Chapter 18
Operation and Maintenance of Hydraulic Structures

18.1 General

Accident or failure of hydraulic structures in hydraulic projects may give rise to significant consequences ranging from losses of life, injury to economy, and damage to properties and environment. These highlight the necessity and importance of safe operation and maintenance for hydraulic structures. In China, responsibility for the safety of a hydraulic project rests with its owner as well as the local and state government (Electric Power Industry Ministry of the People's Republic of China 1998). It is routine to establish special management agency committed to the safety according to an effective management program for the hydraulic structures, particularly dam, to minimize the risk of failure and to protect the life and properties.

The major tasks in the operation and maintenance of hydraulic structures addressed in this chapter are as follows:

- Hydrologic observation and forecasting;
- Safety surveillance of hydraulic structures;
- Instrumentation of hydraulic structures;
- Remedial actions;
- Detection and mitigation of aging for hydraulic structures.

18.2 Hydrologic Observation and Forecasting

Safe operation of hydraulic project is based primarily on hydrologic forecasting, which is, in turn, dependent on water regime observation and prediction (Becker and Yeh 1974; Can and Houcks 1984; Eschenbach et al. 1999; Windsor 1973).

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18.2.1 Water Regime Observation

Before the 1980s, the water regime observation and prediction in China's reservoir catchments relied on manual operations and messengers, i.e., the messages concerning hydrology and weather were transferred by telegraph or telephone. From the 1980s, modern automation facilities for water regime observation and prediction were initiated employing the technologies imported abroad, whose first successful application was an IASP automation system (from EU) installed in the Danjiangkou Project (Hubei, China). From 1985, China started to produce automation facilities domestically for water regime observation and prediction, which was firstly installed in the Huanglongtan Project (Hubei, China). Since then, the domestic systems have been widely exercised in a number of hydraulic projects. Nowadays, there are more than 300 water regime observation and prediction systems serving the large-to-medium projects in China.

Figure 18.1 is the schematic diagram of the water regime observation and prediction system of the Huanglongtan Project. Altogether 19 precipitation stations, 5 hydrometric stations, 3 relay stations, 1 data acquisition center, and 1 central station are integrated in the system.

18.2.2 Hydrologic Forecasting

The analysis and estimation technique for determining the hydrologic status in the future via the hydrologic status at earlier and present stages is named as hydrologic

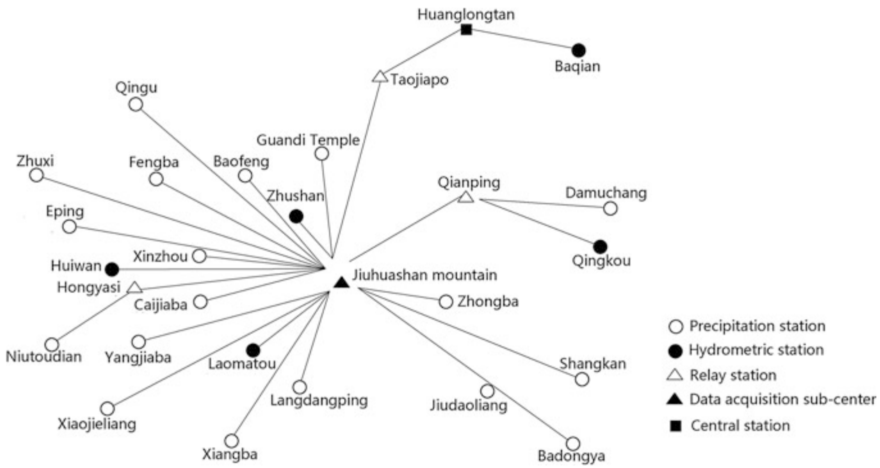


Fig. 18.1 Schematic diagram of the water regime observation and prediction system—the Huanglongtan Project, China

forecasting. It plays an important role in the safe construction and operation of hydraulic projects.

Hydrologic forecasting must provide the forecasting value and forecasting period with competent accuracy. According to the forecasting period, the hydrologic forecasting may be generally distinguished as follows:

- Short-term flood forecasting of several days, whose theoretical base is the theory of runoff yield and flow concentration and mainly serves the purpose for the flood-control operation;
- Mid- and long-term hydrologic forecasting, which is generally developed in the way of meteorology or statistics and mainly serves the purpose for the electric generation operation.

The study on the short-term flood forecasting operation commenced in the early 1950s in China, which concerns the following three basic issues:

- The flood forecasting for a river reach, by which the water regime of downstream section is forecasted;
- The watershed rainfall–runoff forecasting, by which the flood hydrograph at watershed outlet is forecasted using rainfall over the watershed;
- The flood forecasting of stream watershed.

China started to use the stream hydrologic forecasting system to make flood forecasting in the late 1970s. Insofar, a number of stream hydrology forecasting systems have been developed successfully, such as the flood online and real-time forecasting system of the Yangtze River, the Yellow River, the Huaihe River, etc. The combination of these systems with the real-time information processing system and operation system may quickly provide the results of forecasting as long as the information can be received.

It should be indicated that the forecasting accuracy would deteriorated with longer forecasting period due to the presence of chaos. Therefore, methods of mid- and long-term hydrologic forecasting are generally different from those of short-term flood forecasting. At present, there are three types of mid- and long-term hydrologic forecasting methods employed in China:

- Meteorology methods. Based on the relationship between the long-term change of runoff and large-scale climatic change, the long-term change rule of hydrologic elements is forecasted using antecedent atmospheric circulation characteristics;
- Statistic methods. Based on analyzing a lot of long-term historical hydrologic and meteorologic data, and the statistic relationship between the forecasted objective and factors, the mid- and long-term change of hydrologic elements is forecasted;
- Space–earth physics methods. The relationship between the change of space–earth physical factors and the long-term change of hydrologic elements in the regions concerned is founded as the basis for the mid- and long-term hydrologic forecasting.

18.2.3 Reservoir Operation

The reservoir operation is accomplished by normal operation and optimum operation. The normal operation applies the runoff regulation theory and the hydroenergy computation method to determine the reservoir storage–draft process satisfying the task defined by the reservoir operation policy. The optimum operation is firstly to set up the objective function and constraints for hydroelectric system, and then, the systematic equations consisting of the objective function and the constraint conditions are solved using the optimal method, and the control operation manner of the reservoir with maximum/minimum objective function value is obtained.

Reservoir operation concerns aspects of flood-control operation and beneficial operation. China is one of the countries where flood disasters frequently take place. Therefore, the emphasis of reservoir management is commonly on the flood-control operation. However, the development of an operation policy for a multi-purpose reservoir, particularly when the beneficial purposes are combined with flood control, is a complex task attributable to the conflicting nature of the different purposes: A full reservoir is needed to maximize returns from beneficial uses, while an empty reservoir gives rise to maximum benefits from flood control. Therefore, the operative study should be able to optimally resolve the conflicts among the various purposes.

The optimal operation of a single reservoir was realized firstly in USA in the 1940s, which was later introduced into China in the 1960s. The study and application were based on a variety of mathematical tools such as the dynamic programming and the Markov process theory, the countermeasure theory and the practical tactic iteration method, the nonlinear programming model and the multi-dimension dynamic programming model, the multi-purpose dynamic programming model and the flood-control real-time optimal operation model.

In the recent 20 years, real-time flood forecasting and operation of reservoir becomes more and more prevalent, which consists of the following three parts:

- The real-time rain and water regime information sub-system;
- The real-time flood forecasting sub-system; and
- The real-time reservoir operation sub-system.

