TECHNICAL UNIVERSITY OF CIVIL
ENGENEERING BUCHAREST

RESEARCH REPORT
NUMBER 2

MODERN METHODS OF MONITORING SPECIAL CONSTRUCTIONS AND MACHINERIES

PhD student,
Eng. Mircea-Emil NAP

Coordinating Professor,
Prof. PhD. Eng. Petre Iuliu DRAGOMIR

BUCHAREST
2021

TABLE OF CONTENTS

1.INTRODUCTION

The research report will address issues related to the monitoring in time of buildings and machineries, using classical and modern geodetic methods and equipment, but also through non-geodetic sensors, the so-called measuring and control equipment, the term machinery to be used in the general sense of technical systems or installations.

The approach of the measuring methods implementation, for carrying out structures behavior in time works, is part of the current activities in the field, both in our country and internationally, at the industrial level, urban (construction) level, as well as at the level of special purpose land, the issue being of significant importance. [2]

Definition: Monitoring construction behavior in time is an activity that consists of measuring, recording, processing and systematically interpreting the values of the parameters that define the extent to which buildings maintain their requirements of strength, stability and durability. At the same time, construction behavior in time refers to the changes of position and shape of the assembly or of some of its elements, as well as to the notification of the appearance of some evolution phenomena, that will be able to affect the safety of the construction, thus, there is the possibility of taking early measures to prevent accidents and disasters. [2], [6], [10], [11], [12], [15]

The changes mentioned above are found in the literature with the names of *displacement* and *deformation*, where *displacement* defines the change in space of a point's position, located on a structure (construction) or land surface subjected to stress, and *deformation* is the change of the object's shape by enlarging or reducing the distance between specific points of that object. [13], [15]

Measurements performed in order to determine deformations, differ from most other types of geodetic measurements by the need to achieve higher accuracy, by the obligation to perform repetitive measurements (in several seasons, at different periods) and by the unique way of integrating different observation systems for obtaining results, with data as comprehensive as possible. [2], [10], [12]

Constructions have the possibility to be classified into three groups of precision, taking into account the size of the displacements / unit of time and the real needs of precision that help to define the values of displacements, and the importance class of constructions according to STAS 4273-83. [12]

The steps to be followed for carrying out the structures monitoring are, the implementation of the geodetic tracking network, the effective measurement for displacements deduction, the measurements processing, data analysis and presentation. [12]

For the results to be as expected, namely, that the measured elements to highlight the displacements and deformations and at the same time, the evolutionary dispositions, a plan will be needed, developed before the start of the monitoring works. This plan will in principle contain data on the time intervals at which the measurements will be carried out, the accuracy required depending on the objective pursued, the points of the geodetic monitoring network (category, number, positioning), the measurement methods and the instruments to be used for achieving the established accuracy and finally, mentions of the types of information processing, analysis and exposure methods. [10], [11], [12], [13]

1.1. Legislative framework

At the national level, the importance of construction and land behavior in time is highlighted by laws, government decisions, regulations and statuses, including:

- Stas 10493-76 of 1st January, 1976 - "Marking and signaling of points for monitoring the submersion and displacement of buildings and land"; [29]
- Law no. 8 of July, 1977 - "Ensuring the durability, operational safety, functionality and quality of construction" which was conceived after the earthquake of March of the same year, and took into account the regulation of durability, safety in operation, construction functionality, thus establishing the obligation to tracking in time of the construction, the framework of these works being detailed by methodological rules of law enforcement and the rules related to the content and manner of preparation, completion and maintenance of technical details of constructions. The organization of the works is thus structured as "current monitoring of all constructions in the country, through the care of their beneficiaries, with means of observation or measurement of current use throughout their existence", and "special monitoring of selected constructions on certain criteria, in execution of specialists, with sophisticated technical means, for periods determined by the achievement of objectives”; [30]
- Stas 2745-90 of 1st of March, 1990 - “Monitoring the construction displacements by topographic methods”, which replaced Stas 2745-69; [31]
- Government Ordinance no. 25 of 24 August 1992 stating that the monitoring of the construction behavior in time is mandatory for all buildings, especially those located on weak and unstable terrain or on artificially improved terrain; [32]
- Law no. 10/1995 consolidated in 2007 regarding the quality in constructions replaced the Law no. 8 from 1977, which stipulated that: “Art. 18. - The monitoring of the behavior in time of the constructions, is done during their entire existence and includes the set of activities regarding the direct examination or investigation with specific means of observation and measurement, in order to maintain the requirements.”; [33]
- Government Decision no. 766 of November 21, 1997 published in the Official Gazette no. 352 of December 10, 1997 - Regulation on the monitoring of the behavior in operation, interventions and post-use of constructions, which in Art. 5 states: “The monitoring of the behavior in operation of constructions is done by: current monitoring, special monitoring.

The modalities for carrying out the current or special monitoring - periods, methods, characteristics and parameters followed - shall be determined by the designer or expert, depending on the category of importance of the buildings and their other characteristics and shall be included in the technical data sheet of the buildings, which will also include recorded results of these activities"; [34]

- Stas 016-97 has replaced Stas 2745-90 and sets the methodology for determining the deformations of the foundation lands of some constructions, during their execution and operation by topo-geodetic methods. The 6 chapters of the stasis are: General, Scope, Reference marks and compaction marks, Carrying out measurements, Recording, processing and interpretation of observations, Construction displacement file; [35]
- The normative P 130-1999 regarding the activity of tracking the construction behavior in time, normative that responds to the provisions of Law no. 10/1995, and which is structured as follows: General provisions, Terminology, Current monitoring of construction behavior, Special monitoring of construction behavior, Obligations and responsibilities regarding the monitoring of construction behavior, Final provision, Annex 1-Romanian technical regulations in force on monitoring construction behavior , Annex 2-Indicative criteria for assessing the condition of buildings, Annex 3-Indicative list of phenomena to be considered during the current follow-up; [36]
- Law no. 123 of May 5, 2007 for the amendment of Law no. 10/1995 on quality in construction; [37]
- Government Decision no. 1231 of 2008 which replaces the Government Decision 766/1997 for the approval of some regulations regarding the quality in constructions. [38]

1.2. Basic Notions

The displacements and / or deformations that a construction can develop can be linear, angular or specific, the term construction being used with the suggestive meaning of the word, namely, for buildings, bridges, etc. as well as for equipment or machinery. [2], [12]

The groups of *linear* deformations and displacements are:

- Submersions - are caused by infiltration, or design errors, or construction loads and represent the vertical top-down movement of a foundation and foundation ground; [12]
- Flexures - are represented by bends of some components (pillars, beams, plates) caused by vertical or horizontal stresses;
- Bulges - the opposite phenomenon of submersion, represented by vertical movements from bottom to top of foundations and foundation lands; [12], [15]

- Cracks / fissures - are formed due to non-uniform displacements or deformations and are fractures in the plane;
- Linear tilts - are not harmful to the structure, and are produced by stresses with non-uniform distribution;
- Horizontal displacements - the phenomena that lead to their occurrence are landslides and can also be influenced by ground water or overground water, being characterized by the change of position of the building or only some of its assemblies, in the horizontal direction; [2], [12], [15]

Angular displacements and deformations are caused by stresses, infiltrations or landslides, and are defined as vertical rotations, called angular and horizontal tilts known as twists. [2], [6]

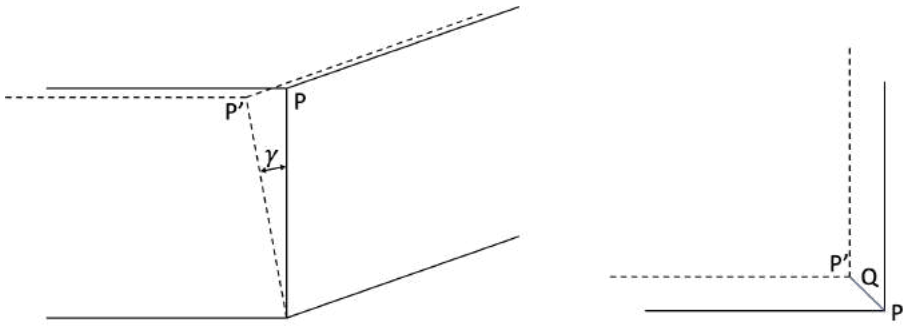

Figure 1.1 (left) Angular tilt, (right) Linear tilt.

Specific deformations are the effect of external forces (tension or compression) and are defined as elongation or shortening of some elements of the construction.

Other ways to classify deformations and / or displacements are according to the duration of the phenomenon as following:

- Oscillations - the time during which it takes place is extremely short, ranging from 1-10 seconds for construction and 0.01-1 second for various machines.
- Short movements - fall within 10 seconds - 10 days
- Long-term movements - 10 days - 10 years (movements of the earth's crust can extend this period up to 100 years) [12]

Displacements and deformations can also be found using two types of measurements, namely, relative or absolute measurements. When determining displacements between specific points of the structures, with instruments mounted on the structure itself, which move with the construction, but without using external references, the measurements are called relative; on the other hand, if the displacements of the defining points of the construction are determined by means of instruments stationed outside the construction, which relate to networks whose points are not positioned in areas of influence of different factors to which both the construction and the ground are subjected, the measurements are described as absolute. [2], [10], [12] In order to be able to achieve the main goal of monitoring constructions and machineries, it is necessary that these works to be carried out within geodetic networks with a particular character, corresponding to the needs of this activity (monitoring). This type of network will have to follow some fundamental principles, such as the fact that the shape of the network will be defined by the position of the points that compose it, or that the positioning of the points will be performed in each measurement era after a rigorous block compensation. [12], [15]

The main components of a geodetic monitoring network are:

- monitoring marks, which aim to accurately reproduce the values and directions of displacement of the structures on which they are mounted;
- observation stations, built with piles with deep foundations made of reinforced concrete and equipped with center align elements, these stations having the role of facilitating the precise installation of observation instruments, being built outside the influence area of the factors that subject the construction to stress; [12]
- checkpoints, which are located 200-300 m away from the building, without being affected by it, and help to establish potential displacements of the observation stations;
- the guidance points, which ensure a high precision in orientation, both from the control points and from the observation stations, being also fixed outside the area of influence. [6], [12], [15]

Following the points position determination, the tracking networks can be divided into:

- Planimetric networks, where the points will be determined only horizontally in a two-dimensional coordinate system (x, y); [12], [15]
- Altimetric networks, where the positioning of points with a single (z) coordinate is used, but albeit with less precision, the planimetric position will also be defined for the points of this type of network; [12], [15]
- Three-dimensional networks, of which (x, y, z) rectangular coordinate systems are used;

Another classification of the monitoring networks was made according to the nature of the observations made within them, thus there are *triangulation networks* for angle measurements, *trilateration networks*, for distance measurement works, and combined networks. [11], [15]

Monitoring networks are made up of several types of points, thus forming 3 types of networks called:

- Complete networks, comprising all the points listed above, and reciprocal sights are provided between the control points and the observation stations, while sighting can be performed to non-stationary monitoring marks on the target from the observation stations (for monitoring first-order precision structures); [11], [12]; [11], [12]

- Incomplete networks, in which the points mounted on the objective are stationary, but reciprocal sightings cannot be performed between control points and observation stations (they also include all types of points);
- Superficial networks, used to monitor either lands subjected to landslides or easily accessible buildings (observation stations usually replace monitoring marks and also include checkpoints and observation points); [12]

Once the monitoring issues have been established, networks with sides of average length of up to 10 km, or local monitoring networks with sides of 50 m - 1.5 km will be formed.

Knowledge of an initial state is imperative when discussing the determination of displacements and deformations and these values are found in a first series of works, defined as the *initial measurement era (basic measurement)*, which needs to be structured in such a way that the measurements can be repeated at set periods of time, which means maintaining the characteristic points. Therefore, at the established intervals, new measurement eras will be performed, in order to be compared with the basic measurement, and / or between them, these moments must be chosen with greater care in order to record as accurately as possible any difference. [2], [12], [15]

2. CLASSIC METHODS OF MONITORING CONSTRUCTIONS AND MACHINERIES

2.1. Geodetic measuring methods for the determination of horizontal displacements

The main methods used to monitor horizontal displacements are *the trigonometric method*, *the alignment method* and *the polygonometry method*.

2.1.1. The trigonometric method

a) Micro-triangulation

As mentioned above, since there are several types of measured elements, the planimetric geodetic networks are divided into 3 categories as follows:

- Triangulation networks (only for measuring planimetric angular directions);
- Trilateration networks (only for defining distances);
- Triangulation-trilateration networks (combined measurements of distances and horizontal angles) [12]

The method is recommended for small-scale networks, due to the inconveniences created by the atmospheric conditions and the points of the network must be represented according to a projection plan, which means that the observations also need to be reduced to that system. Prior to the advent of electro-optical instruments for measuring distances, this technique was based on the creation of local geodetic monitoring networks. [5], [9]

It is applicable to the determination of the planimetric displacements and / or deformations magnitude of the monitoring marks, rigorously mounted on the object, in relation to a reference system. [12], [15]

In addition to designing and determining the geodetic tracking network and performing measurements in each measurement era, the most important step in a monitoring work performed with this technique, *consists* in the compensation, or rigorous processing of the performed observations. [4], [5], [10], [12]

b) Micro-trilateration

The method is recommended for medium size geodetic tracking networks, being used more often after the advent of instruments for measuring distances based on electromagnetic waves. Also, before compensation, the distances must be reduced physically, geometrically, to the projection plane, for this being necessary to find out the temperature and pressure of the target point and the station point, and the elevations of the points, together with the ellipsoidal altitude. [12], [15]

Physically reduced distances and points altitude can be introduced directly into the compensation at local geodetic tracking networks, without having to be reduced to the ellipsoid surface and to the projection plane. [15]

As in the case of micro-triangulation we will have a functional-stochastic model, with similar correction equations:

- Correction equation for a distance measured between two new points *i* and *j*:

$$v_{ij}^D = A_{ij}dx_j + B_{ij}dy_j - A_{ij}dx_i - B_{ij}dy_i + l_{ij}^D$$

- Correction equation for a distance measured between an old point *i* and a new point *j*:

$$v_{ij}^D = A_{ij}dx_j + B_{ij}dy_j + l_{ij}^D$$

- Correction equation for a distance measured between a new point *i* and an old point *j*:

$$v_{ij}^D = -A_{ij}dx_i - B_{ij}dy_i + l_{ij}^D$$

A_{ij}, B_{ij} – distance coefficients (calculated from provisional coordinates)

$$A_{ij} = \frac{\Delta x_{ij}^0}{D_{ij}^0} = cos\theta_{ij}^0$$

$$B_{ij} = \frac{\Delta y_{ij}^0}{D_{ij}^0} = sin\theta_{ij}^0$$

$$l_{ij}^D = D_{ij}^0 - D_{ij}^*$$

l_{ij}^D – free term, D_{ij}^0 – distance calculated from coordinates, D_{ij}^* - measured distance

As in the case of micro-triangulation, a weighting can be added to each correction equation.

$$P_{ij}^D = \frac{1}{\left(S_{ij}'^D\right)^2};$$

$S_{ij}'^D$ – standard deviation

$$S_{ij}'^D = a + b \cdot D_{ij}[km]$$

The notions of accuracy will be calculated for both Micro-trilateration and Micro-triangulation, as:

- Standard deviation of the unit of weight:

$$\sigma_0 = \pm\sqrt{\frac{V^TPV}{m-n}}$$

m – the number of measured azimuthal distances / directions

n – number of new unknowns

- Standard deviation of an individual measurement:

$$\sigma_i = \pm\frac{\sigma_0}{\sqrt{P_i}}$$

- Standard deviation of unknowns:

$$\sigma_{Xi} = \pm\sigma_0\sqrt{N_{ii}^{-1}}$$

- Total standard deviation of a point:

$$\sigma_t = \pm\sqrt{\sigma_{Xi}^2 + \sigma_{Yi}^2}$$

- Average standard deviation of the network:

$$\sigma_{network} = \pm\sqrt{\sum_{i=1}^{2} \sigma_{ti}}$$

[4], [5], [9], [15]

2.1.2. Alignment method

This technique considers the planimetric positioning of an alignment of monitoring marks, mounted on a construction with a linear configuration, these types of structures allowing the mounting of points on the same alignment. This will require, in addition to the points fixed on the object, to be also end points, fixed outside the area of influence of the construction, the principle of this technique consisting in determining the changes in the position of some points in relation to a vertical plane. [10], [12]

Given the simplicity of this method, it can be used for projects that require repetition of measurements at short intervals. [12]

If the conditions do not allow the installation of the base points on stable land, they can be located on land with poor stability, or even on construction, but this requires combining the method with both micro-triangulation and micro-trilatation, the deviation of each point being determined according to the displacement of the basic points of the alignment. [10], [12]

Whether the planimetric displacements are determined by observing a single alignment over its entire length, or by observing sections of the same alignment or by observing multiple alignments, or whether the determined position changes will be relative or absolute, there are several methods or techniques, which are used in areas of different distances, from centimeters to hundreds of meters, with accuracies ranging from one millimeter to a few micrometers. [10], [12], [13]

a) Sighting procedure along the alignment

The basic instrumentation used in this process consists of alignment telescopes with magnifications up to 60x and a sensitivity of 10", equipped with detachable levels, which move only vertically, not equipped with any degree circle (horizontal / vertical), and as attachment elements, will be used mobile and fixed sight marks, equipped with micrometric screws for fine movement, 100mm being the maximum displacement they can reach. [6], [12]

There is also the possibility of using alignment telescopes equipped with micrometers, thus making it possible to move in both directions (horizontal and vertical).

