced setting up in the base and tripod setting up in the rover.

Rates of displacement of the Vallcebre landslide were up to 130 mm/month at the lower unit during rainy periods, with an average of 30 mm/month. During the 26 months GPS monitoring period, average velocities for the whole landslide ranged from 100 to 800 mm/year. Under such circumstances, the GPS has been revealed as a very useful tool for monitoring.

The 14 field campaigns carried out at the Vallcebre landslide have involved about 300 GPS positionings. Among them, only one supposed GPS error persists without explanation. Much care has to be taken to avoid human errors during setting up and antenna height measurement and input. At a few times and places it has been impossible to position with GPS in a reasonable amount of time.

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