18.3 Safety Surveillance for Hydraulic Structures

For a hydraulic structure such as a dam, safety management program begins with the initial investigation of the dam foundation and continues through its design, construction, and operation (Dunnicliff 1988, 1990; Gu and Wu 2006; ICOLD 1988a, b, 1989, 2014; Iida et al. 1979; Luzi et al. 2010; Malakhanov 1990; Merkler et al. 1985; Wu 2003; Wu and Gu 1997). While many problems may emerge and need to be handled during these phases, there is always a risk that not all problems have manifested themselves or been detected by the time the dam has been completed.

Surveillance is the continuous examination of the physical condition and operation of hydraulic structures such as dams. Surveillance program should be capable of detecting problems related to unsafe factors at an early stage so that remedy measures can be taken on time to secure the structure safety without compromise. To obtain a historical context for defects, surveillance should commence as early as possible in the whole life cycle of the structure, to detect the development of any unsafe trends and to provide full background information on a structure's performance. Any unusual behaviors, regardless of how seemingly insignificant, should be identified and documented because this may be the forewarning of a newly developed unsafe scenario.

Each hydraulic structure should have its own surveillance program, whose scope should be appropriate to the size of the dam and the storage of the reservoir, the population at risk and other consequences of dam failure, and of course the value of the structure to the owner.

The surveillance program should comprise a range of inspection activities from routine inspections by operational staff through to comprehensive inspections by experts; a system of comprehensive monitoring by instrumentation for collecting information or data relating to structural performance; and a series of safety evaluation (safety review or safety appraisal) based on the interpretation of the collected data.

In the following, dams will be taken as example to elucidate the principles of the surveillance program for hydraulic structures.

18.3.1 Safety Inspection for Dams

One of the most important activities in a surveillance program for hydraulic structure, particularly dam, is the frequent and regular safety inspection for its abnormalities and deteriorations, and for determining its status and features related to the structural and operational safety (Duscha and Jansen 1988). Different types of safety inspections should be undertaken for different purposes such as routine inspections, periodic inspections, and special inspections.

1. Routine inspections

The purpose of routine inspections is to identify physical deficiencies of the dam, which are commonly undertaken by the dam owner, as a part of their normal duties at the dam. Routine inspections are best carried out by someone involved in the day-to-day running of the dam such as the field and operating personnel, because much of the inspection and observation should be incorporated in the daily work. The standing operating procedures (SOP) should outline the guidelines with regard to the time and frequency of the inspections, who should be involved, and the inspection reporting requirements. However, there is no report standard for these inspections as they can vary from a short weekly check for a small farm dam to a twice-daily check for a large dam. Routine inspections are generally carried out on one-weekly basis during construction period, on 1–2 daily bases during impounding period, and one-monthly basis during normal operation. In flood season or high-storage-level situations, the inspection frequency should be raised.

For concrete dams, the inspection checklist may comprise an advisable content in the following:

- Dam body. Differential movement between adjacent dam monoliths; expansion of joints and water stops; external surface cracking and leaking; damage and erosion and leaching of concrete; state of draining holes; seeping discharge and its chemical composition; etc.
- Dam foundation and abutments. Squashing, shearing, loosening, spalling, etc., in rock; shearing, cracking, and leaking, at the contact surface of dam and foundation; cracking, sliding, leaching, and bypass seeping, at the abutment rock mass; draining devices; seepage discharge and transparency (chemical composition).
- Diversion and flood releasing works. Silting and blocking and damage of intake inlets; cracking and damage of flood releasing works; scouring and abrasion of energy dissipaters; silting and scouring of downstream riverbed.
- Others. Outcrop variation of groundwater around dam site; cracking variation in bank slopes; situation of gate, slot, support, and sealing; hoist, electricity control system, and backup electric sources.

For embankment dams, the checklist may comprise an advisable content in the following:

- Dam cracking, sliding, collapsing, scouring, slope toe heaving.
- Downstream slope dispersed saturation, mechanical piping, pop-off, swamping; abnormal variation of pressure relief devices (relief wells, drain ditch, filters, etc.).
- Abnormality of slope protection.
- Damage and silting of surface draining.
- Activity of harmful insects (e.g., termite) and animals (e.g., rats and badger).

2. Periodic inspections

Periodic inspections are generally carried out by engineers for the purpose of identifying physical deficiencies of the dam by visual examination and analysis of monitoring data against prevalent knowledge. The inspection report should fully document the status of the dam and all deficiencies or unsafe factors and outline a strategy for taking remedial action. These inspections are generally carried out on 2–3 yearly bases.

The time of periodic inspection depends on the regional weather pattern. For example, if a distinct wet season exists, inspections are advisable to be carried out immediately after the wet season, to allow for remedial work to be planned and undertaken prior to the next wet season.

3. Special inspections

The purpose of special inspection is for a particular physical feature or operational aspect of a dam due to some special reasons, which, for example, has been identified as having a possible deficiency or has been subject to abnormal loading conditions (e.g., earthquake and check flood). Special inspection is undertaken by specialists in dam engineering. The inspection report should fully document the status of the particular physical feature or operational aspect of the dam as well as any other deficiencies or unsafe conditions and outline a strategy for taking remedial action. Special inspection is often carried out with a degree of urgency, which will merely address issues that relate to the subject feature, and in addition to the routine and periodic inspections.

18.3.2 Monitoring for Dams

Monitoring is the collection, presentation, and evaluation of information by instrumentation devices installed at/in the hydraulic structures, which is intended to detect deterioration with respect to the actual performance of the structure, to detect structural trends or behaviors for establishing compliance with design expectations or providing some confirmation of the validity of design assumptions, to rectify structure design issues which could not be resolved to high reliability during the design and construction stages, and also to establish an initial datum pattern of performance against which subsequent observation can be assessed.

Today, the provision of monitoring instruments is accepted well for all new major hydraulic structures in large-to-medium projects. Instruments must cover known critical features of the dam, but for the purpose of comparison, some of them also should be placed at locations where normal behavior is anticipated. The planning and specification of a comprehensive suite of instruments involves a logical sequence of decisions, and it is good practice to draft an ideal instrumentation plan in the first instance and then to progressively eliminate the less essential provisions until an adequate, balanced, and affordable plan is determined.

In addition, a basic level of instrumentation is now frequently installed retrospectively to monitor existing major hydraulic structures of large-to-medium projects.

1. Parameters in monitoring

The scope and degree of sophistication of individual suites of instruments varies greatly. The designer, review engineer, or inspection engineer should identify the issues that need to be monitored and incorporate appropriate instrumentation into the dam. For instance, for a farm dam, it may be concluded that there is no need for any instrumentation. For the hydraulic structures of grades 1 and 2, the most significant parameters in monitoring are grouped as follows:

- Work condition monitoring (environmental factor monitoring). Including up- and downstream water levels, reservoir water temperature, ambient temperature, silting in front of dam, and downstream silting and scouring.
- Seepage and leakage monitoring. Including discharges of under seepage, bypass, transparency and chemical analysis of seeping water, uplift in concrete dam body, phreatic line in embankment dam body, and uplift in foundation.
- External and internal deformation. Including horizontal and vertical displacements, open and shear of joints and cracks, flexural and inclined deformation of concrete dam, and consolidation of embankment dam.
- Stress/strain and temperature. Including stress and strain within concrete dam, stress of steel bar, stress of the steel plate of penstocks and spiral case, temperature in concrete and foundation rock, pore pressure in embankment dam, and earth pressure.
- Others. Including bank slope stability of dam site, seismic response of dam, and hydraulic items.

2. Frequency of monitoring

The preferable frequency of monitoring varies over time and is related to the factors with respect to the nature of the performance being monitored, the stage of maturity of the dam, and the existence of any problems or events.