Figure 2.1 Alignment telescope with micrometers.

An alignment will be performed on the longitudinal central axis of the structures, or parallel to it, which will be materialized with two concrete pilasters at each end, outside the area of influence, equipped with forced centering devices, and the movable targets will be placed in the targeted points, the operator will adjust “0” on the graduated ruler, in alignment, by means of the micrometer screw. The displacement will be recorded at least 3 times, for each point in the alignment, the values of the readings must be within a tolerance of 0.3 mm. At least 3 series of determinations will be performed within each measurement epoch, the observations being made from both ends, the average of the values being calculated for each epoch.

The horizontal displacement, δ , corresponding to each monitoring mark, is calculated with the relation: $\delta = a_{total}^{j} - a_{total}^{0}$, where a_{total} represents the average of all observations made during the current measurement epoch. [12]

An analysis of the stability of the piles will be performed in each measurement cycle by relating them to a geodetic tracking network, so that if changes in the positions of the support points are confirmed, subsequent measurements will be reduced and centered accordingly.

Only the transverse displacements can be determined by applying this procedure of the alignment method.

b) The procedure of parallactic angles

It is useful for determining the transverse displacements of alignment monitoring points, and involves the use of high-precision instruments to measure the parallactic angles between the directions starting from the ends of the alignment towards the marks mounted on the object and the main alignment. [6], 12], [13]

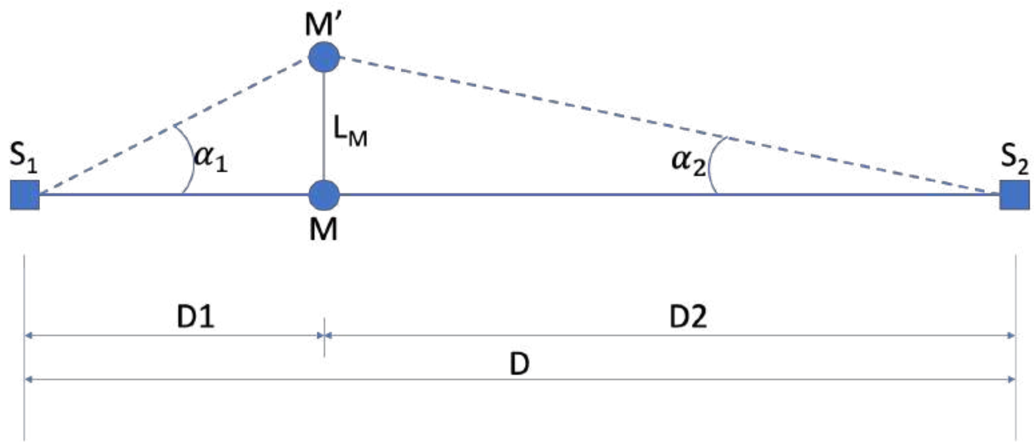

Figure 2.2 The procedure of parallactic angles.

The values of the deviation of the monitoring mark M from the initial era will be calculated according to the following relations, after they have been determined from both end points:

$$l'_M = \frac{\alpha_1}{\rho^{cc}} D_1$$

$$l''_M = \frac{\alpha_2}{\rho^{cc}} D_2$$

α_1, α_2 – parallactic angles

D_1, D_2 – distances from the control point to the end [12], [15]

For the initial measurement period, the final value of the point deviation will be calculated using the arithmetic mean:

$$L_M^0 = \frac{l'_M + l''_M}{2}$$

The deviation magnitude of the monitoring mark corresponding to the current measurement epoch L'_M will be calculated analogously.

The planimetric displacement of the monitoring mark is calculated according to the difference:

$$\Delta L_M = L'_M - L_M^0$$

[10]

c) Errors in optical alignment procedures

1) Baseline is too short:

A target with a tolerance of 0,005 mm is sighted at a distance of 2 m from the alignment telescope, which results in a maximum error of 0.005 mm. The next measurement will take place at a distance of 10 m, for instance, using this base, then the error entered will increase 5 times in relation to the tolerance of the sighting target, reaching 0.025 mm. [3], [11]

The correct use of this technique is thus determined to be in the opposite direction, namely, the placement of a sighting target at the greatest distance from the telescope, in this case 10 m, the error introduced being equal to the tolerance of the sighting target (0.005 mm) and only then causing changes in the position of some closer points, in this example, at a distance of 2 m, in which case the error introduced decreases proportionally to 0.001 mm. [3], [11], [12]

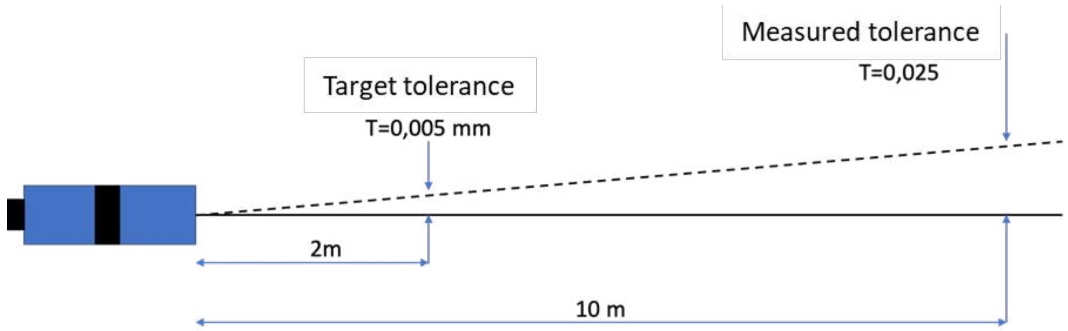

Figure 2.3 Sight line set on a target at 2 m from the telescope and determination at 10 m.

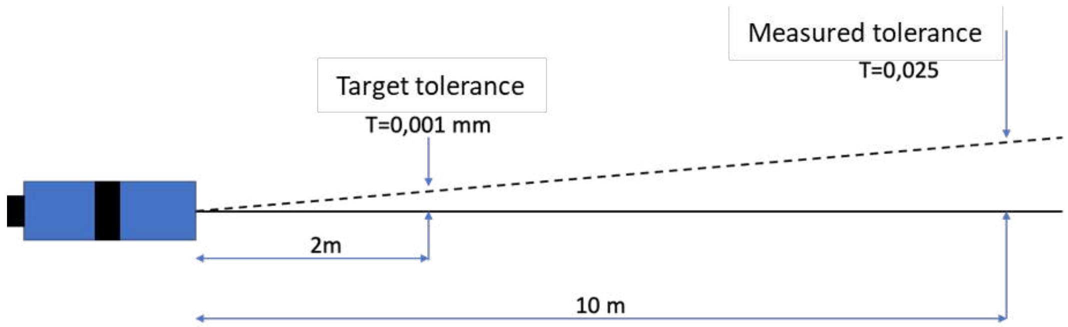

Figure 2.4 Sight line set on a target at 9 m from the telescope and determination at 2 m.

2) Atmospheric refraction and curvature of the earth:

The moment when the effect produced by the curvature of the earth is considered to be taken into account is when, at long distances, a horizontal surface established with a level and a line of sight verified with a telescope are compared, so that if the surface is determined by means of a controlled gravity instrument, it will be curved and if the respective surface is checked with a telescope, by processing of the measurements on the targets, it will result that the surface is not horizontal, existing also the problem of atmospheric refraction, will cause the deviation of the telescope sight line from the correct line. [3]

Density changes occur at different altitudes due to changes in the temperature of the air layers. If these changes are gradual and eventually constant, a target that appears to be at point *A* will actually be placed at point *B* because the sight line through the atmosphere will be a smooth curve.

According to these data, a level surface will deviate, from the true tangent to the earth's surface, in a constant and calculable way, and a sight line established with a telescope, being exposed to variations caused by atmospheric refraction, will suffer a deviation for which only average values can be obtained. [3]

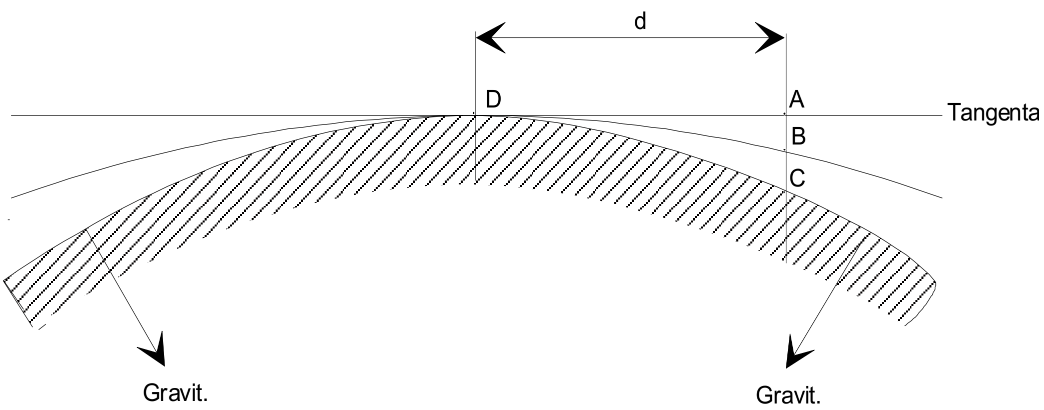

Figure 2.5 Effect of atmospheric refraction and Earth curvature. [3]

where: DA – sight line established by a telescope in ideal conditions (vacuum)
DB – sight line established by a telescope through the air (constant temperature variation)
DC – surface made with a tool having controlled gravity

DISTANŢA [mm]	A - B [mm]	B - C [mm]	A - C [mm]
2,50	0.000075	0.0049	0.00056
5	0.00030	0.0020	0.0023
10	0.0012	0.0078	0.009
25	0.0074	0.049	0.056
50	0.030	0.20	0.23
75	0.067	0.44	0.51
100	0.12	0.78	0.90

Table 2.1 The values of deviation at different distances.

Where:

A-B – deviation from the tangent of a sight line realized with the telescope (average values only)

B-C – deviation of a line established with a level from the sight line determined by a telescope (average values only)

A-C – deviation of a sight line verified with a level from the tangent to the earth's surface (straight) [3]

3) Unstable structures:

Small deviations within a structure can occur due to various reasons such as instability of the foundation, or due to moving loads along the support, but the most common cause is the temperature variation between the ceiling and the floor, its effect, although often unnoticed, causes significant errors. Therefore, at a temperature difference of 5 °C between the base and the upper side of a calibrator, which in theory are equal, there will be an elongation of 0.38 mm of the upper side, which causes a displacement of 20^{cc} of the lateral sides. [3]

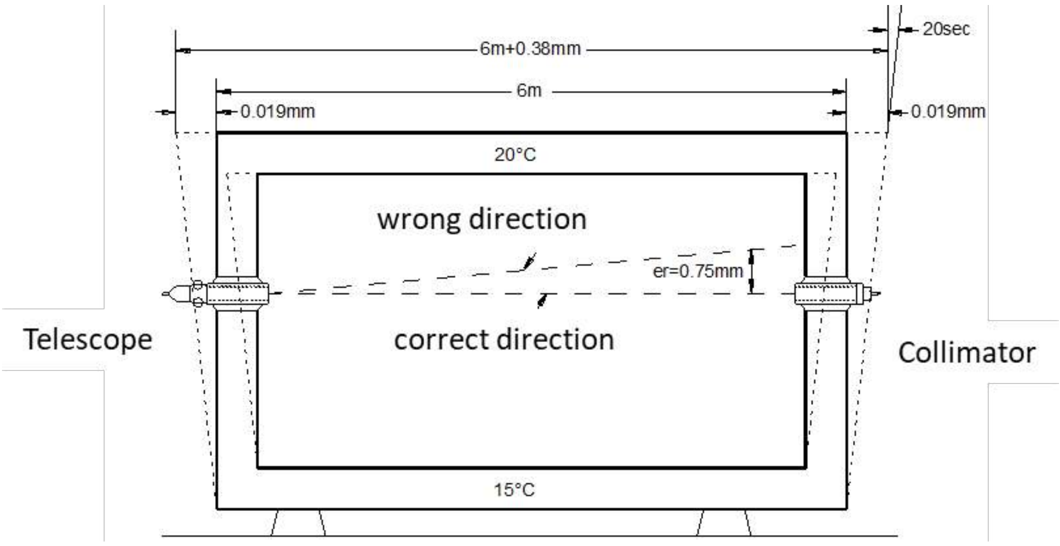

Figure 2.6 Elongation effect.

The sight error in the optical alignment has the greatest influence in terms of the accuracy of determining the transverse deviations of the intermediate points, so in theory, the centering errors are reduced to 5-10 μm if using regulated nuts and devices with spherical centering.