The first impounding and the following five years, usually once of monitoring is demanded for each ten days or each month, of which the first impounding period requires once of monitoring for each day or ten days. After the five years of service, usually once of monitoring is required for each month or quarter. The headwater level and tailwater level and ambient temperature should be taken every day. The internal monitoring frequency should be concentrated from the embed time within one month, in a series of 4 h, 8 h, 24 h, 5 d, until routine interval. Special events, such as record floods and earthquakes, will demand more intensive monitoring. Frequency of monitoring is subject to adjustment after long period of service.

18.3.3 Safety Review for Dams

The safety review of a dam is to assess its safety based on data obtained from safety inspections and monitoring, which can be quite complex and personnel engaged. Safety review is generally undertaken by the safety agency belonging to the water resources administration of the state. Where necessary, the services of suitably experienced geologists, hydrologists, and other specialists should be provided. Consideration should also be given to independent review by engineers other than those who carried out the original design of the dam.

The frequency of safety reviews is generally based on the age of the structure and the appropriateness of the technology used on that structure—usually within 3–5 years firstly from the starting of service. After the first safety review, the safety review is carried out at interval of 5–10 years (USA 5–6 years, France 5–10 years). For dams having served over 30 years, comprehensive checking and safety review is demanded, since the natural conditions (such as silting, scouring) and operation mode could be changed greatly after long service, problems of aging of appurtenance structures and materials are exposed obviously, and the same generation of experts concerning the dam design, construction, and management, are still living, which enable to reveal incipient faults.

Following the safety review, a safety review report should be documented by experienced dam engineers familiar with the entire history of the dam. A safety review report should comprise the following:

- Statement on the safety indicating whether or not the dam is in a satisfactory condition and capable of meeting current design criteria;
- Report on comprehensive inspection;
- Parameters adopted and assumptions made (and their bases for review analyses);
- Methods of review analysis and results (numerical and physical);
- Identification of any deficiencies in the dam including criticality ratings for these deficiencies;
- Recommendations for remedial work, emergency action, and/or further studies which should be undertaken and schedules for these activities.

By the safety review, dams are classified into three groups as dangerous dam, defective dam, and normal dam. Anomalies and concerning trends identified in the report should be considered as deficiencies. It is the responsibility of the dam owner to ensure that appropriate remedial actions are taken and documented.

18.4 Instrumentation for Hydraulic Structures

18.4.1 Deformation Monitoring

All structures move as the effects of exerted actions. The structural movements can be divided into three types: surface movement, internal movement, and crack or joint movement. Surface movement is defined as horizontal or vertical deflection of a point on the structure surface relative to a fixed point off the structure. Internal movement is defined as horizontal or vertical deflection within the structure relative to some points on the structure or in the foundation. Joint or crack movement is defined as slip, opening, or compressing of one part relative to another part, of a structure.

Movements in response to actions are usually normal and acceptable, provided that they are within tolerable ranges and do not cause structural damage. Embankments are less brittle than concrete structures and can undergo larger movements without distress. As a result, measuring of surface movements of embankment dams is typically less precise than that for concrete structures. Sudden or unexpected direction, magnitude, or trend of surface movement could indicate developing deficiencies.

Measuring points for all movement should be so installed that they are not subject to the movement from freezing/thaw or traffic actions.

1. Horizontal displacement and deflection monitoring

Conventional methods prevalent are alignment surveys covering an extremely wide spectrum of engineering applications, and each of them may require specialized equipment. They have been used in the past and will surely continue to be used for many years. Alignment surveys are accomplished using a "line of sight" as reference line (or benchmark line) by optical or mechanical ways (Williams 1993). Mechanical alignment employs a reference line established by stretched wire (e.g., steel, nylon), such as the plumb line and tension wire. Direct optical alignment (collimation line method) employs optical line of sight or a laser beam to build the reference line. In diffraction alignment, the reference line is created by projecting a pattern of diffraction slits (laser alignment). Geodetic measuring techniques, combined network of triangulation and trilateration, etc., are also widely exercised. For concrete dam foundations and embankment dams, the internal horizontal displacement also may be measured by extensometers, inclinometers, etc.

(a) Plumb lines

Plumb lines, inverted plumb lines, and optical plummets are designed to measure bending, tilting, or deflecting, of concrete structures resulting from external loads, temperature changes, sliding, or deformation of the foundation. Through the measurement operation, they will provide information with regard to the general elastic behavior of the entire structure including foundation.

The plumb line consists of a plumb bob suspended from a wire in a vertical shaft within the structure. Measurements for the location of the wire relative to the suspension point are taken at one or more elevations along the shaft by a micrometer or microscope. Plumb lines are simple, inexpensive, accurate, and reliable. These instruments should be located in the structure where unusual structural deflections are anticipated or where information on deflection is required. They should be located in the highest monoliths of the structure and at locations where reading stations will be easily accessible. These reading points are customarily provided in one or more of the galleries in the lower portion of the structure and at other elevations, if practicable. Plumb bob systems based on an inverted pendulum or deflectometer may be installed in structures where a reading station cannot be constructed near the base of the structure, or where it is desired to extend the reference points into the foundation.

Two kinds of mechanical plumbing are prevalent, i.e., suspended plumb line (Fig. 18.2a) and floating or inverted plumb line (Fig. 18.2b). The former installs the fix end near the dam crest, tensions the lowest end with plumb bob; the latter consists of an anchor at the base or in the foundation, a plumb wire, and a floating assembly at the top end of the plumb line which moves freely in a container of oil

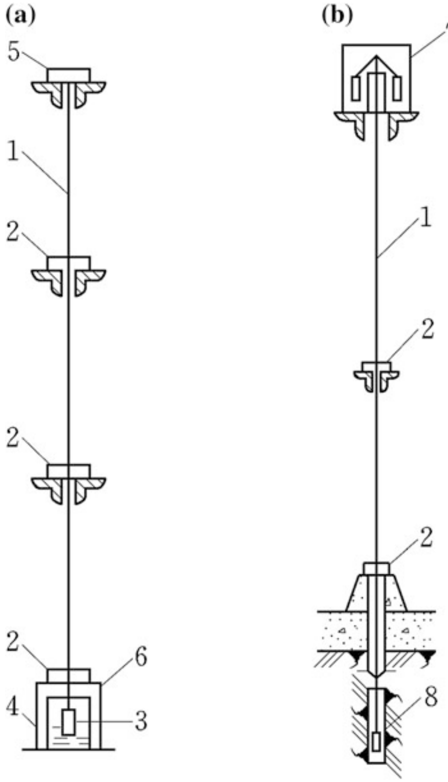


Fig. 18.2 Schematic diagram of plumb lines. **a** Suspended plumb line; **b** floating or inverted plumb line. *1* plumb line; *2* measuring instrument; *3* plumb bob; *4* oil container; *5* fixed end; *6* measuring pier; *7* floating assembly; *8* anchor point

and establishes the plumb of its wire from being perpendicular to the surface of the oil. Inverted plumb lines are used in conjunction with conventional plumb lines to extend the length of measurable plumb path and to make reading of a long path easier since both reading stations can be combined in the same shaft. Inverted plumb lines have an advantage over suspended plumb lines in the possibility of monitoring absolute displacements of structures with respect to deeply anchored points in the foundation rock that may be considered as fixed. In the case of dams, the depth of the anchors must be 50 m or even deeper below the dam foundation in order to obtain absolute displacements of the surface points. If invar wire is used for the inverted plumb line, vertical movements of the structure with respect to the bedrock can also be determined.

The number of plumb lines is related to the engineering scale, dam type, and monitoring requirements, and usually, it is no less than 3 for large dams and 2 for medium dams. For a dam with special structure features, it may be divided into 2–3 sections along the height, for each section a plumb line is installed. For each plumb line, no less than 3 observing points should be arranged.

Several types of recording devices that measure displacements of structural points with respect to the vertical plumb lines are available from different companies. The simplest are mechanical or electromechanical micrometers. With these, the plumb wire can be positioned with respect to reference lines of a recording (coordinating) table to the accuracy of ±0.1 mm or better. Traveling microscopes may present a same accuracy. The microscope or micrometer slide is precision instrument designed for precise laboratory work and should be used in a manner conforming to good laboratory practice. So far as practicable, all microscope readings should be made by the same individual personnel who is thoroughly familiar with procedures prescribed herein. Whenever it becomes necessary to change observers, either for a short period of time or permanently, the recommended step-by-step operations to be followed in making readings should be carefully explained and demonstrated to the new observer. A written instruction sheet often will be found indispensable.

Automatic sensing and recording is possible, for instance, with a telecoordinator (Huggenberger, Switzerland) and with a telependulum (Telemac, France).

Two sources of error that may sometimes be overlooked by users are the influence of air currents and the spiral shape of the wires. Therefore, the plumb line should be protected within a pipe (e.g., PVC tube) with openings only at the reading tables to reduce the influence of the air currents.

The reading station should be located in the lower portion of the structure in the case of a conventional plumb bob system, as a recess in one of the galleries. There are two measuring methods for the plumb line:

- One-point support and multi-point measurement. This is exercised both for suspended plumb lines and for inverted plumb lines. The measured data of suspended plumb lines are the relative horizontal displacement of the suspended point with regard to the measuring point, denoted as S in Fig. 18.3a, while the deflection at the measuring point N is $S_N = S_0 - S$. The measured data of

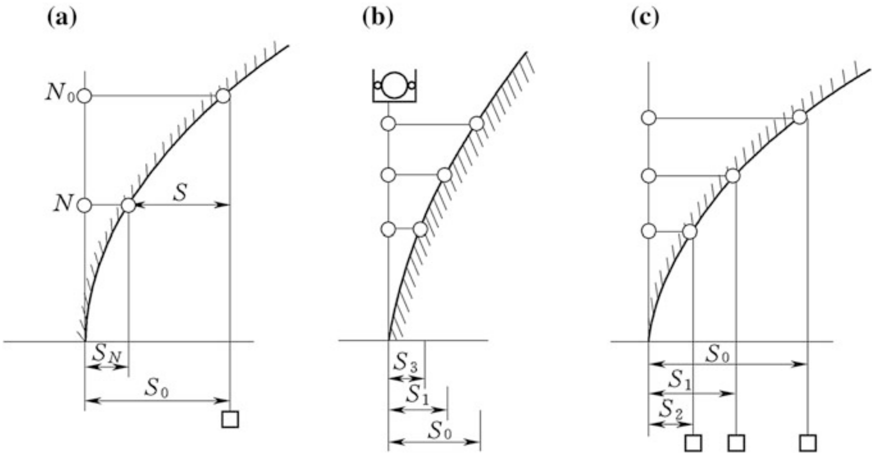


Fig. 18.3 Diagram to the measuring principles of plumb line. **a** One-point support and multi-point measuring (suspended plumb line); **b** one-point support and multi-point measuring (inverted plumb line); **c** multi-point support and one-point measuring (suspended plumb line)

inverted plumb lines are the relative horizontal displacement of the measuring point with regard to the anchored point in deep foundation, as shown in Fig. 18.3b.

- Multi-point support and one-point measurement (Fig. 18.3c). This is only exercised for suspended plumb lines. Supporter is installed at each measuring point, which clamps the plumb line in sequence during the measuring operation. The instrument is installed at the bottom of the plumb line, to obtain the relative horizontal displacement with regard to the measuring instrument.

(b) Tension wire alignment

Tension wire alignment belongs to mechanical alignment methods, where tensioned wires of 0.8–1.2 mm in diameter using stainless steel are installed as the reference lines. It has found many applications including dam deformation surveys, attributable to its simplicity, high accuracy, and easy adaptability to continuous monitoring of structural deformations using inductive sensors applicable over distances up to several hundred meters. Accuracies of 0.1 mm are achievable using mechanical alignment method.

Tension wire alignment comprises benchmark monument piers, reading points, stainless steel wire, and protecting pipe of wire. Tension wire is commonly installed at the dam crest or at longitudinal galleries of different altitudes, while the benchmark monuments are installed on the banks. Where restrained by topographic conditions, monuments also may be installed on dam, and under such circumstances, the displacements of monuments should be measured by the other methods, to obtain absolute horizontal displacement.

Monument piers are constructed using reinforced concrete, on which tension disk, pulley, and plumb bob are installed. On the reading point, there is a floating box and a staff gauge, as shown in Fig. 18.4. The protect pipe for wire is ordinarily transparent PVC tube of 10 cm in diameter.

Tension wire alignment is widely exercised in gravity dams, such as the Gezhouba Project (H = 53.8 m, China) and the Danjiangkou Project (H = 97 m,

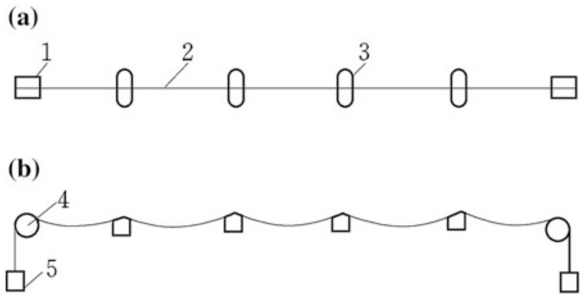


Fig. 18.4 Schematic diagram of tension wire alignment. **a** Plan; **b** elevations. *1* end; *2* tension wire; *3* displacement reading point and floating box; *4* pulley; *5* plumb bob

China). Automatic remote sensing and recording are also possible to improve its monitoring accuracy and efficiency.

(c) Collimation line

Collimation line belongs to direct optical methods utilizing either an optical telescope and movable targets with micrometric sliding devices or a collimated laser beam (projected through the telescope) and movable photo-centering targets. In addition to the aforementioned influence of atmospheric refraction, pointing and focusing are the main sources of error when using optical telescopes.

Horizontal surface movements are conventionally measured as offsets from a baseline. The measuring points for the level surveys are normally used for alignment surveys, too. The methods and equipments depend on the type of dam and the desired accuracy.

For embankment dams, one or more lines of measuring points are established along the crest and on the slopes parallel to the crest. Instrument and target monuments are set at the ends of the lines on the abutments far beyond the dam. To measure the dam movement, a theodolite is laid on the instrument monument on one abutment and sighted to the target monument on the opposite abutment. Offsets from the line of sight are then measured for each measuring point using a plumb bob and tape. Typically, survey methods and equipments should be sufficiently accurate to discern a movement on the order of 30 mm.

For concrete dams, a similar procedure is employed, but with refinements to raise the measuring accuracy. Measuring points are established along straight lines on the crest and, in some cases, along the face of the dam. The measuring points are markers set in the dam concrete. Instrument and target monuments are established outside the dam at the baseline ends on the banks. The monuments are customarily 0.2–0.25 cm diameter concrete-filled pipes buried at least 3 m into the ground. The top of the instrument monument is fitted with a threaded plate to accommodate a theodolite. The target monument is fitted with a threaded plate to accommodate a target. The line of sight is established using a high-precision theodolite set on the instrument monument and sighted to the target on the target monument. Offsets from the baseline are measured with a micrometer attached to a moveable target leveled over each measuring point. Typically, survey methods and equipments should be sufficiently accurate to discern a movement on the order of 3 mm.

Alignment surveys are the simplest for determining horizontal movement in straight dams. However, their application is restricted in cases of curved dams, irregularly shaped dams, or where the line of sight is limited and the number of measurement points along any line is insufficient. Another limitation of alignment surveys is the labor cost, although modern surveying equipments have reduced the time needed to perform a survey.