The length of an alignment may not exceed 50 m because, under favorable conditions, the sight error may be $\pm 1{,}5^{cc}$, corresponding to a transverse error of$\pm 0{,}4\ mm$. [3]

d) Checking the alignment of a machine part

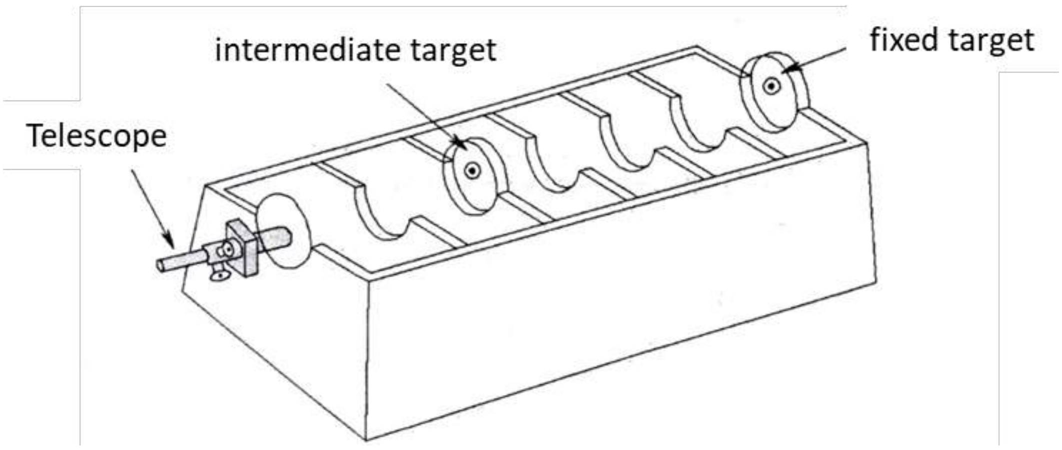

Figure 2.7 The method of checking the alignment of a series of holes in a machine part. [3]

The alignment telescope is fixed in A bearing, by means of a flange so that the sight axis coincides with the holes axis, in the last of which (G) being mounted a washer with incorporated target sight. Once the sighting of A target has been carried out, the theoretical axis of the whole set is determined. All intermediate sight targets will be measured in all bearings, following that from the obtained values (angles and distances) to calculate the values for adjustment. [3]

If for instance, a part is too short for a classic check, the alignment telescope will be mounted separately from it, following that on the equipment will be installed two targets, of equal heights, collinear with telescope. There are situations when, for ease of work and for obtaining a sight line without obstacles, special stands are used both for the telescope and for the targets. [3]

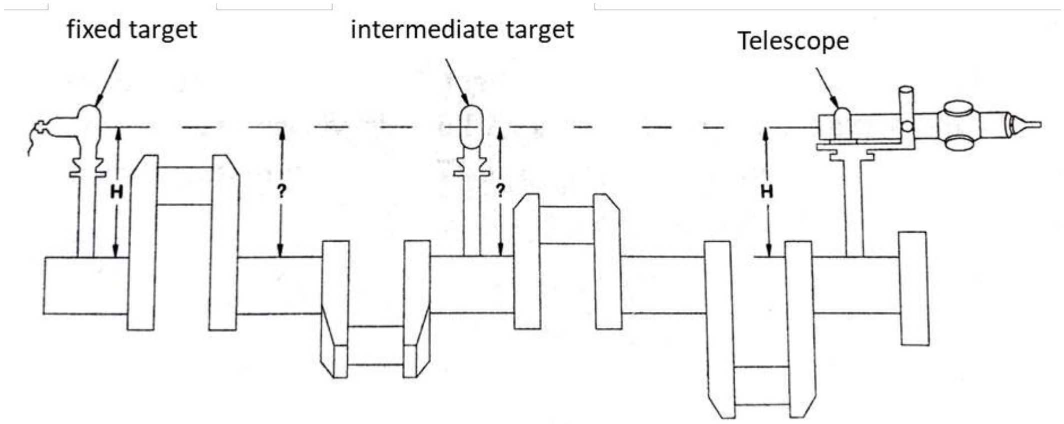

Figure 2.8 Checking the bearing alignment of a crankshaft without removing the engine. [3]

In conclusion, the alignment telescope can be used both in classic construction monitoring works, but in particular it is found in the alignment and verification of longer guides in machine construction, in linearity and / or curvature measurements in equipment construction, in linearity verification guidelines in the field of elevator construction and in checking the loading or floor plan of foundations in the machine building industry.

e) Collimation method

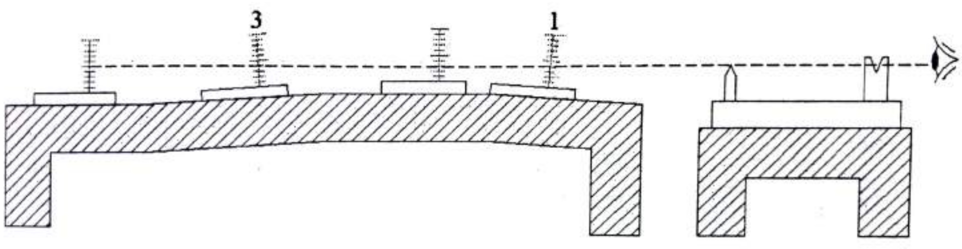

Figure 2.9 Checking the surface of an equipment. [3]

In the example presented above (Fig. 2.9), the sight axis was established by the intersection of the reticular wires and the center of a leveling rod at the end of the alignment. It is observed that the surface is not parallel to the sight axis although, in points 1 and 3, the height measured between the part and the axis is similar. For this reason, the use of a procedure called collimation will be required to establish the parallelism between the object and the sight axis. It is based on a simple principle, namely, the alignment of two infinitely focused telescopes at which the image of the aiming bars materializes in the object space, so that their optical axes are parallel. The light sources behind the marks will be replaced with a collimator, which projects the image of the aiming bars to infinity, them being therefore placed in the focus plane of the objective. [3]

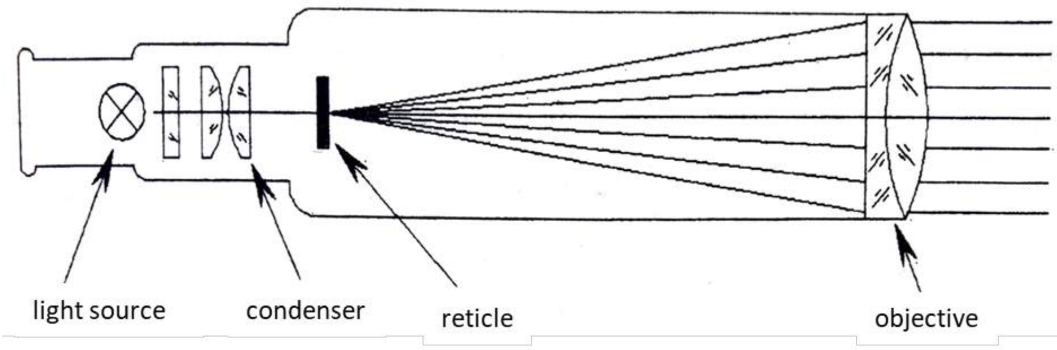

Figure 2.10 Collimator. [3]

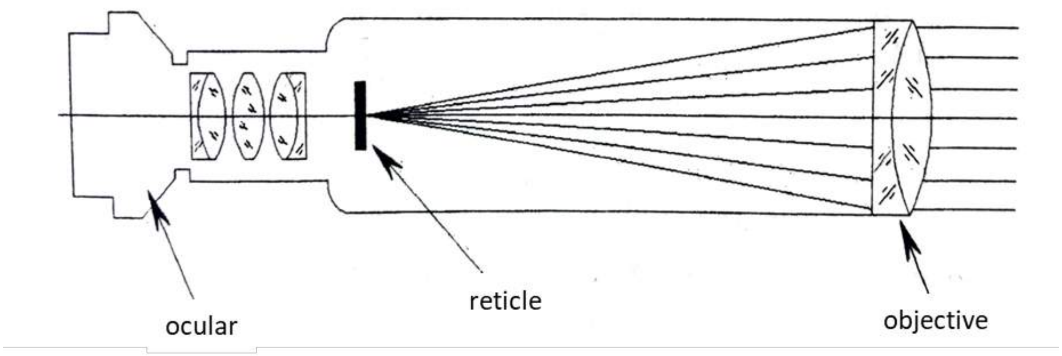

Figure 2.11 Telescope. [3]

As an optical principle, in a collimator, light rays pass through the focus of the system, which enter the lens, and come out of the objective like a beam of rays parallel to the main axis. The use of a collimator together with an infinitely focused telescope allows to measure the differences in direction between the telescope and the collimator.

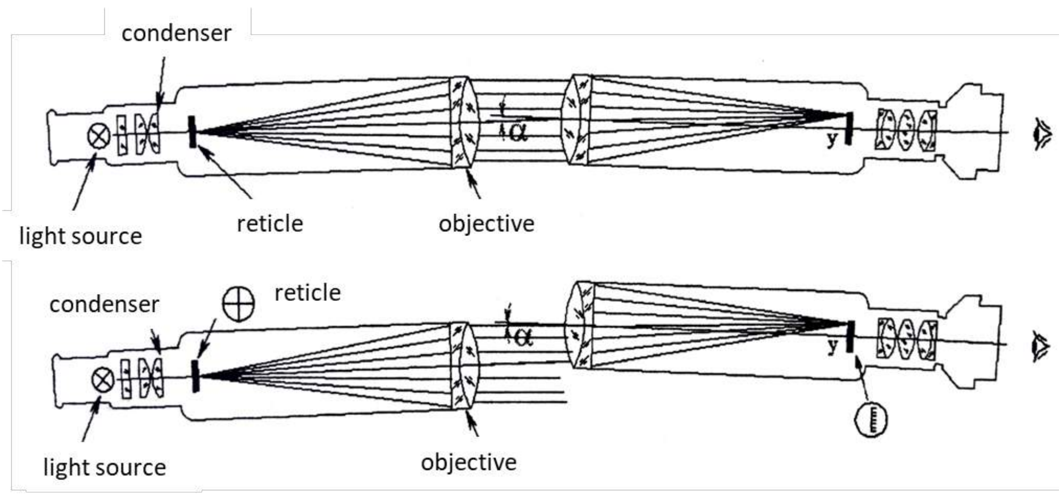

Figure 2.12 Collimator + telescope. [3]

When an angle (α) is found between the two axes, the image of the features of the collimator reticular plate will be displaced by y from the center point of the aiming bars inside the telescope.

If, on the other hand, the axes are parallel, the features of the collimator aiming bar are considered to be centrally placed to the telescope aiming bar, but this does not cause the collimator axis to be centrally placed in a concrete way, in order to establish this, the telescope will be focused on a target, which will be mounted on the collimator objective, so that the deviation can be detected, and if that deviation is 0 both horizontally and vertically, and if on the collimator aiming bars, the reading was in the center, then the axes of the two instruments are both parallel and centered. [3]

The angular displacement θ which will facilitate the calculation of the value of the linear quantity q (deviation of the collimator axis in relation to the direction of the fixed sight axis of the telescope) will be measured with the ocular micrometer *2*:

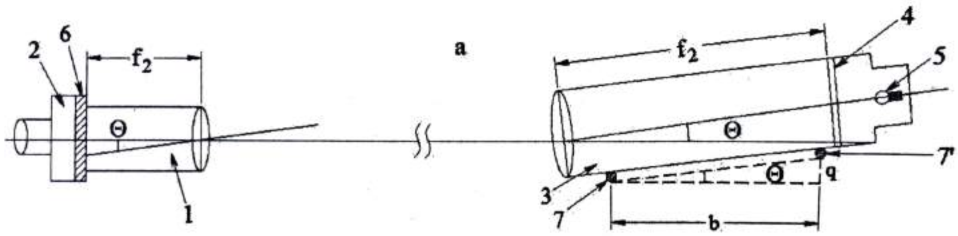

Figure 2.13 Using the telescope and collimator. [3]

where: 1 – telescope, 2 – ocular micrometer, 3 – collimator, 4 – collimator reticle, 5 – light source, 6 – telescope reticle, 7 – collimator supports

$$q = b \cdot \frac{\theta^{''}}{\rho^{''}}$$

$$\theta^{''} = d^{''} \cdot n$$

$$q = b \cdot \frac{d^{''} \cdot n}{\rho^{''}} = k \cdot n$$

$$k = \frac{b \cdot d^{''}}{\rho^{''}}$$

where b – base of the device, $d^{''}$ – the value of a division of the micrometer drum, n – number of divisions.

In terms of accuracy, the standard deviation σq, which helps to find the transverse displacements q, is calculated as follows:

$$\sigma = \left(b \cdot \frac{\sigma_{\theta^{''}}}{\rho^{''}}\right)^2 + \left(\frac{\theta}{\rho} \cdot \sigma_L\right)^1 = \left(b \cdot \frac{\sigma_{\theta^{''}}}{\rho^{''}}\right)^2 + \left(\frac{q}{b} \cdot \sigma_b\right)^1$$

where σ_θ – standard deviation of angle measurement θ, σ_b – standard deviation of knowledge of the base

Given that b is known with high accuracy, and q has small values it is considered that:

$$\sigma = b \cdot \frac{\sigma_\theta}{\rho}$$

Although the accuracy of q does not depend on distance, for long lengths there are errors in determining θ, so $\sigma_\theta = 0{,}5'' \dots 0{,}8''$ for d < 400 m. [3]

2.1.3. Polygonometric method

It is used to determine the planimetric displacements of buildings with large surfaces and curved shapes, without visibility and at the same time when the application of the trigonometric methods presented above would be impossible. It is also used with high efficiency in case of landslides, where a high accuracy is not required and the displacements have high values. [12]

Upon verifying the stability of the reference points within the monitoring network, a polygonometric network will be set up, which will consist of some reference points within the geodetic network and monitoring marks fixed on the construction or land being monitored. [10], [12]

Because the method is based on cyclically performed operations to accurately determine the angles and sides of the path, the meaning of quantities will be as follows:

β_i – the angles measured in the initial epoch

β_i' - angles measured in the current epoch

D_i – distances measured in the initial epoch

D_i' - distances measured in the current epoch

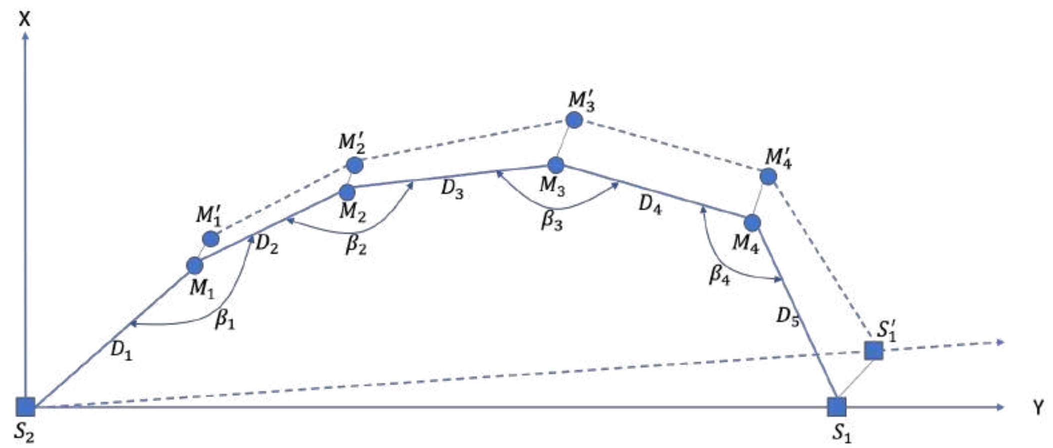

Figure 2.14 Polygonometry method.

The first stage in solving consists in compensating the orientations that were calculated using the values of the measured horizontal angles, followed by the calculation of the non-closing on the orientations and the application of the unit correction. [4], [6], [12]

The second stage consists in calculating the values of the coordinates of the points within the route, the calculation of the non-closing on the coordinates, and ends with writing the system of equations. By solving it, the values of the coordinates of the points on the polygonometry route result. [4], [6], [12] The recommendation, to obtain the desired results, is that the polygonal lines should have directions as perpendicular as possible to the main direction of possible displacements.