Collimation line using theodolite is convenient, but its accuracy is not competent for long-distance observation, due to the influence of telescope amplification and refraction of the theodolite. Therefore, the collimation line for concrete dams has a tendency of being replaced by the plumb line and tension wire alignment.

(d) Laser alignment

Laser alignment belongs to diffraction alignment methods, in which the pinhole source of monochromatic (laser) light, the center of a plate with diffraction slits, and the center of an optical or photoelectric sensor, establish three basic points, as shown in Fig. 18.5. If two of the three points are fixed, then the third may be aligned by centering the reticule on the interference pattern created by the diffraction grating.

As shown in Fig. 18.6, the pinhole source of monochromatic (laser) light is located at position A, and the zone plate is located at position i. If the point i moves toward j by a deflection l_i, the deflection is L_i on the sensor. According to the principle of similar triangles, we have

$$l_i/L_i = S_{Ai}/S_{AB} \tag{18.1}$$

where S_{Ai} and S_{AB} = distance from the point A to the points i and B, respectively.

Let

$$S_{Ai}/S_{AB} = K_i \tag{18.2}$$

where K_i = correction coefficient of the deflection at B with respect to the reading point i.

Therefore, Eq. (18.1) may be rewritten as

$$l_i = K_i L_i \tag{18.3}$$

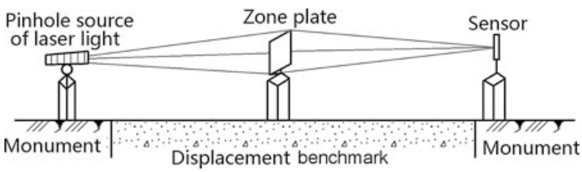


Fig. 18.5 Laser collimation with zone plate

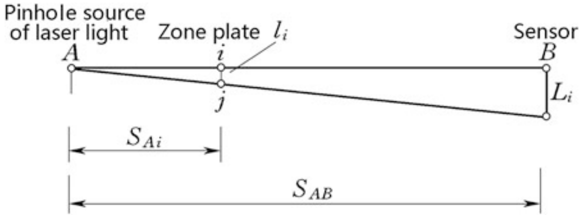


Fig. 18.6 Diagram to the deflection computation of laser collimation with zone plate

It should be pointed out that the movements of the laser and of its output do not influence the accuracy of this method because the laser serves merely as a source of monochromatic light placed behind the pinhole and not as the reference line. Therefore, any kind of laser may be employed in this method, even the simplest and least expensive ones, as long as the output power requirements are met. Various patterns of diffraction slits are used in engineering practice. The highest accuracy and the longest range are obtained with the so-called Fresnel zone plates that perform as focusing lenses.

The influences of the thermal turbulence of air in the open atmosphere may be reduced by vacuum pipe. Laser collimation with airless pipe in a vacuum, rectangular Fresnel zone plates with an electro-optical centering device was used in the deformation measurements of a 3-km-long nuclear accelerator giving relative error of 10^{-7} with respect to the distance measured. The key issue in the laser collimation with airless pipe is the rational value of the vacuity in tube.

The laser diffraction alignment has been successfully performed in the monitoring of both the straight and curved (arched) dams using self-centering targets with automatic data recording. In December 1978, the first laser collimation with airless pipe was installed in the Fengman Gravity Dam (H = 90.5 m) in China, which is 194 long and possesses 4 reading points. Until 1984, altogether 999.4 m-long laser collimation using airless pipes and with 52 reading points had been installed on the crest of the same dam and in its galleries, and automation measuring was accomplished in 1986. So far, more than 2000 laser collimations with airless pipes have been installed for the displacement monitoring in the Three Gorges Project (H = 175 m, China).

(e) Borehole inclinometer

Inclinometers are commonly installed in vertically drilled holes in dams and foundations/abutments. They are instruments ideally suited to long-term and precise monitoring of the position of a borehole over its entire length.

An inclinometer consists of specially shaped plastic casing with four longitudinal grooves cut in the inside wall, a probe that is lowered down the casing on an electrical cable with graduated depth markings (Fig. 18.7) and readout device.

The probe contains two accelerometers detecting the inclination angle of gravity acceleration g at the relative plane of their axis by measuring the tilt of the probe in two mutually perpendicular directions. The probe is also equipped with a pair of wheels that run in the grooves in the casing and maintain the rotational stability of the probe. Inclination of the casing is measured using the probe at regular intervals by which the lateral movement with respect to the bottom of the casing is calculated. By making a series of readings over time, it is also possible to monitor the rate of movement.

The primary requirement for accurate measurement is to extend the borehole below the depth of movement so that readings made from the end of the hole are referenced to as a stable base. Precautions are also needed during the installation of the casing to maintain the vertical alignment of the grooves and to prevent spiraling. Readings are made by lowering a probe to the end of the hole and then raising it in

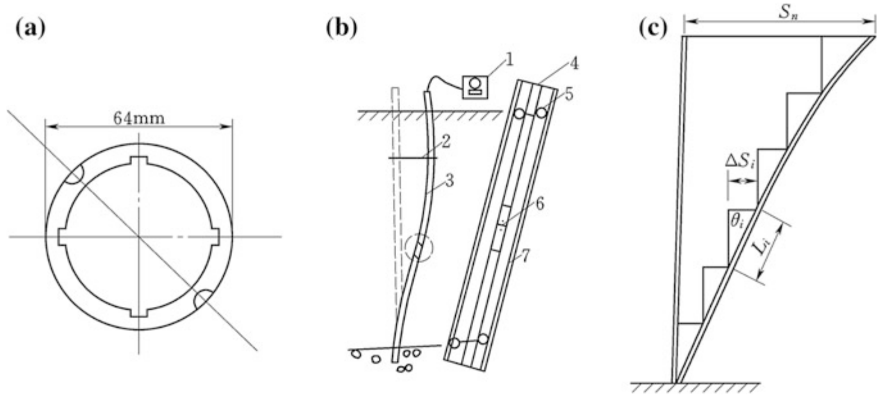


Fig. 18.7 Schematic diagram of inclinometer

increments equal to the length of the wheelbase L (=0.5 or 1 m) of the probe. At each depth increment, the tilt θ_i is measured, and then, the displacement $(L \sin \theta_i)$ for each increment and the total displacement at the top of the hole $\left(\sum_i L \sin \theta_i\right)$ are calculated. A check of the results is conventionally made by rotating the probe by 180° and taking a second set of readings. Another precaution is to allow time during the readings for the probe to reach temperature equilibrium in the borehole.

2. Vertical movement and inclination monitoring

Vertical surface movements are commonly measured by differential leveling techniques including geometric leveling and hydrostatic leveling (Unguendoli 1984). Reading points are established on the crest or slopes of the dam. Reading points for embankment are commonly materialized using steel bars embedded in concrete placed in the fill, whereas reading points for concrete dam are commonly materialized using bronze markers set in the concrete or scratch marks. Typically, survey methods and equipments for embankments should be sufficiently accurate to discern a movement on the order of 30 mm, whereas survey methods and equipments for concrete structures should be sufficiently accurate to discern a movement on the order of 3 mm. Level surveys are simplest and most accurate for determining vertical movement of a hydraulic structure.

(a) Geometric leveling

Geometric leveling is an old method of geodetic surveying employed to measure elevation difference between two points at the structure surface. Two types may be distinguished as geometric (or direct) leveling and trigonometric (or indirect) leveling. The geometric leveling is featured of highly precise, lighter monument, less expensive in equipment and fieldwork, and easier in operation procedures.

In geometric leveling, the difference of height between two points is determined by the difference of readings to the staffs placed on those points. The readings are made with a leveling instrument such as optical level.