2.2. Geodetic measuring methods used to determine vertical displacements

2.2.1. Geometric leveling method

a) Generalities

It is the most widely used technique in monitoring special constructions such as viaducts, locks, nuclear reactors, etc.

Similar to most monitoring methods, this technique requires the development of a geodetic monitoring network depending on the size or shape of the object. This network will contain *control points* represented by submersion marks mounted on the structure and *reference points* (fixed landmarks) materialized outside the area of influence of the construction. [6], [10], [12]

In principle, the submersion marks will be distributed along the axes of the foundations, in order to find the directions, at the expansion joints, where the most precarious geological conditions would be. An important detail that must be taken into account when materializing them is that the fixation in the resistance components of the construction must be made in such a way that the levelling rod can be placed upright. [11], [12]

At the same time with the materialization of the submersion marks, reference marks will be made, at least three months before the beginning of the works, in order to achieve maximum stability. In this stage, the station points will be also made, so that both their position and number can remain the same at each measurement series. [12]

The precision imposed to define deformations and displacements is the element that practically requires the use of a certain type of tool and, definetly, a certain technique.

The geometric leveling network, composed of landmarks at which measurements are made in order to find out the level differences and also the lengths of the routes, must subsequently be rigorously compensated, and for this, the necessary data are:

- Distances covered in order to determine differences in altitude;
- Level differences determined by geometric leveling, their number being higher than that of the unknowns in the model, concretely higher than the number of landmarks that do not have known altitudes;
- Elevations (H_i) of several or at least one leveling mark;
- Provisional elevations (H_i^0) of new landmarks;
- Any other details useful in making the matrix of the observation’s weights and, implicitly, of the functional-stochastic model. [12], [15]

Rigorous compensation using these concepts will result in absolute values calculated based on known notions of point elevations in the chosen system, and in compensated values of differences in altitude on the measurement path and accuracy of determination in the processing.

b) Geometric leveling in industry

The classic techniques for performing leveling works need to be supplemented when it comes to their use in the industrial field, so that they do not exhibit errors greater than 0.2 mm. In today's precision installations, one of the most common methods is that of precision geometric leveling with short sighting distances. [3]

Vertical temperature variation frequently affects measurements and in the industrial field, there are also limitations related to local conditions, where a classical approach for the determination of differences in elevations would be impossible to perform, therefore it has to proceed as follows [3]:

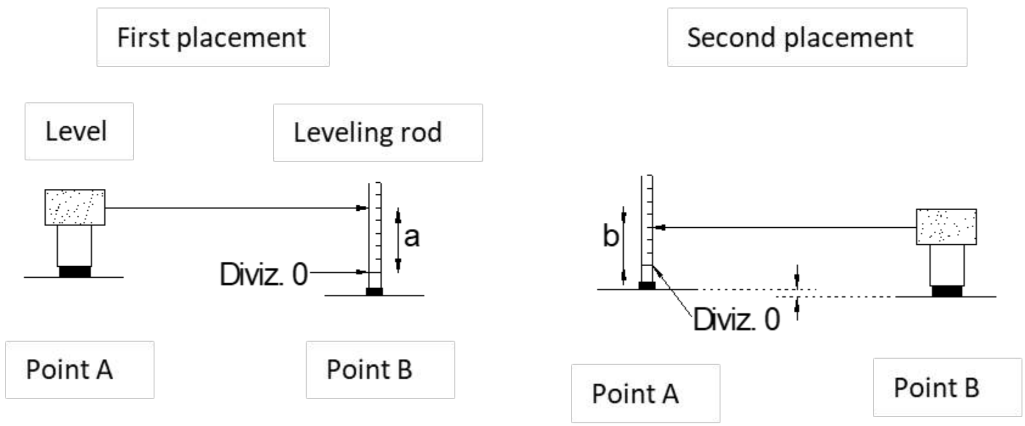

Figure 2.15 Direct determination of level differences in machineries components. [3]

- points A and B are on the horizontal surface of the machinery for which we want to determine the level differences
- the device is adjusted horizontally on point A upon installation in the station
- a reading is performed at point B on a leveling rod (there is no constraint for division 0 to be on the surface of the object)
- exchange places between the instrument and the leveling rod, making sighting to point A this time
- the level difference to be determined will be given by half of the difference between the two readings.

For distances of 10 to a maximum of 15 m, the procedure makes it possible to determine elevation differences with errors of not more than 0,05 mm. [3]

2.2.2. Trigonometric leveling method

The major disadvantage of this method is the effect of atmospheric refraction on zenith angles; therefore, it is rarely used to determine submersion, being mainly used for land monitoring, but not for construction. In order to reduce this effect of refraction, it is recommended to measure both zenith angles by mutual observations and, definitely, these observations should be performed simultaneously. In most cases, however, angular measurements are unilateral in current practice, thus the effect of atmospheric refraction is significant. [10], [12], [15]

To calculate the level difference, the slope angle and the distance between the two points are used:

$$\Delta h = D \cdot tg\alpha$$

or

$$\Delta h = D \cdot ctg\varphi$$

(if the device measures the complement of α, φ)

The formulas presented above are valid when practicing trigonometric leveling at short distances, for situations where the distance between the points is large, it is necessary to:

$$\Delta h = D \cdot tg\alpha + I - S$$

where I – the height of the instrument, S – sight elevation on leveling rod [3], [10], [12]

The correction for the effect of atmospheric refraction and also for the effect of earth curvature must be taken into account when sight exceed 300-400 m. [15]

$$\Delta h = D \cdot tg\alpha + I - S + C$$

where C – sphericity and refraction correction

$$C = D^2 \frac{1 - K}{2R}$$

where R – average radius of curvature, K – refractive index

When creating the leveling network, for tracking the behavior in time of different objects, the geometric leveling method is taken into account, so that the necessary accuracy can be obtained, and indeed, the actual measurements will be performed by trigonometric leveling where appropriate. [12]

The corrections for the measured values and for the refraction and the elevations of the new points, will result after writing the linear system of correction equations, its normalization, and solving the normal system of equations, which will lead to the final calculation of the most probable values for both sizes, the works being closed by the calculation of the precision elements. [6], [12], [15]

2.2.3. Hydrostatic leveling method

a) Generalities

The technique of tracking the buildings behavior in time by hydrostatic leveling is based on the principle of communicating vessels, being used when the places where measurements are to be performed are inaccessible to other instruments (eg: inside buildings), having the ability to achieve high accuracy. [3]

In the standard mode of the technique, graduated tubes are fixed on the object, in the monitoring points, which are connected to each other by a hose, the level difference resulting from the difference between the readings on the tubes, and in the case of double circuit, on one of the paths, the liquid flows freely, while the second path contains air to equalize the pressure, and the level is defined by a capacitive sensor. [3]

Bernoulli's simplified equation:

$$p + \rho \cdot g \cdot h = const$$

where p – atmospheric pressure

ρ – liquid density

g – gravitational acceleration

h – the height of the liquid column

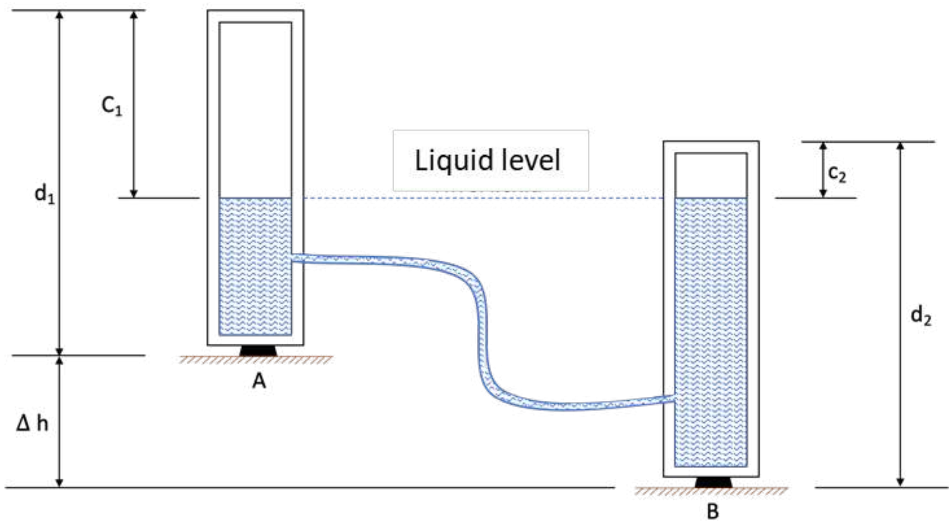

Figure 2.16 Hydrostatic leveling.

b) Industrial use

Since geometric leveling makes it difficult to automate measurement processes and to transmit information, as in many cases automatic levels cannot be used, measurements that are carried out, for instance, in areas with high radiation of nuclear power plants, can only be performed when maintenance works take place. Therefore, the measurement series are set at too long intervals, the selection of conclusive data becoming difficult. In these situations, it is suitable to use hydrostatic leveling with the help of different systems. [1], [3]

Hydrostatic systems are also suitable for verifying machine elements in terms of mutual positions in height, the points where observations are made, send information to a processing unit that can be set to indicate a position or even send an alarm signal, if there are significant changes in position. [1], [3] In the near domain, up to 50 mm, by developing electrical or electro-optical techniques, in order to both automate the work and achieve extremely high accuracy, values of measurement uncertainties of several micrometers have been reached in terms of setting the column fill elevation relative to a reference. The 3 main types of hydrostatic measurements are [1], [3]:

1) Measurements that are suitable for highlighting changes in slow altitudes, the principle of operation being to measure the upward movement of a float, a movement necessary to reach the surface with a touch tip. [3]

The instruments for this type of measurement are very rarely mass-produced, one of which is the Meisser Hydrostatic Level:

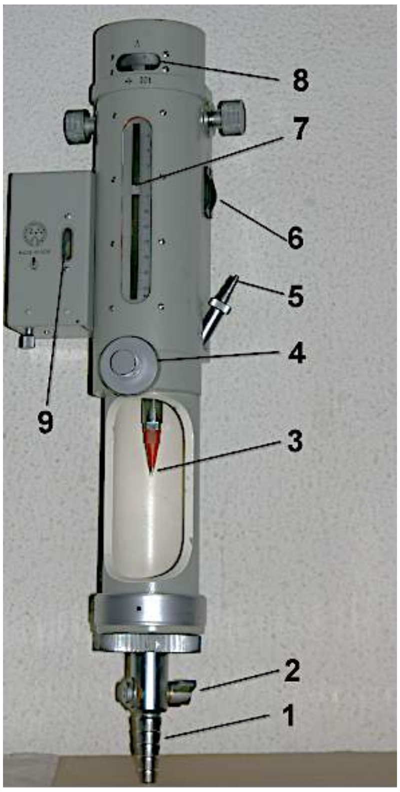

Figure 2.17 Meisser hydrostatic level. [3]

where:

1 - hose connection device;

2 - stopcock;

3 - measuring tip;

4 - screw for driving the micrometric device;

5 - device for connecting the pressure hose;

6 - light bulb switch;

7 - millimetric divided scale;

8 - drum;

9 - bulb indicating that the measuring tip is reaching the water.

By manually operating a micrometric screw, a measuring tip reaches the liquid level, the contact between them being signalized by turning off a light bulb. The length scale is divided millimetric, the tip stroke being a division. The readings are made on the drum of the micrometer, being possible readings of 10^{-2} mm and approximations of 10^{-3} mm, and for the application of the corrections for the temperature differences in the cylinders, there are interconnected thermometers.

Due to the unequal distances between the zero point of the scale and the hanging point of the instrument, the so-called zero-point error occurs, so that the correction constant K needs to be applied to the simply observed elevation differences. However, this zero-point error can be eliminated by permuting the measuring systems. [3]

$$K = K_A - K_B$$

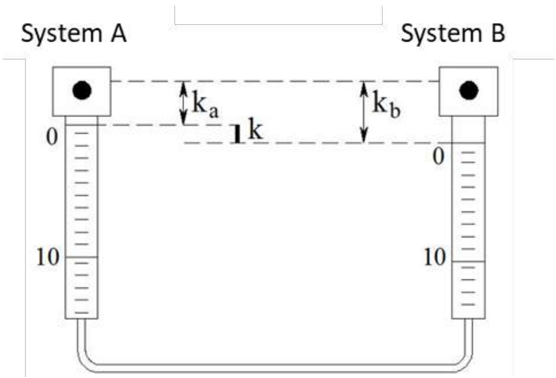

Figure 2.18 Determining the *K* constant.

2) The procedure for measuring distances without contact between the mark and the surface makes it possible to realize extremely accurate observations (a few micrometers) for very small measuring strokes, or to adjust some vibrating surfaces. [3]

An example of a device developed based on this principle is the HLS (High-Precision Leveling System) of the ESRF (European Synchrotron Radiation Facility), displaying capacitive measurement of the distance between the liquid mirror and the sensor positioned above it. The vessels of the system are manufactured from a good conductive material, stainless steel, so that their temperature is identical to that of the liquid columns, whose level is about 30 mm. Due to the horizontal mounting of the connecting hoses between the fittings, the changes in the height of the liquid columns caused by temperature differences are observed at 1 μm. In addition to the very high accuracy of determination, another advantage of the system is the rapid verification of all measuring points.

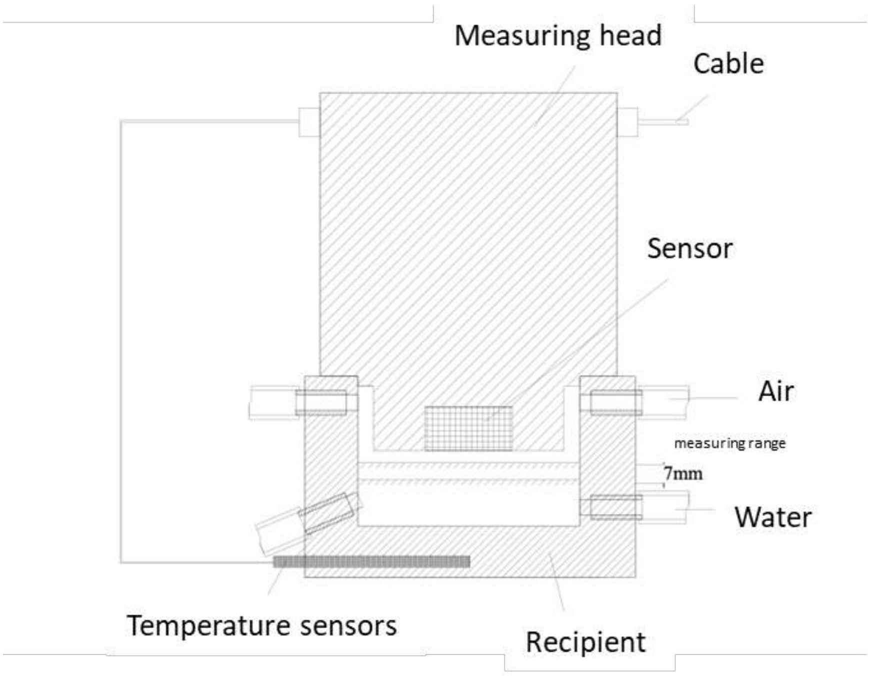

Figure 2.19 HLS horizontal tube principle of ESRF. [1], [3]

3) The process of measuring the pressure of an ascending body or the bottom pressure of the liquid column ensures to achieve some accuracies only at the submillimeter level, however, the instruments developed on this principle are particularly robust and offer measurement opportunities in a wider field.

Ordinary liquid vessels are replaced by pressure vessels, and the whole system is completely filled and sealed, the level differences being the result of measuring the pressure by means of pressure transducers. The determination of the zero-point correction of the vessels is noted as a special operation with regard to mobile versus static systems. [1], [3] [1], [3]

3. MODERN METHODS OF MONITORING CONSTRUCTIONS AND MACHINERIES

Based on the classical principles of construction and machineries monitoring techniques and technologies, new methods and tools have been developed to verify the positions of different structures.

Definitely, as a support of geodetic equipment for building/machineries behavior tracking in time, the so-called non-geodetic measuring and control devices are employed, which can be used independently in monitoring operations, however, their best performance being shown in cases where they are simultaneously operated with the geodetic instrumentation, completing it in this way. [7]

3.1. Photogrammetric methods

During a photogrammetric operation, all data will be recorded in a series of three-dimensional coordinates, in graphical form.

This technique was originally developed to help create plans or maps using aerial or terrestrial images, taken and then processed, but over time its applicability has extended to the creation of three-dimensional models, thus having a wide spectrum in terms of fields of activity. [7], [12]

With the passing of the years and with the technological development, the photogrammetric systems have undergone changes, becoming more and more accurate and efficient, but also much more accessible, the latter aspect resulting from the transition from cameras fixed on large or small aircraft to high-performance cameras found on remote-controlled air vehicles (UAV-Unmanned Aerial Vehicle). [7]

There are several types of drone classifications (UAVs), the most important depend on the type of engine: electric or with combustion (piston, turbine) and on the design features: fixed wing (horizontal takeoff) or helicopter type with a propeller or multirotor (vertical takeoff).

High-precision sensors weigh at least 300g, so models chosen for monitoring works should be able to carry heavier weights. [28]

3.1.1. DJI Phantom 4 RTK

A very common model used in photogrammetric measurements with the help of drones, is produced by DJI and is called Phantom 4 RTK. This electric multi-rotor UAV has an integrated RTK (Real Time Kinematic) module, thus providing real-time positioning data at centimeter-level accuracy. [22]

In terms of in-flight measurement safety (data collection), this model has the ability to store satellite observations so that they can be used in PPK (Post Process Kinematic) within the DJI Cloud PPK Service. [22]

Another quality in terms of accuracy is the possibility of this positioning system to be connected via WiFI or 4G to a D-RTK 2 station, a high-precision mobile GNSS station, functionality that provides to the aircraft real-time differentials data, being practically used as an RTK rover. [22]

The controller, camera and RTK module are synchronized via TimeSync, which ensures that each photo uses the most accurate metadata and fixes positioning information in the optical center of the objective, resulting in optimized results. [22]

In this model, the controller can be of two types, namely: with integrated screen or with the possibility of connecting a device with IOS or Android system.

The camera in this system has a 1-inch CMOS sensor and 20 megapixels. A CMOS sensor slightly differs from a CCD (Charged Coupled Device) sensor in terms of operating principle, although they are inspired by them and is an electronic chip that converts photons (light) into electrons (electrical signals) for digital processing. CMOS (complementary metal oxide semiconductor) sensors are used to create images in digital cameras, digital video cameras, and digital CCTV cameras. The notable difference between CMOS and CCD is when photocells are processed, as on CDD sensors they are passive, and processing is performed after storage, while on CMOS sensors, they are active and can perform processing on the spot before storage. [22]

Figure 3.1 DJI Phantom 4 RTK drone (top right), its built-in screen controller (bottom right), D-RTK 2 high-precision mobile GNSS station (left). [22]

Two of the applications in the field of behavior tracking in time for which the photogrammetric technique is best suited are the monitoring of tailings dumps, or sinkholes.

3.1.2. Waste dump monitoring [12]

A geodetic network for monitoring, and a topographic plan is firstly realized, based on which both the storage method and different angles will be monitored.

The volume of stored material is determined using the section procedure or the horizontal network.

The whole operation consists in measuring the influence of the tailings dump on the land around it, and the deformed slopes. For this purpose, near the object, but also outside the area of influence, reference points are materialized which are determined by rigorous operations of topographic or photogrammetric nature. [12]

The situation with regard to the movements of the dump or the surrounding terrain may be determined according to the slope angles and the differences in the coordinates of the points, thus being possible to calculate the displacement vectors. [12]

3.1.3 Sinking riverbeds

Determining rock displacements as an effect of mining is the main objective of this type of monitoring.

Sinking riverbeds are particularly active in areas of underground mining, that operate on the subfloor method with collapsing roofs or directional chambers with collapsing roofs, the phenomenon being called collapse. [12]

In this study of rock displacements, the monitored characteristic elements are, its diving and velocity, displacement vectors, the ratio between the growth volume of the diving bed and the volume exploited underground, the submersion (compaction) coefficient, the breaking angles and the limit angles.

The use of photogrammetric methods for tracking the behavior in time of this type of objects is recommended because of their pattern, with non-uniform and large slopes, with deep and wide cracks and vertical or steep walls. In this sense, the monitoring network is positioned in profiles that go far beyond the boundaries of the riverbed, its points being topographically determined and marked with landmarks. [12]

3.2. GNSS determinations

Due to its particularly important advantages compared to classical methods, GNSS (Global Navigation Satellite System) technology has obviously prevailed in the field of monitoring both land and construction.

The unbeatable efficiency is given by the extraordinary efficiency shown in the realization of the geodetic networks, in this case, of monitoring, with enormous extensions, of up to several thousand kilometers. The points within these networks are determined with constant precision, without the visibility between them playing any role in achieving it. [15]

Tracking the behavior in time of buildings and lands, performed with the help of GNSS technology is divided into 3 broad categories, depending on the nature of the displacements and / or deformations to be measured, as follows:

3.2.1. Use of GNSS to determine horizontal displacements and deformations

There are two ways in which planimetric displacements can be determined using GNSS technology, namely:

- by means of trilateration method
- using the relative coordinates of the vectors, only planimetric notions are taken into account in the analysis [15]

In the case of the first option, definitely, the distances will be measured using GNSS technology. The use of the method involves a number of additional measurements concerning the tracking network geometry in order to achieve the required accuracy to determine the planimetric position of the points.

Additional guidance for setting up the tracking network are: at least three vectors converge towards each point of the network and the accuracy with which the level differences and the inclined distances are determined by means of GNSS is high. [15]

The coordinates obtained by transcale will be used as provisional coordinates to reduce the distances to the projection plane, and the tilted distances will be reduced to horizontal, following that the correction equations to be written using the horizontal distances, the most likely coordinates being obtained upon solving the normal system, the displacements resulting from the differences between at least two measurements epochs.

If, instead, you do opt for the second option - to determine the plane displacements - it will be found that it is more appropriate compared to the trilateration, since the relative coordinates of the vectors are determined with a millimeter level accuracy. [15]

3.2.2. Use of GNSS to determine vertical displacements and deformations

Similar to the situation described above, two methods can be used to define the level displacements and / or deformations by means of GNSS technology:

- use of ellipsoidal distances and elevation differences, resulting from vector compensation ratios
- the use of the relative coordinates of the vectors, only the vertical elements enter the analysis

There are situations when GNSS technology also appears to track vertical displacements and deformations, although, in the field of monitoring, it is more common in works dedicated to planimetric displacements. [15]

Since the accuracy of the vector determination is of the order of centimeters, or even millimeters, the principle of relative positioning provides good results for determining the points of the network, its mode of operation being the determination of the position of a new point with respect to a known point, subsequently the vector between the two points - called base - being determined. [15]

At the same time, however, if the second procedure is considered, its principle is based on the compensation reports generated by different information processing software taken with GNSS technologies, for each vector in the network. A compensation, similar to that of the geometric leveling method on ellipsoidal notions (distances, level differences) extracted from the reports, will be specified. [15]

In terms of accuracy, the use of GNSS technology in this type of work is on the same level as the trigonometric leveling method, being clearly inferior than the geometric leveling.

3.2.3. Use of GNSS technology to determine spatial displacement

In this case, two ways of compensating the measurements are used, depending on their reporting, either in a three-dimensional geocentric cartesian reference system (X, Y, Z), or on a reference ellipsoid (B, L, h). The latter example is quite rare due to cumbersome implementation using relative values, which are rather converted into equivalents such as ds (ellipsoid distance), dA (geodesic azimuth) or dh (level differences on ellipsoid). The easiest to realise, and at the same time, the most common choice for compensation is the one in which the measurements were made in a three-dimensional cartesian reference system, for the simplicity of the correction equations, the measured elements being the vectors formed between the network points (relative values ΔX_{ij}, ΔY_{ij}, ΔZ_{ij}). [15]

3.3. Reflectorless method

Due to the development of reflectorless technology, i.e., measurements without reflector, the manufacturers are now able to provide total stations that reach accuracy in determining the distance in the reflectorless mode of 2mm + 2ppm. Taking into account the high accuracy in determining distances and the high angular accuracies of 0.5", 1", 2", as well as the fact that certain models of total stations allow automatic or semi-automatic collection of measurements, tracking the buildings behavior in time using measurements without reflectors becomes possible. [15]

Currently available technologies allow automatic or semi-automatic monitoring, using total motorized stations for instance. Some of the disadvantages of traditional reflectors tracking methods are: the acquisition costs; the time required to perform the measurements, if they are performed semi-automatically; the difficulty of positioning or the impossibility of positioning the required reflectors on construction for various reasons; and last but not least, due to the long time of the works, the reflectors lose their reflectivity due to the condensation that is created or the dust cover. [15]

3.3.1. The principle of the method

It starts from the fact that the object points are not materialized on the object, thus the displacement can only be determined in a certain direction.

The object points are measured in the basic measurement epoch, their coordinates being determined with the help of observations: tilted distances L_{ij}, azimuthal directions / orientations θ_{ij} and zenithal directions ζ_{ij}. [15]

The coordinates of the tracking points "j" are calculated according to the coordinates of the station points "i" and the measured elements:

$$X_j = X_i + D_{ij} cos\theta_{ij}$$

$$Y_j = Y_i + D_{ij} sin\theta_{ij}$$

$$Z_j = Z_i + L_{ij} cos\zeta_{ij}$$

where $D_{ij} = L_{ij} sin\zeta_{ij}$

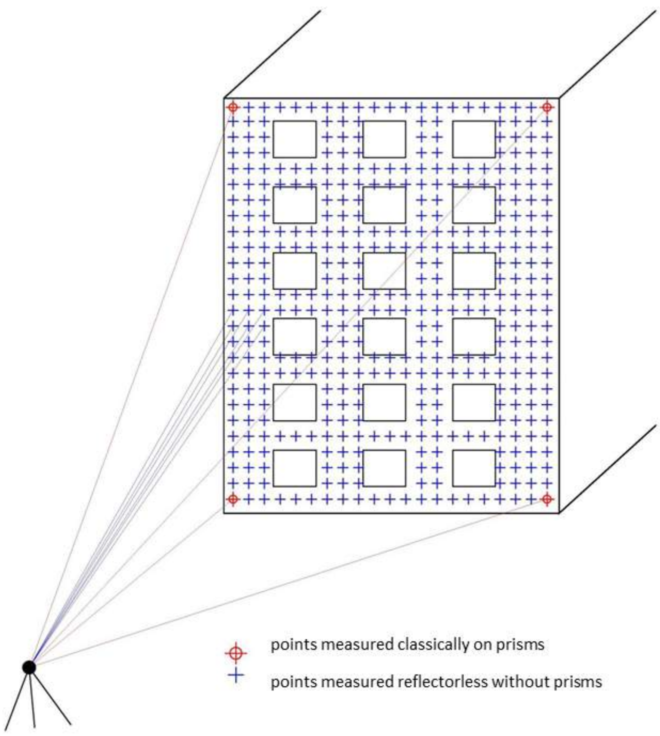

Figure 3.2 The principle of classical methods of tracking with the help of reflectors and the principle of monitoring without the aid of reflectors. [*15*]

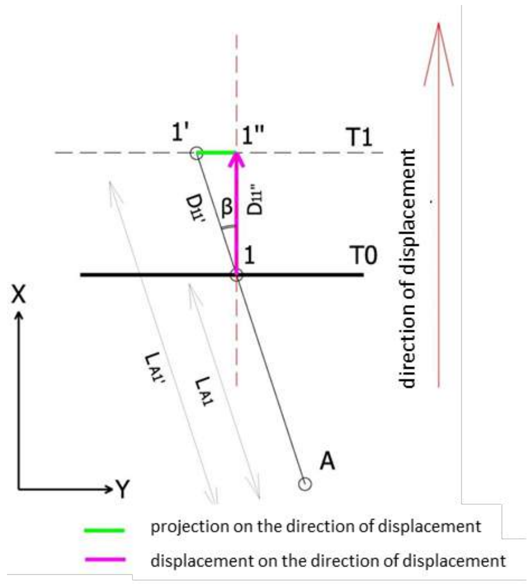

Figure 3.3 The principle of the Reflectorless method. [15]

If in the initial measurement epoch, the elements mentioned above are measured, in the next cycle the same types of observations will be performed to the same points determined in the basic measurement era, following that the notions necessary for the angular positioning to be calculated by the instrument (total servo-motorized station) starting from the known coordinates of the station point:

$$\theta_{A1} = arctg\frac{X_1 - X_A}{Y_1 - Y_A}$$

$$\zeta_{A1} = arccos\frac{H_1 - H_A}{L_{A1}}$$

where $L_{A1} = \sqrt{(X_1 - X_A)^2 + (Y_1 - Y_A)^2 + (H_1 - H_A)^2}$

ζ_{A1} – reduced zenith angle at point elevations

Since the observations are made from the height h of the instrument, we will have the final zenith angle corrected as follows: $\zeta_{A1}^{*} = arctg\frac{(H_1 - H_A) - h}{L_{ij}}$, because the height of the s signal is 0 in this monitoring technique. [15]

The horizontal and vertical orientation angles are automatically calculated based on the orientation θ_{A1} and the corrected zenith angle ζ_{A1}^{*}, to direct the instrument to the point of interest on the object measured in the initial epoch.

The tilted distance to be used in calculating the coordinates of the tracking point, in this example, point 1', will be measured after securing the position of the instrument in the theoretical direction of point 1. [15]

$$X_{1'} = X_A + L_{A1'} sin\zeta_{A1}^{*} cos\theta_{A1}$$

$$Y_{1'} = Y_A + L_{A1'} sin\zeta_{A1}^{*} sin\theta_{A1}$$

$$Z_{1'} = Z_A + L_{A1'} cos\zeta_{A1}^{*} + h$$

The horizontal displacement $D_{11''}$ will be calculated according to several factors, thus it will be necessary to know the characteristic direction of displacement, and the variation of the distance $D_{11'}$ which is calculated along the sight axis, the latter being represented by the positions of point 1 and the station point from the basic epoch. [15]

$$D_{11'} = \sqrt{(X_{1'} - X_1)^2 + (Y_{1'} - Y_1)^2} = D_{1'} - D_1$$

3.3.2. Usage conditions

The positioning of the total station at an angular level in the direction of the points tracking in the base epoch and the comparison of the distances measured towards the object are the principles underlying the method, thus resulting several statements that need to be followed in order to use the method under normal conditions. It is imperative to use high accuracy instruments (total stations), if possible, robotic ones, then to carefully select the points tracking in the initial measurement epoch, to reduce the effect of reflectivity, to apply atmospheric corrections to distance measurements and to reduce the systematic errors. [15]

3.4. Radar interferometry

3.4.1. IDS GeoRadar IBIS-FS

In this sense, it is to be noted that the terrestrial radar interferometry systems can determine extremely precisely the oscillations or deformations of the researched objectives. The systems developed by IDS Georadar, called IBIS (Image by Interferometric Survey), are available in different configurations and can statically or dynamically track different types of buildings or areas. [14]

Figure 3.4 IDS GeoRadar IBIS-FS Plus system. [23]

The instrument is a microwave interferometer and consists of:

- the sensor module
- computer
- power supply unit

The sensor module is a radar that stores the signal phase and generates, transmits and receives the electromagnetic signals to be processed to provide the measurements of deformation and vibration. [14], [23]

It is installed on a tripod with a rotating head, capable of supporting its weight of about 12 kg. In order to measure distortions, the unit transmits a continuous wave with a gradual change in frequency, which is reflected on natural or artificial targets. However, in order to identify a target, the signal strength must be greater than the surrounding noise.