An optical level consists of a telescope fitted with crosshairs and rotating around a vertical axis, with a very sensitive spirit level or other device fixed to it that enables the line of sight to become horizontal. The reading on a graduated vertical staff is measured through the telescope. If staffs are placed on successive ground points, and the telescope is truly leveled, the difference between the readings at the crosshairs will be equal to the altitude difference between the points.

The points in a leveling line fall into three categories:

- Object points, i.e., points that are to be measured;
- Reference points;
- Benchmark points.

Under certain circumstances, auxiliary points also may be installed.

Benchmark points are the basis of absolute vertical displacement leveling. They should be located at a proper distance from the structure without the influence of its displacement and with solid foundation. For medium-to small project, one or two benchmark points are sufficient, whereas for large project, two or three benchmark points may be needed; for particular large project, fine leveling network system is demanded, which includes all benchmark points and reference points.

When the reference points are located in the area to be controlled (and therefore might undergo displacements), only relative displacements can be determined. Where reference points are located outside of that area, tied to bedrock or other non-moving structure, absolute displacements can be determined. Although theoretically only one reference point is needed in leveling lines, the experience advises to place at least three reference points, to identify unstable reference points. Auxiliary points are placed, for instance, to avoid too long distances between level and staffs or to link sectors of a leveling line that, otherwise, would be independent.

For concrete dams, object points in rows should be layout both on the dam crest and in foundation gallery, while another row of object points is advisable to be installed in a gallery at middle elevation. Each dam monolith ordinarily possesses one object point, while additional point may be set for important portion. For embankment dams, the object points are ranged along transverse sections located in controlling profiles. The monitoring sections are spaced 50–100 m apart, and at least 4 object points should be set for each monitoring section. Figure 18.8 shows the layout for the vertical movement measurement of dams.

Object points are usually materialized by pegs sealed on the floor or, less usually, by metal pieces, sealed on a wall. The first one is used to place staffs with inferior support, and the second one is to place hung measuring rules. All points must be well tied to the structure concerned. Sometimes, it is necessary to place the points in protected places to prevent them from damage.

Automatic levels, i.e., optical levels with a built-in compensator that employs an extremely sensitive pendulum device, which automatically makes the line of sight horizontal, could be employed. To improve the accuracy, a parallel plate

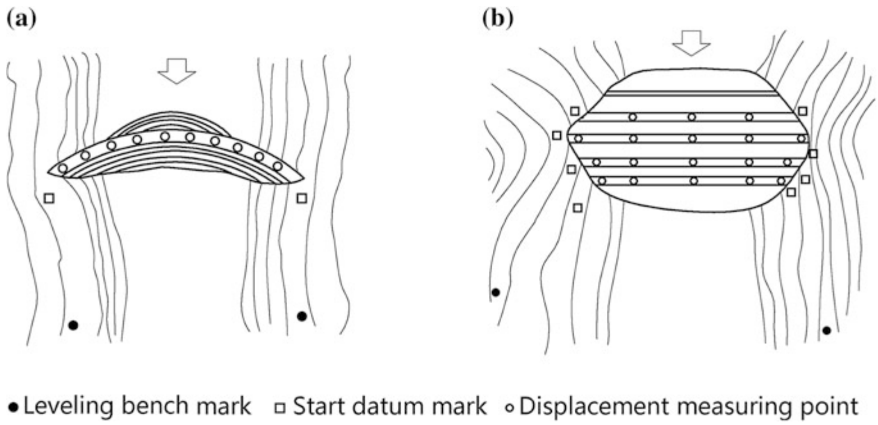


Fig. 18.8 Layout for vertical movement monitoring of dams. **a** Arch dam; **b** embankment dam

micrometer must be fitted over the telescope objective, which permits direct readings on a centimeter graduated staff, to 0.1 mm, and estimated readings, to 0.01 mm. Digital levels are automatic ones with a built-in digital image processing system that permits automatic reading of special staffs (coded bar) and electronic recording. Errors caused by manual reading and recording are eliminated, and the speed of leveling can be raised (by about 30 %).

Rigid invar staffs with scales engraved directly into the paint coat on an invar strip are made of nickel steel alloy that has a linear thermal expansion coefficient of 0.7×10^{-6} °C (about 15 times smaller than steel)—a quite important characteristic since the measurements would be undertaken in extreme temperature conditions.

For transferring elevation to the gallery, the shafts for plumb lines or vertical galleries may be employed as shown in Fig. 18.9, where A is a point on the dam with elevation H_A. To measure the elevation of B within the gallery, invar steel rule is hung in shaft, under which the rule plumb bob is submerged in floating box filled with transformer oil for the stability of the rule. Theodolites are installed both on the dam surface and in gallery, and invar staffs are installed at both A and B. The altitude at B is determined by the readings a_1, b_1, a_2, b_2 as follows:

$$H_B = H_A + a_1 - (b_1 - a_2) - b_2 \tag{18.4}$$

in which the readings b_1 and a_2 of invar steel rule should be revised according to the temperature and rule length.

(a) Hydrostatic leveling

Hydrostatic leveling may be employed to measure differential settlements of floors, footings, columns, walls, and galleries in dams or any structures.

If two connected containers are partially filled with a liquid, then the heights h_1 and h_2 of the liquid in the containers are related through the hydrostatic equation.

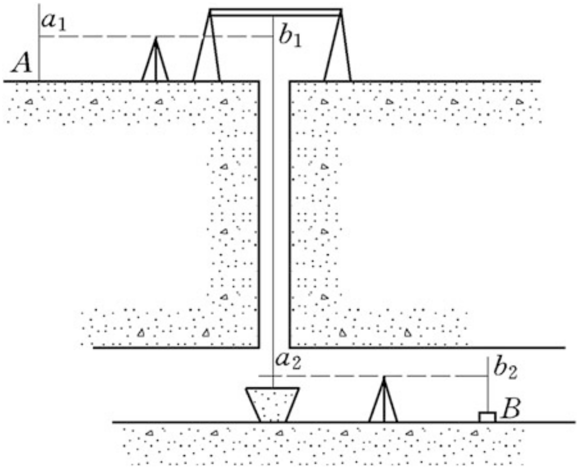


Fig. 18.9 Elevation transferring by shaft

Hydrostatic leveling is frequently exercised in the form with a network of permanently installed instruments filled with a liquid and connected by hosepipes to monitor the change in height differences of large structures. The height differences of the liquid levels are automatically recorded. The accuracy ranges from 0.1 to 0.01 mm over a few tens of meters depending on the types of instruments. The main factor affecting the survey accuracy is the temperature. To reduce this effect, either the instrument must be installed in a place with minor temperature variations, or the temperature along the pipes must be measured and corrections applied, or a double liquid (e.g., water and mercury) is employed to derive the correction for temperature effect. Liquid of a constant temperature is pumped into the system just before taking the readings for the highest accuracy applications. The instruments with direct measurement of the liquid levels are restricted in the vertical range by the height of the containers. This problem may be overcome if liquid pressures are measured instead of the changes in its levels, where pneumatic pressure cells or pressure transducer cells may be installed.

3. Crack and joint measuring

In both new and existing structures, the development of cracks and the movement of joints are indications of stresses on the structure that are probably abnormal. In some cases, these conditions can be anticipated beforehand, while in others, the situation arises spontaneously. Measurement of these areas is provided through the displacement indicators that can be either installed in predetermined locations to monitor expected cracks, or placed at the location of a known crack or joint as the need arises for its monitoring.

(a) Crack measuring

These instruments are either manual or electrical, and the latter is adaptable to displacement measurement (Morrison 1984; Raphael and Carlson 1965; Seippel 1983;

Sheingold 1980). The monolith joint displacement indicator, relative movement indicator, multi-position strain gauge, dial gauge, "L"-shaped gauge, and scratch gauge are all manual gauges requiring periodic reading to determine the displacements. The Carlson joint meter and the multiple position borehole extensometer are electrical instruments utilizing the change in electric resistance of a stretching wire as a measure of strain.