An important aspect for a good signal reflection is the corner reflectors (cube corner). These can already exist on the analyzed construction as parts of it, without the need to install artificial ones. Corner reflectors are mainly used if the measured object is made of a material that does not reflect microwaves, these help to identify characteristic points of construction. [14], 23]

Figure 3.5 Corner type reflector. [24]

The device detects the phase difference of the signal reflected by the specific elements of the structure, from one moment to another, which is given by the relation: $d = \varphi_2 - \varphi_1$, and the measurement of the displacement and the measurement of the phase variation are connected by the relation: $d = -\lambda/4\pi \cdot \Delta\varphi$ where "d" represents the displacement, "λ" is the wavelength, and Δφ is the phase difference. [14]

In particular, it should be noted that the IBIS system, simultaneously measures the movement of all pixels or fields in the microwave illuminated area, it generates the measurement signal, using the transmitter-receiver antennas, and then receives the reflected signal. [23]

In the first stage, the radar detects the elements of the target, following to determine their distance from it.

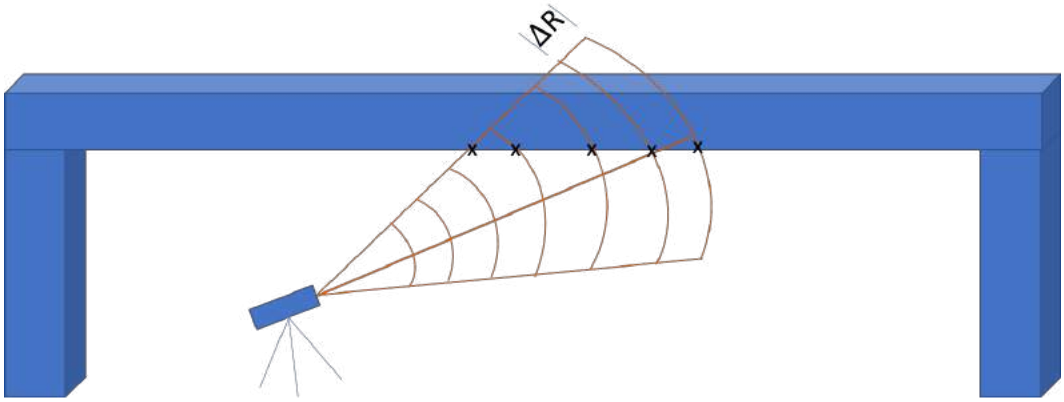

Figure 3.6 IBIS system measurement.

As can be seen in the figure below, the system measures the radial displacement (dp) from which the actual displacement (d) of the point can be obtained by taking into account the positioning geometry of the instrument with respect to the object under investigation:

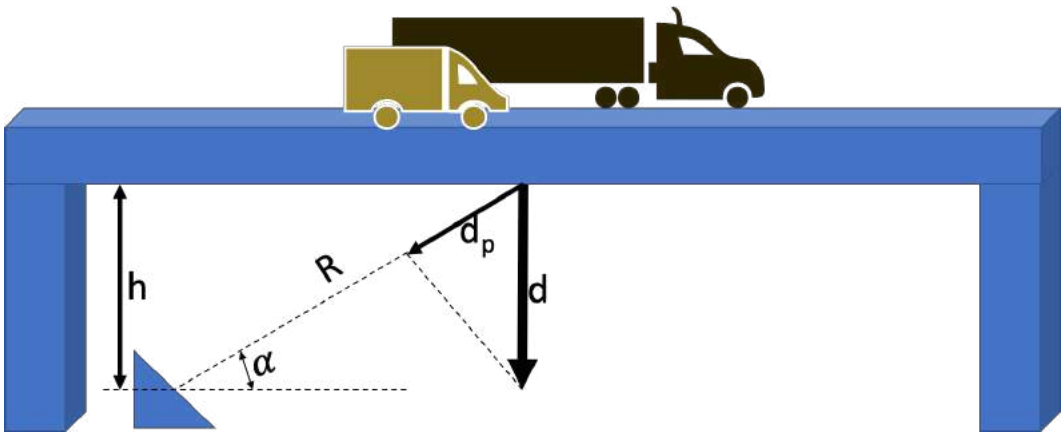

Figure 3.7 Displacement d measurement.

The resolution is the "ΔR" dimension, which is the smallest distance within a single point of observation can be distinguished. Depending on the frequency used, the maximum resolution is between 0.5-0.75 m. [14]

$$d = \frac{d_p}{\sin\alpha} \implies \sin\alpha = \frac{h}{R} \implies d = d_p \cdot \frac{R}{h}$$

The accuracy of the displacement value measured by this interferometric technique (the phases of two distinct images are compared) depends on the intensity of the recorded radar echo and the signal-to-noise ratio (SNR) of the tracked point. A high SNR (signal-to-noise ratio) is a basis in the selection of tracking points on the construction. [14]

In practice, an accuracy of 0.1 mm can be achieved, although the theoretical accuracy of the instrument it is around 0.01 mm. For static and dynamic monitoring of different constructions, the system can be used only 1D, with a range of 1 km and can monitor several points at once at frequencies up to 200 Hz.

Upon mounting on the tripod and orienting it to the construction under investigation, the horizontal direction and the inclination of the receiving transmission unit are adjusted. Subsequently, the field computer is connected, after which the geometry of the instrument is set in relation to the objective. [14] Regarding the use of software to purchase data, it has different windows, namely:

- Acquisition mode selection window: static (data retrieval time range can be set between 0.1 and 10000000.0 sec) and dynamic (with sampling frequency between 10Hz and 200Hz);
- Instrumental parameters setting window: allows the setting of acquisition parameters (maximum distance, resolution, antenna type, sampling frequency and measurement duration);
- Geometry setting window: allows the setting of the acquisition geometry (position of the instrument in relation to the investigated object and the tilt angle of the instrument);
- Mission selection window: allows you to stabilize the names of data action missions;
- Time management window: allows the recording to be started and stopped and allows the user to view the recorded data in real time;
- Measurement processing window: allows the analysis of data acquired with different types of processing, depending on the selected acquisition type. This window remains active even when the instrument is not connected to the PC.
- The Projection Options button allows the user to switch from viewing radial displacement (None) to orthogonal, longitudinal, vertical, or horizontal displacement (calculated using the parameters set in the geometry configuration window).

[14], [23]

3.5. Precision laser alignments

Within this procedure, a laser beam is used as optical reference direction. All deviations from this alignment will be optically or electro-optically investigated. [1], [3]

The elements that increase the measurement uncertainty are the displacement of the rays coming out from the emitter, the changes in the direction of the laser rays caused by temperature or the positioning of the emitter components, the variation of spot intensity, and the spot deformation caused by atmospheric refraction. These aspects that can influence the accuracy can be improved by simultaneously observing a reference line when installing the reference sensor, according to the measurement principle. [1], [3]

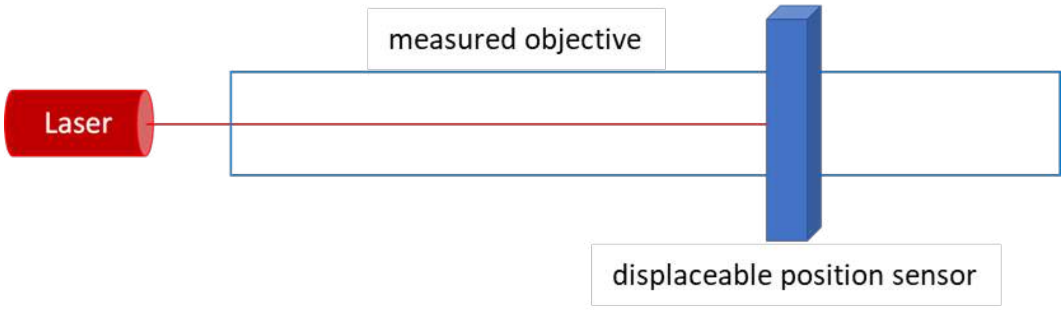

Figure 3.8 Linearity measurement with laser beams and position sensors.

For systems developed based on the principle described in Figure 3.9, to which an optical system is used to design the radius on the sensor, the longitudinal axis of the sensor system, when measuring, must be parallel, in order not to notice alterations of non-existent distances. [3]

The possible range of accuracy that can be obtained, depending on the type of technology based on these principles at distances of up to 30 m, is 0.01 mm - 0.1 mm.

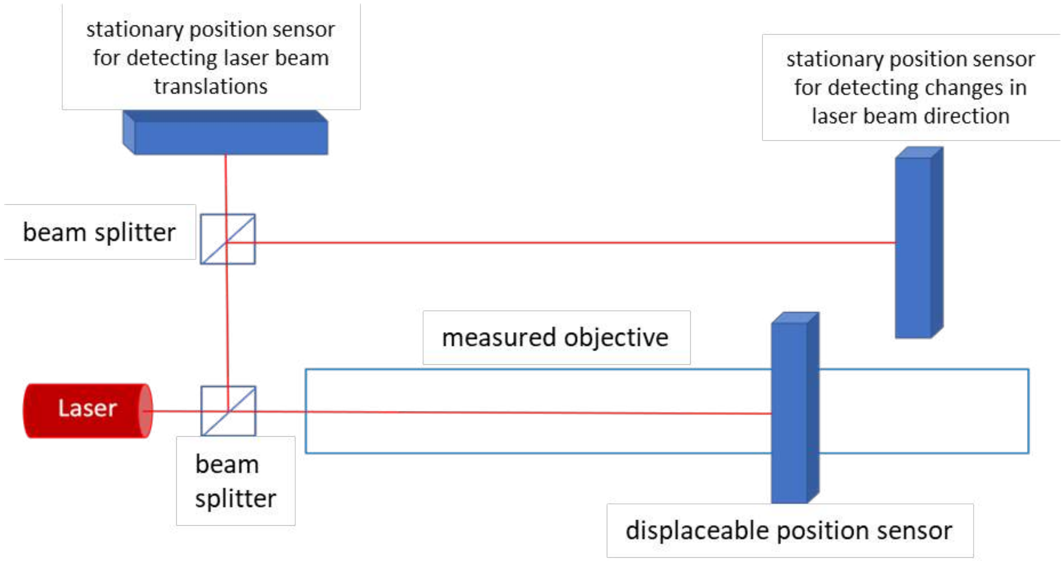

Figure 3.9 Observation of systematic influences.

3.6. Autocollimation method

It was specifically developed for use in the industrial field for the control of directions and the research of angle changes, the basic technology being the autocollimation telescope, and improved geodetic equipment with accessories such as flat mirror with autocollimation oculars or autocollimation reflectors. [3]

3.6.1. The autocollimation telescope

The autocollimation image is achieved by the projecting an image of the aiming bars in the direction of the parallel rays by telescope, on a mirror that will reflect the light beam back into the collimator.

The aiming bars can be seen through the ocular in the focus of the objective if the mirror is perpendicular to the axis of the beam, so that it is reflected on it. The image of the projected aiming bars will be observed as being displaced in the focal plane ($s = f \cdot 2tg\varphi$) if the mirror does not respect the perpendicularity described above, appearing a tilt φ, thus resulting in a deviation of the rays equal to 2φ. [3]

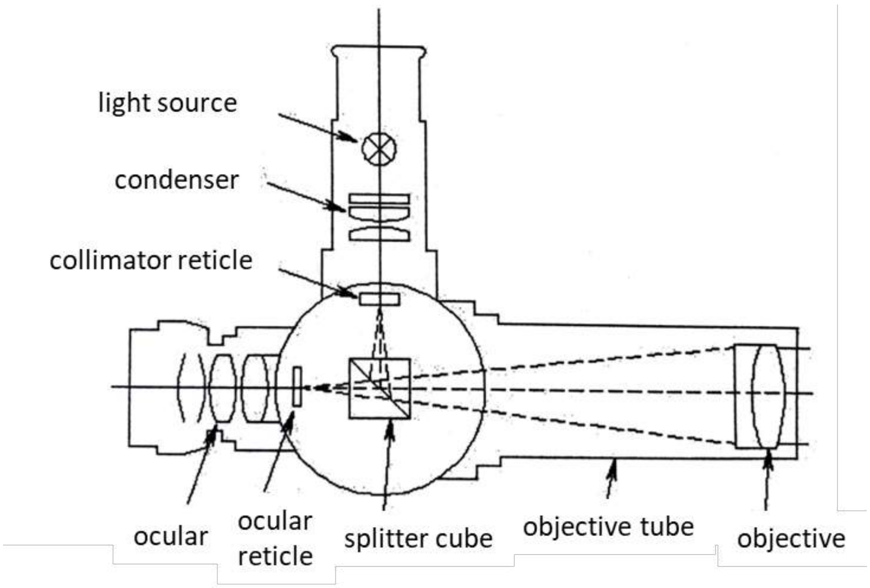

Figure 3.10 The autocollimation telescope. [3]

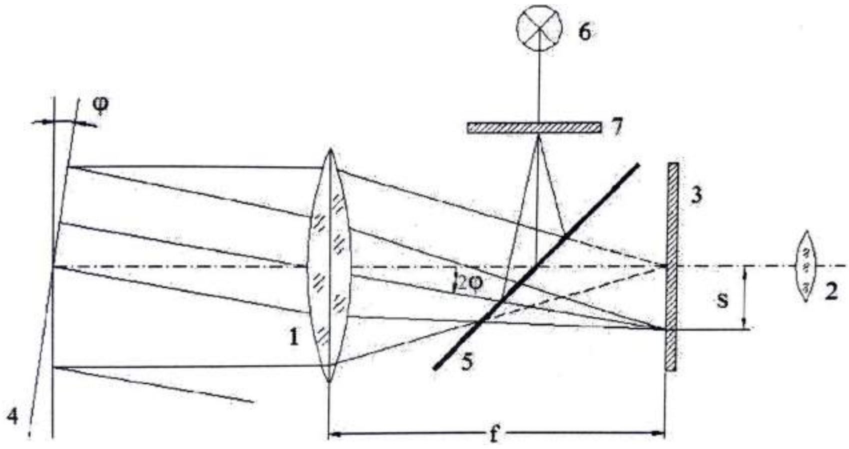

Figure 3.11 The path of the rays in the autocollimation telescope. [3]

where: 1 – objective, 2 – ocular, 3 – the ocular aiming bars, 4 – movable mirror, 5 – semi-permeable mirror, 6 – light source, 7 – reticular threads

It is possible to calculate the rotation of the mirror and to investigate the displacement of the image *s* only if the telescope has a micrometer ocular, or if the reticle has a horizontal / vertical scale:

$$\varphi = \frac{s}{2f}$$

where φ – mirror rotation angle (radians)

s – the displacement of the projected reticle image relative to the telescope reticle

f – focal length of the objective [3]

Nowadays, there are modern electronic autocollimators that use a Charged Coupled Device (CCD) sensor to determine the position of the illuminated aiming bars image. [3]

Depending on the measurement accuracy of the image displacement and the focal length of the telescope, the measurement accuracy will be determined using this type of technology.

3.6.2. Autocollimating ocular

It is used to project the image on the autocollimating telescope in place of telescope aiming bars, being directly fixed on it.

As a working principle, the image is directed by means of a beam divider, in the direction of the telescope's rays, so that the rays reflected from the mirror will be projected in the plane of the aiming bars, i.e., in the focal plane of the object. [3]

Equipping the levels or theodolites with autocollimating oculars is considered a special advantage for measurement works performed on different industries, the measuring range being strongly extended compared to that of the telescope due to the horizontal and / or vertical circles. [3]

3.6.3. Rectilinearity checking

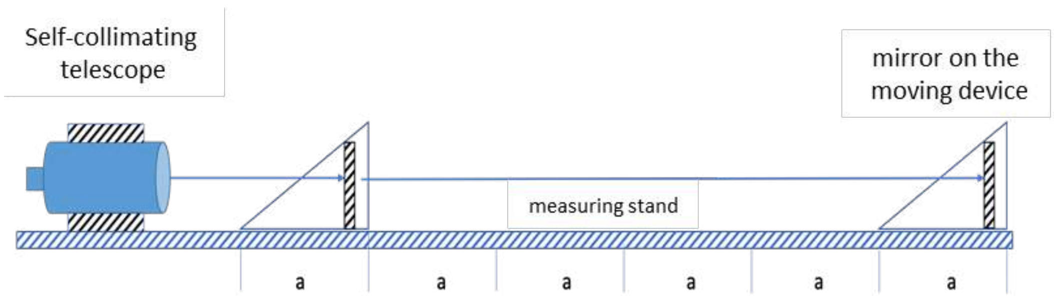

Figure 3.12 a) Checking the linearity with the autocollimating telescope.

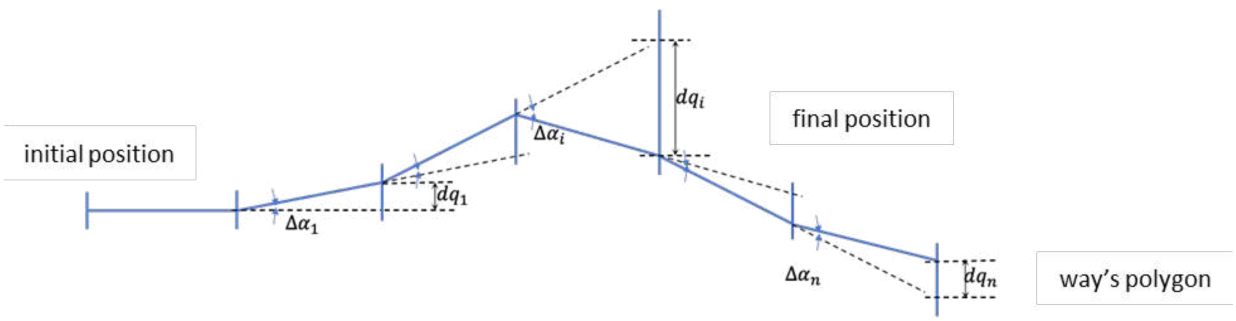

Figure 3.12 b) Checking the linearity with the autocollimating telescope.

The linearity of a measuring stand is controlled by moving a mirror, only along the stand, at predetermined distances. Thus, the mirror needs to be fixed on a device that determines the altimetric positioning with three points, two of which will be positioned in the direction A. The horizontal and vertical rotation angles (α_0) of the normal to the mirror with respect to the telescope axis shall be measured at the initial position of the mirror, while the angles $\alpha_i \dots \alpha_n$ shall be measured along the stand in each predetermined position, including final position. [3]

Changes in directions along the measuring path are determined by means of angular changes $\Delta\alpha_i = (\alpha_i - \alpha_{i-1})$. These changes of direction along the measuring path, by means of the distance a between the support points will be transformed into changes of shape. According to the shown figure, the successive deviations of the device position are given by: $dq = a \cdot \Delta\alpha$. [3]

3.6.4. Study of the perpendicularity of two surfaces

The pentagonal reflector system shown in Figure 3.13 is used in situations where it is necessary to investigate the rectilinear character of two surfaces arranged relative to each other at an angle of 90° and, certainly, the accuracy of that angle.

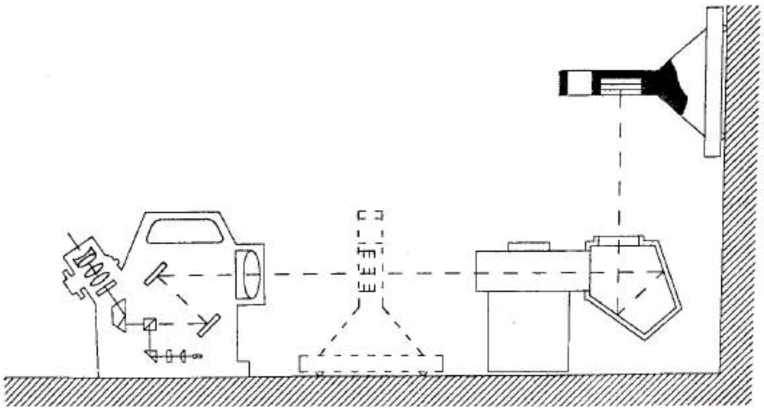

Figure 3.13 Checking the perpendicularity of two surfaces using a pentagonal reflector. [3]

The first action to be taken, is to check the rectilinear nature of the horizontal surface, which is normally performed with a probing mirror, the readings being continued without moving the autocollimator, inclusively on the vertical side, by interposing a pentagonal reflector in the corner formed by the two surfaces, the measurements being performed similar to the case of horizontal surfaces. [3]

3.7. Measurements with invar wires

The measurements carried out using invar wires are suitable for determining the relative changes of the lengths in the range of meters between two points.

Two of the most important and relevant technologies in this field that underpin this principle are:

3.7.1. ISETH dystometer

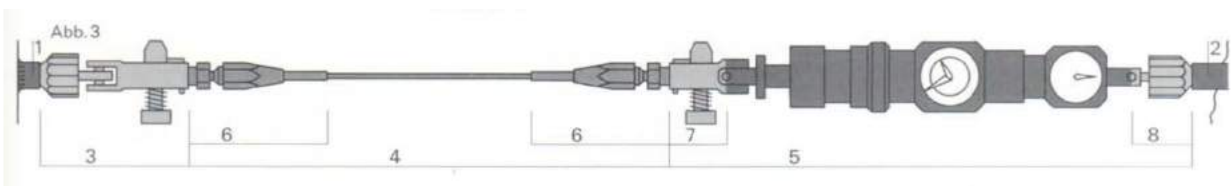

Figure 3.14 System components. [17]

where: 1 – Welded measuring bolt, 2 – Concrete measuring bolt, 3 – Connecting joint with stop screw for coupling the invar wire, 4 – Invar wire, 5 – Dystometer, 6 – Invar wire connection, 7 – Stop screw for connecting the invar wire to the dystometer, 8 – Dystometer connection

This instrument is manufactured by Kern Aarau (Swiss) and the measuring clock of the instrument is suitable for small measuring ranges. The device is also used alone, but it is very common as a component of other systems. [3], [17

Essential technical data:

- Invar wire length: 1 - 50 m
- Maximum measurable length of length change: 100 mm
- Accuracy:
    - Wire ≤ 20 m : 0,02 mm
    - Wire > 20 m : $1 \cdot 10^{-6} \cdot l$, where l – the length of the wire

3.7.2. Distinvar

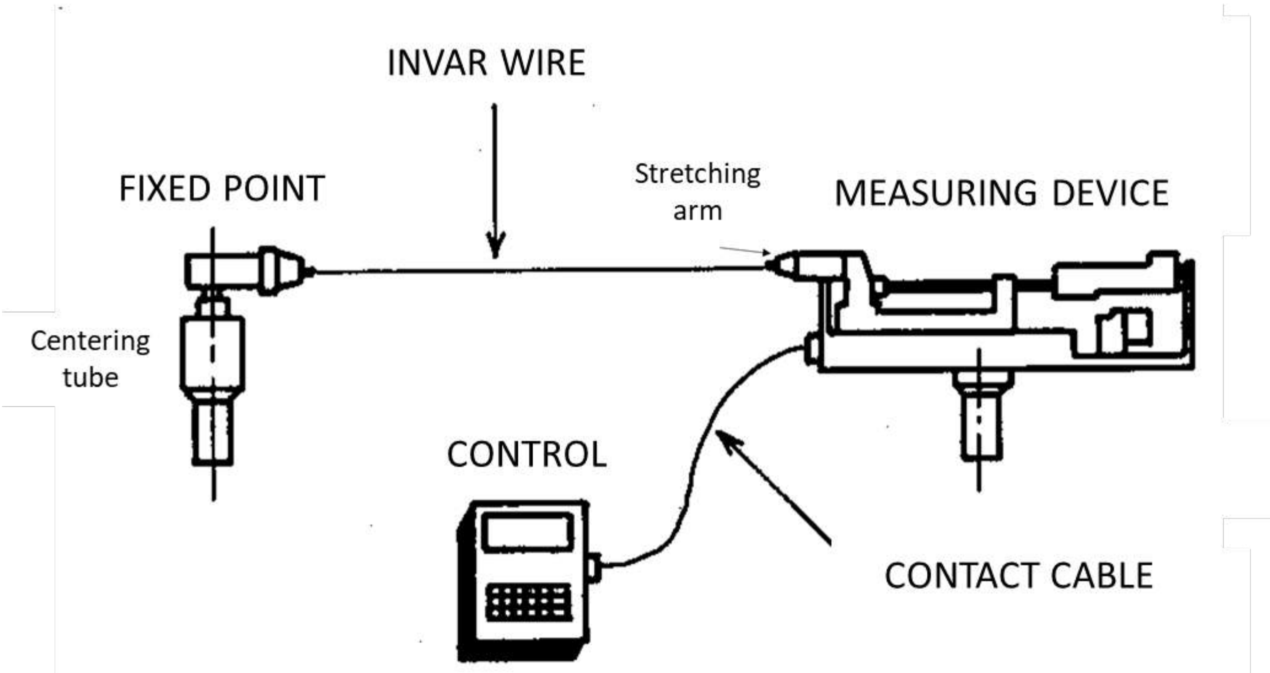

Figure 3.15 Principle of measurement with Distinvar. [18]

Within this technology produced by CERN (Geneva), the device is mounted into a centering tube instead of a bolt, and the tension is produced by an arm. The position of that arm is also defined photo-electrically, while the whole apparatus is coordinated based on its structure within a range of 50 mm by a servo-motor such as to obtain the optimal positioning of the arm. [3], [18]

As a disadvantage compared to the previously presented dystometer, the technology is only suitable for horizontal measurements or very close to the horizontal. Instead, the major difference, and at the same time, the significant advantage compared to the above ISETH technology, consists in the possibility of automating the measurements (works) due to the constructive principle, exhibiting electric drive. [3], [18] The accuracy is superior to the ISETH instrument presented above, being 0.02 mm higher.

3.8. Fiber optic sensors

The use of fiber optic sensors is found in most works of structures behavior tracking in time, that require very high accuracy in execution. There are several types of sensors.

3.8.1. Optical extensometer

It is used in various ways. For instance, it can be mounted on wooden or metal structures, or it can be applied in the field of static or dynamic recordings, or in the recording of cracks. The transformation of the mechanical variations recorded by the instrument into optical signals is performed by means of a sensor, which allows the measurements to be carried out in both dynamic and static mode. [7]

3.8.2. Sensor with YES or NO answer

This type of sensor is used to perform control measurements on cracks up to 4 mm, but is also employed in the case of surveillance of bulges and flexures. [7] Changes in the distances between two points are signaled only when the set limit is exceeded, hence the name (Yes or No).

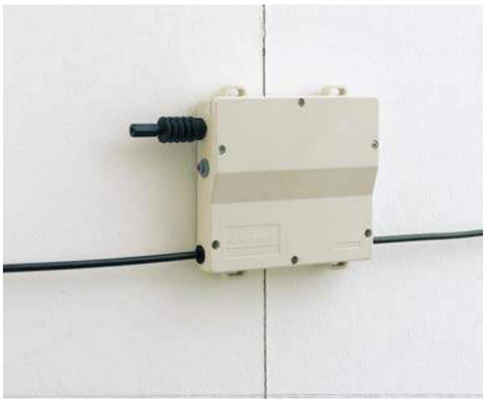

Figure 3.16 Sensor with YES or NO answer.[7]

3.8.3. Structure Monitoring by Fiber Optics (SOFO)

It was developed to monitor concrete / reinforced concrete structures from the first hours after pouring.

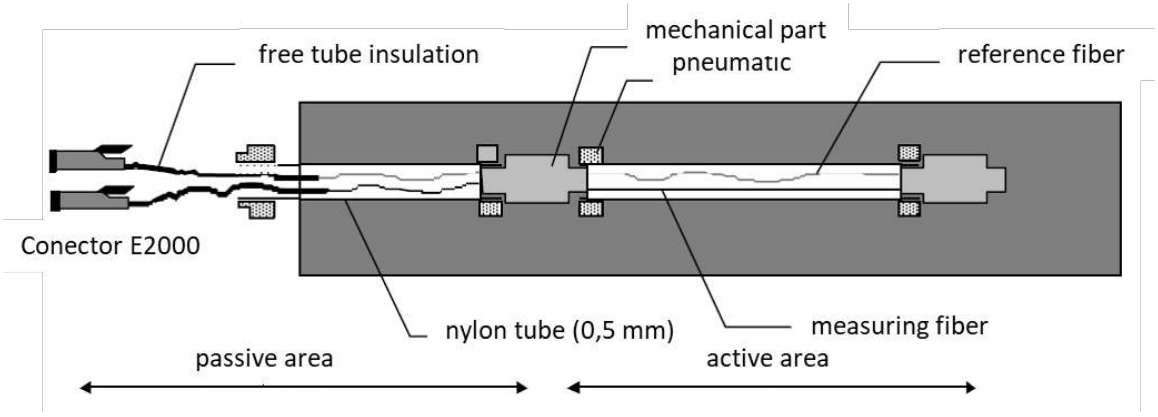

Figure 3.17 Functional diagram of the sensor. [16]

The manufacturing principle of the sensor has been designed in such a way as to allow to be fixed as quickly as possible on the objective, in order to determine the displacements between two points. For this, the sensor has a pair of single-mode fibers, one of which is pre-tensioned between the anchor points to define elongation (shortening), while the second one is free. They are mounted on the objective, one in direct contact with the structure (measuring fiber) and the second one mounted in parallel (reference fiber). A nylon tube provides protection, exhibiting silver facets at one end and connectors for the connecting unit at the other end. [16]

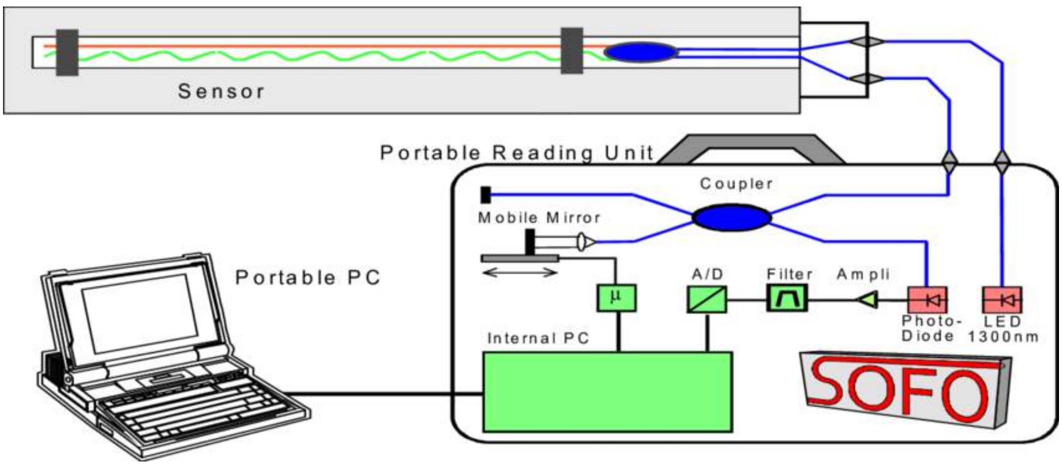

Figure 3.18 Principle of operation of the System. [16]

By using a Michelson dual interferometer, with the first interferometer mounted on the objecti and the second one fixed into the portable reading unit, it will be possible to calculate the deformations and / or displacements of the construction by the difference between the lengths of the two optical fibers, thus making it possible to read several pairs of fibers mounted on different constructions using a single reading unit. [16]

3.9. Clinometers and electronic levels

By means of electronic levels or clinometers, parameters such as flatness, rectilinearity, angles, or level differences can be determined with different accuracies depending on the field of use (either in civil and maritime industry or in production engineering) or the system used.