Reference points can be scratch marks in the concrete, metal pins, or metal plates on opposite sides of a joint or crack. The distance between the scratch marks is measured with a micrometer or dial gauge to determine the movement. Sometimes, three points are used in a triangle to measure both horizontal and vertical movements.

(b) Joint measuring

Movement of one side of a crack or joint in a concrete structure relative to the other side is commonly measured with reference points or crack meters. Grout or plaster patches can be used to evaluate whether or not a movement is occurring. Crack meters are commercially available devices that allow for the measurement of the movements in two directions. A common device consists of two plastic plates: One is opaque containing a grid, and the other is translucent containing a set of crosshairs. The plates are fixed on each side of the crack or joint with the crosshairs set over the center of the grid. Movement is measured by noting the location of the crosshairs with respect to the grid. In addition, a variety of other crack meters, including Carlson and vibrating wire sensors, dial gauges, and mechanics feeler gauges, may be employed to measure the movement of crack or joint.

Figures 18.10 and 18.11 show the configuration of the horizontal and vertical deformations measured by means of joint meter.

Conventional surveys, however, require "line of sight" and do not lend themselves to unattended, continuous field operations. Their instruments are also restricted in range and do not offer connection to an absolute reference frame. All these make it exceedingly difficult to assess how the dam might have moved with respect to surrounding bedrock.

Monitoring the integrity of a hydraulic structure demands very high precision displacement measurements from a robust system with real-time response. Continuous data recorded from the GPS satellites, using ground-based receivers and robust telemetry, are now more and more prevalent for the safe and health monitoring of hydraulic structures (Colesanti et al. 2003; Wang et al. 2004). Attributable to much less complex, robust automation of highly precise GPS analysis is now reasonably routine (Featherstone et al. 1998; Li et al. 1996).

4. Rock slope movement monitoring

Many rock slopes move to some degrees during the course of their service life. Such movement normally indicates that the slope is in a quasi-stable state, but this condition may continue for many years, or even centuries, without the occurrence of failure. However, in other cases, initial minor slope movement may be a precursor for accelerating movement followed by landslide or collapse. Because of the

Fig. 18.10 Horizontal deformation measuring

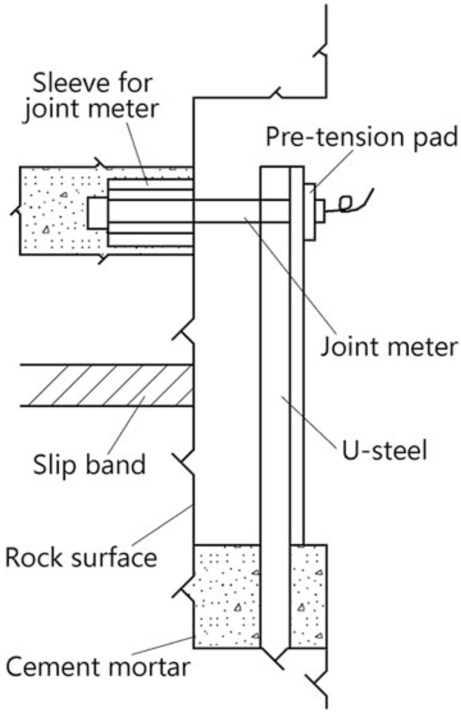
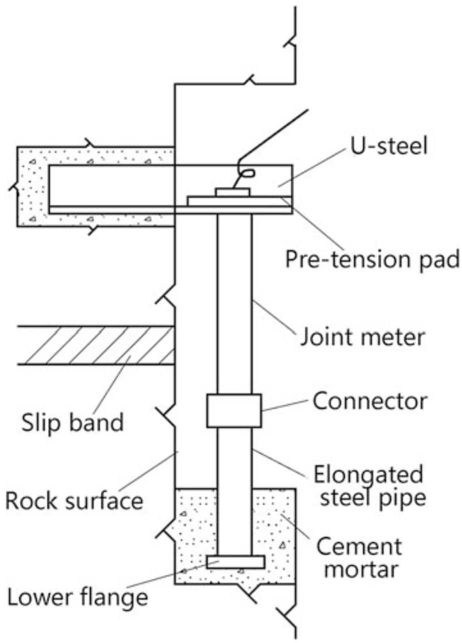


Fig. 18.11 Vertical deformation measuring



strong uncertainty of slope's behavior, movement monitoring programs can be of crucial value in managing slope hazards, which also provide information that is useful for the design of remedial or stabilization works (Dunnicliff 1988; Hanna 1985; Peck 1969).

(a) Surface monitoring

In general, monitoring of a slope surface is likely to be less costly to set up and maintain than subsurface monitoring that will demand drilling holes to install the instruments. However, surface monitoring can only be exercised where the surface movement accurately represents the overall movement of the slope. For example, it would not be appropriate to make surface monitoring where loose rock blocks on the surface were toppling and rotating independently of the main slope movement. Other factors to consider in the selection of a monitoring system include the time available to set up the instruments, the rate of movement, and safe access to the site.

Options for monitoring equipment comprise automation for collecting data at preset intervals on loggers and using telemetry to transmit these results to another location for analysis and plotting. The system can also incorporate alarms that are triggered if preset movement thresholds are exceeded. An important consideration of such automated systems is the cost of installation and maintenance, which normally restrain their use to high hazard locations, and for temporary situations while longer-term stabilization is implemented.

Surface monitoring for slopes comprises monitoring on land surface deformation, surface crack, etc.

Land surface deformation is a routine item in the slope mentoring, which makes use of theodolite, water level gauge, tiltmeters, electronic total station, and the Global Positioning System (GPS). Figure 18.12 is the layout of the land surface deformation monitoring of the Yangjiacao Landslide (Hubei Province, China).

Since tension cracks are an almost universal feature of slope movement, crack width measuring is often a reliable and inexpensive means of movement monitoring. The simplest procedure is to install a pair of pins on either side of the crack and measure the distance between them with a steel tape. If there are two pins on either side of the crack, then the diagonal distance can also be measured to check the transverse (shear) displacement. The maximum practical distance between the pins is ordinarily 2 m. The wire extensometer can be employed to measure the total movement across a series of cracks over a distance as far as 20 m. The measurement station is located on stable ground beyond the cracks, and the cable extends to a pin located on the crest of the slope. The cable is tensioned by the weight, and the movement is measured by the position of the steel block threaded on the cable. The wire extensometer also can incorporate a warning system comprising a second steel block threaded on the cable that is set at a selected distance from a trip switch. If the movement exceeds this preset limit, the trip switch is triggered and an alarm is activated. The main requirements for the crack width monitoring are that the upslope pin or reference point must be on stable ground and that people should access the crest of the slope to make the measurements. This monitoring work could be hazardous where the slope is moving rapidly. Under such circumstances, it

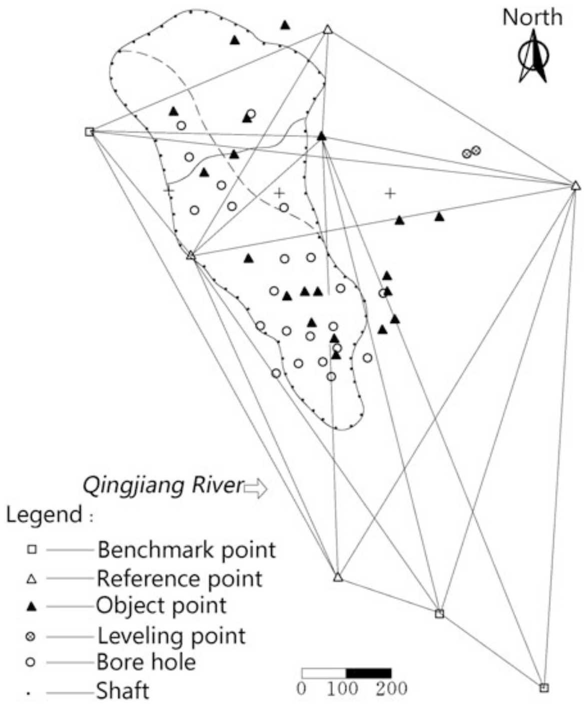


Fig. 18.12 Layout of the surface deformation monitoring—the Yangjiacao Landslide, China

should be replaced by automated system using vibrating wire strain gauges and data loggers, to read and record the measured results from a remote location.