3.9.1. Clinometers

The manufacturing principles differ depending on the field of use, or, in the case of machineries parts, even on the dimensions of the parts for which the angle measurements are determined.

If we are talking about angle measurements of small parts, we will use, for example, a high-precision micro-optical clinometer such as the TB100, which has a glass circle with divisions, fixed on a shaft connected to a precision bearing. For horizontal reference, a toric level was incorporated at the end of the shaft. The accuracy for direct readings is 10^{cc}, while for micrometer readings it can reach 2^{cc}. [7]

Figure 3.19 TB100 micrometer.

Figure 3.20 Digital clinometer. [19]

The technology has evolved over time, with the advent of digital clinometers, which are useful for measuring angles at large surfaces with high accuracy. They are designed to provide relative or absolute values with built-in calibration software that makes it possible to read angles of -45^g $și$ $+45^g$ directly up to 4^{cc}.

A clinometer instrument used in large areas of measurement, in constructions such as dams or dikes, is the one that is inserted into tubes mounted on / or incorporated inside the objective.

The principle of measurement consists in launching the clinometer into the tube, following that during the extraction, readings will be performed at previously established positions, the values of the angular quantities precisely determined being transformed into linear quantities. [7]

Figure 3.21 Digitilt clinometer.

3.9.2. Electronic levels

A representative and current model of electronic level is produced by Taylor Hobson company and is called Talyvel 6. It is used to study the flatness (e.g., measuring tables), to check the rectilinearity (e.g., the paths of industrial robots) and to monitor the absolute elevations of some supports (detection of displacements from a reference level) [20], [21]

Figure 3.22 Talyvel electronic level (Taylor Hobson). [21]

3.10. Tilt sensors

There are optical, mechanical or electronic tilt sensors, used for tracking the behavior in time of tall structures and last but not least to check the measurement pilasters. Among many companies that produce such sensors, one of the most widespread models belongs to Leica, being simply called Nivel220. [26] The model is part of the newest category of sensors, which combines two modes of construction, being developed under the optical-electronic principle.

In general, as well as this model, this type of sensor achieves extremely high accuracy, with a resolution of up to 0.001 mrad for measuring the tilt direction, two-way tilting or temperature. In addition, a major advantage is the way the data is transmitted, in real time, to a device that uses GNSS technology or to a software for processing the monitoring observations. [7], [26]

Figure 3.23 Leica Nivel220. [26]

Figure 3.24 Elcometer 137 illuminated Magnifier. [25]

3.11. Cracks measurement

The technological equipment that can be used to monitor cracks or fissures are:

- Magnifiers for measuring cracks
- Mobile ruler for measuring cracks: this device represents the economical option for the microscope to determine the width of cracks in concrete or other construction materials. This transparent credit card-sized device is marked with a graduated scale. Each line represents a certain width of a crack. [7]
- Fixed rulers mounted with adhesives on the affected area

Figure 3.25 Elcometer143 mobile ruler. [7], 27]

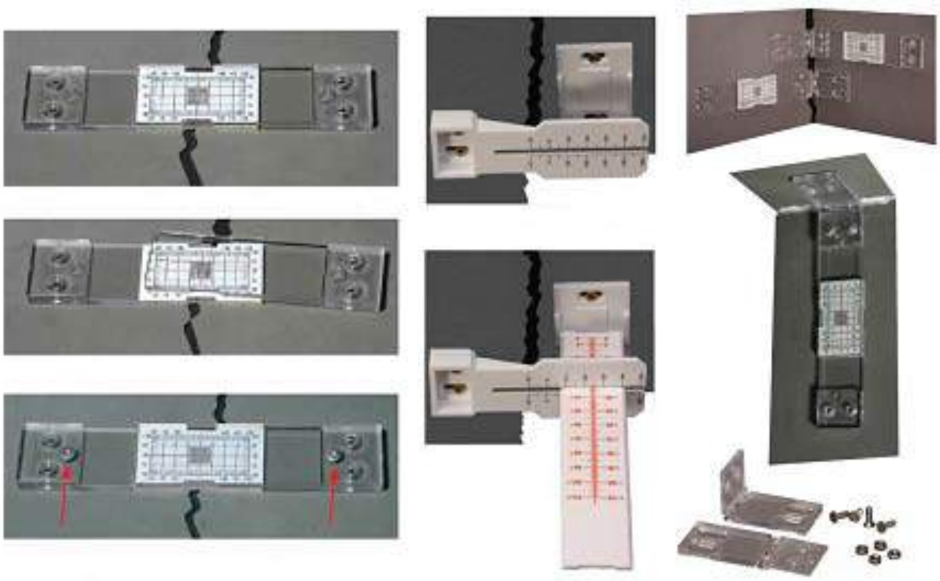

Figure 3.26 Fixed rulers. [7]

4. CONCLUSIONS

The aim of this research report was to investigate aspects related to the evolution of techniques used for monitoring the behavior of the classic construction and machineries, the latter being defined in the introductory part of the paper.

Therefore, in the first part, after framing the topic in the legislative field, and defining the basic notions, the classic techniques for tracking the behavior of construction and / or machineries were approached.

As can be seen in the second chapter, the classical methods of measuring displacement and / or deformation are classified according to the nature of the position changes pursued, thus it is found that horizontal displacements or deformations are investigated using generally three techniques.

Firstly, the trigonometric method was discussed, followed by the technique of measurements in alignment, where notions regarding the close domain were presented, the subchapter ending with the polygonometry method.

The notification of the position changes in the vertical plane, highlighted the use of different techniques, herein being investigated the leveling, trigonometric, geometric and hydrostatic methods.

In the third chapter, a study on the evolution of monitoring technologies and techniques was proposed, highlighting the photogrammetric method, GNSS determinations, Reflectorless method, RADAR interferometry, precision LASER alignments, autocollimation method, invar wire measurements, fiber optic sensors, clinometers, electronic levels, tilt sensors and simple crack measuring equipment.

Carrying out both a punctual retrospective examination and a comparative study on the analyzed methods, the following can be stated:

- The photogrammetric method - although with the evolution of technology has provided satisfactory accuracy in three-dimensional positioning, while becoming much more accessible than in the early years - balanced with terrestrial methods used for monitoring, has substantial disadvantages, thus being more indicated for tracking objectives with large areas, for which the measurement uncertainty is not so strict.
- Measuring changes of the position of the tracked points by means of GNSS technology it is in direct comparison with the classical methods of determining horizontal, vertical or spatial displacements; the conclusions to be drawn from this comparison are that, for planimetric monitoring, the two approaches are in the same field of precision, thus being clear the option to choose the technique that uses GNSS technology due to the much shorter working time in terms of data acquisition, as well as in their processing. Instead, within the verification works of the vertical displacements of the constructions, the level of precision reached by the GNSS

equipment is clearly lower than the geometric one, being approximately the same as that of the trigonometric leveling method.

- In the case of displacements in the vertical plan, it is therefore recommended, as noted above, especially in the industrial field, to be investigated by means of the geometric leveling, which provides accuracies up to 0.05 mm at a maximum distance of 15 m, or with the help of the hydrostatic levelling, its major advantage compared to the geometric one being the possibility of automating the works, which appeared with the development of electro-optical techniques.
- On the other hand, regarding to the conclusions presented above, for the systematic monitoring and verification of machineries displacements (in industry) in the horizontal plan, the most common methods are those based on alignments, thus the classical instruments such as the alignment scope, is also very suitable for the systematic alignment and verification of guideways, for linearity measurements in the equipment manufacturing industry, or for researching the loading of foundations in machine building. It can be mentioned, however, that perhaps the most balanced and modern systems that can be used for precision alignments are the LASERs, which provide fast, portable, micron-accurate systematic observation means, while probably the most important aspect, increased adaptability to the working environment, their use being extremely little influenced by differences in temperature on the studied object or in the atmosphere.
- The Reflectorless method has many advantages over the classical methods of checking the alteration of the positions of the specific points of a structure, such as: the costs are substantially reduced due to non-use of targets, the measurement time is considerably reduced by automation of the sighting process, the possibility of observing points inaccessible to the mounting of the reflectors and avoiding the problems of targets reflectivity that occur in the long periods of measurements. The particularly important condition that appear in this process is the knowledge of the displacement directions prior to the measurements. As determinable elements are defined, the displacements on the X axis, along the sighting axis, thus, imposing in case the displacement of the point also occurs on the Y axis (perpendicular to the sighting axis), or the use of a classical method simultaneously with the one without reflector, or measuring from two stations, which will record separately position changes in the X and Y direction, respectively.
- For ultra-high accurate determination (in the submillimeter range) of the oscillations or deformations of some objectives, the most recommended option is RADAR interferometry, with the mention that the presented system, although located at the top of the manufacturers hierarchy, due to the constructive principle can be used only for 1D static or dynamic monitoring (1 dimension), at distances not exceeding 1 km; instead, an advantage of this

device is its ability to track a multitude of points simultaneously with frequencies up to 200 Hz.

- Last but not least, the measuring and control equipment not only supports basic instruments and techniques, but can sometimes replace it, thus the fiber optic sensors such as optical extensometers, or the entire SOFO system, tilt sensors, invar wire measurements, clinometers, electronic levels, or perhaps even the simplistic crack-defining apparatus, are used in many monitoring works displaying particularly satisfied results, offering according to the constructive principle of each technology, the option of automation, and small measurement uncertainties, which leads to a short time and low workload, while maintaining the quality of the work.

BIBLIOGRAPHY

1) **Coșarcă C.** – „Sisteme de măsurare în industrie” – Editura Conspress, București, 2009
2) **Dabija A.** – „Contribuții privind analiza deformațiilor” – Teză de doctorat, Universitatea Tehnică de Construcții București, 2018
3) **Dragomir P. I.**, **Clinci T. S.** – „Tehnici de măsurare și poziționare în domeniul apropiat” – Editura Conspress, București, 2014
4) **Ghițău D.** – „Geodezie și gravimetrie geodezică” – Editura Didactică și Pedagogică, București, 1983
5) **Moldoveanu C.** – „Geodezie. Noțiuni de geodezie fizică și elipsoidală, poziționare” – Editura MatrixRom, București, 2002
6) **Neamțu M.**, **Neuner J., Onose D.** – „Măsurarea topografică a deplasărilor și deformațiilor construcțiilor” – Institutul de Construcții București, 1988
7) **Negrilă A.** – „Contribuții la perfecționarea tehnologiilor geodezice de achiziție, prelucrare și interpretare a datelor spațiale în zone de hazard și risc” – Teză de doctorat, Universitatea Tehnică de Construcții București, 2013
8) **Neuner J.** – „Senzori - Tehnici de măsurare și prelucrare” – Note de curs, Facultatea de Geodezie, Universitatea Tehnică de Construcții București
9) **Nistor G.** – „Geodezie aplicată la studiul construcțiilor” – Editura Gheorghe Asachi, Iași, 1993
10) **Onose D.** – „Urmărirea comportării construcțiilor” - Note de curs, Facultatea de Geodezie, Universitatea Tehnică de Construcții București
11) **Ortelecan M., Pop N.** – „Metode topografice de urmărire a comportării construcțiilor și terenurilor înconjurătoare” – Editura AcademicPres, Cluj-Napoca, 2005
12) **Palamariu M., Popa A.** – „Urmărirea comportării terenurilor și construcțiilor” – Editura Risoprint, Cluj-Napoca, 2009
13) **Rădulescu A. T. G. M., Rădulescu G. M. T.** – „Urmărirea comportării terenurilor și a construcțiilor, Metode, Tehnologii și Instrumente” – Editura UTPress, Cluj-Napoca, 2017
14) **Sărăcin A. , Negrilă A. , Clinci T.** – „Posibilități de monitorizare a construcțiilor, utilizând interferometria radar terestră”
15) **Trifan A.** – „Contribuții în domeniul analizei deformațiilor și deplasărilor construcțiilor și terenurilor” – Teză de doctorat, Universitatea Tehnică de Construcții București, 2014
16) **Inaudi D.**, **Casanova N., Vurpillot S., Kronenberg P., Martinola G., Steinmann G., Mathier.** – „SOFO:Structural Monitoring with Fiber Optic Sensors” – FIB, „Monitoring and Safety Evaluation of Existing Concrete Structures”, Vienna, Austria, 12-13.12.1999

Web:

17) https://www.kern-aarau.ch/fileadmin/user_upload/Aldo/Prospekte/Distometer_ISETH.pdf
18) https://lib-extopc.kek.jp/preprints/PDF/1984/8409/8409279.pdf
19) https://www.hahn-kolb.net/Toate-categoriile/Clinometru-electronic/5521CL04_0316150403.cyid/5521.cgid/ro/RO/EUR/
20) https://pdf.directindustry.com/pdf/taylor-hobson/talyvel-5-electronic-levels/7159-390745.html
21) https://www.taylor-hobson.com/products/alignment-level/electronic-levels-and-clinometers/talyvel-6-electronic-levels
22) https://www.dji.com/phantom-4-rtk/info#specs
23) https://idsgeoradar.com/products/interferometric-radar/ibis-fs
24) https://www.datrex.com/product/echomax-trihedral-corner-reflector/
25) https://roctest.com/wp-content/uploads/2017/01/c33.pdf
26) https://leica-geosystems.com/products/levels/leica-nivel210_220
27) https://www.elcometer.com/en
28) https://wingtra.com/fixed-wing-drones-the-evolution/

Legislation:

29) Stas 10493-76 from 1st of January 1976
30) Law no. 8 from July 1977
31) Stas 2745-90 form 1st of Mars 1990
32) Government Ordinance no. 25 from 24th of August 1992
33) Law no. 10/1995 consolidated in 2007
34) Government Decision no. 766 from 21st of November 1997
35) Stas 016-97
36) Normative P 130-1999
37) Law no. 123 from 5th of May 2007
38) Government Decision no. 1231 from 2008