(b) Subsurface monitoring

Figure 18.13 shows the layout of the subsurface monitoring of the Yangjiacao Landslide.

Subsurface monitoring of slope movement is often an important task of the monitoring program in order to provide a more complete image of the slope behavior. The main purpose of the measurement is to locate the slip surface or surfaces and to monitor the rate of movement. Inclinometers are commonly employed in the subsurface monitoring. In some cases, the holes (shafts) are used for monitoring both the movements and water pressures. When the slip surface is revealed by the shaft or adit, joint meter or time-domain reflectometry may be employed to monitor the slip deformation.

(c) Shear deformation measuring

The shear deformation measuring usually makes use of joint meter, as shown in Figs. 18.10 and 18.11.

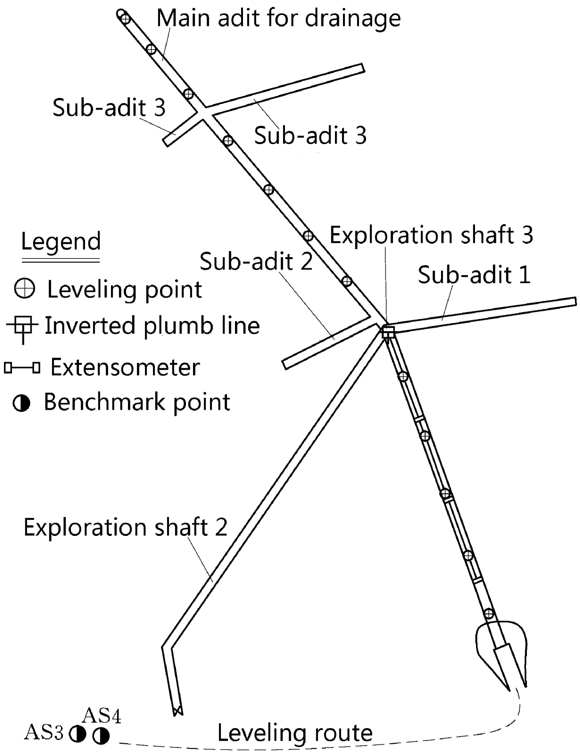


Fig. 18.13 Layout of the subsurface deformation monitoring—the Yangjiacao Landslide, China

(d) Time-domain reflectometry

Time-domain reflectometry is another means of locating a slip surface, which is also able to monitor the rate of movement. This involves grouting into a borehole a coaxial cable comprising inner and outer metallic conductors separated by an insulating material. When a voltage pulse wave is sent down the cable, it will be reflected at any point where there is a change in the distance between the conductors. The reflection takes place due to the change in distance that alters the characteristic impedance of the cable. Movement of a slip surface that gives rise to a crimp or kink in the cable will be sufficient to change the impedance, and in this way, the location of the slip surface can be detected.

The primary advantage of time-domain reflectometry compared to inclinometers is that the cable is inexpensive so that it can be sacrificed in a rapidly moving landslide. In addition, the readings can be made in a few minutes from a remote location either by extending the cable to a safe place off the slide, or by telemetry, to reduce travel time and to directly show the movement without the need to download and plot the results.

18.4.2 Seepage Monitoring

1. Uplift pressure

(a) Uplift pressure under concrete dams

Monitoring of uplift pressure under concrete dams is made to check the validity and accuracy of the design assumptions pertaining to uplift and to provide information for the future designs with regard to the exerting surface and magnitude of uplift pressure.

The measuring points are layout according to the importance, type, size, foundation geology, seepage prevention and draining devices, etc., of the dam concerned. Customarily, several sections perpendicular to the dam axis are set for uplift monitoring, and they are commonly located at the monoliths with maximum dam height, on main river stream, with poor foundation bedrock, and where the stability analysis is conducted in design. In addition, it is advisable that they are at the positions with transverse galleries, to facilitate the measuring operation. The section number is normally 2–7 and mostly 3–4. The point layout in a monitoring section is dependent on the section size and structural and geological features along creep line and within foundation, for the purpose of good representation of uplift distribution and variation. Generally, the following principles are to be observed in the layout:

- To understand the effects of anti-seepage devices, one point on the downstream side of grout curtain, cutoff wall, dental wall, sheet pile, etc., is desirable;
- To understand the effects of seepage pressure relief, one point on the downstream side of draining curtain is desirable;
- One point at the base of structure and at the vicinity of downstream toe is desirable.

Apart from the above transverse sections, additional longitudinal section is also commonly established, which is along the dam axis, at which 1–2 points are located for each dam monolith. An illustrative instrumentation layout for the uplift pressure of a new concrete dam is shown in Fig. 18.14.

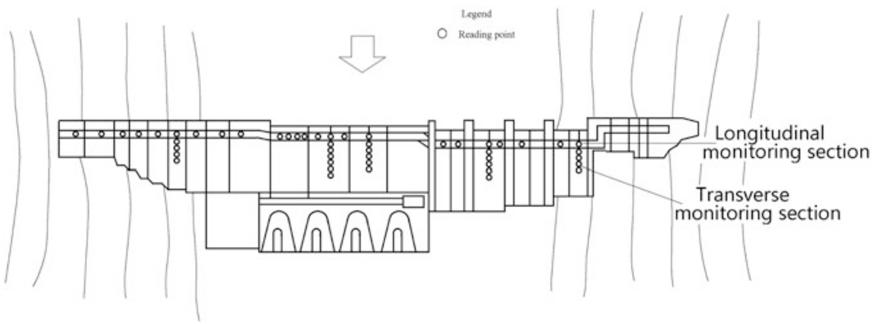


Fig. 18.14 Instrumentation layout of the uplift pressure in a concrete dam

(b) Uplift pressure in embankment dams

Measuring of uplift in embankment dams is accomplished according to the importance and scale of the reservoir, the type and size of the dam, foundation stratum, seepage prevention and relief devices, etc. The monitoring sections are usually the most important and the most representative ones in the design of seepage control, and they are generally spaced 100–200 m apart. For medium-height dam or above, at least 3 monitoring sections are demanded, whereas for small dam, at least 2 sections are required.

The measuring points in each monitoring section are adequate to illustrate the seepage regime such as the phreatic line and the working situation of important components (anti-seepage device, seepage pressure relief, and filter). Figure 18.15 shows the minimum number of monitoring points in different types of embankment dams.

(c) Uplift pressure under embankment dams

The typical monitoring layout for uplift pressure under embankment dams is shown in Fig. 18.16.

(d) Bypass seepage and groundwater at the vicinity of dams

Bypass seepage monitoring is mainly intended to obtain sufficient observed data available for isopotential line plotting, whose basic requirements are as follows:

- At least two rows, each of them consisting of at least 3 points, are laid out along the anticipated streamline;
- At mesa terrace along the main stream, 2–3 rows, each of them consisting of at least 3 points, are laid out perpendicular to the anticipated streamline;
- 1–2 rows of measuring points may be laid out at the permeable stratum with potentially concentrated seepage;
- The points for phreatic line observation should be embedded at least deeper than the water table before the dam impounding.

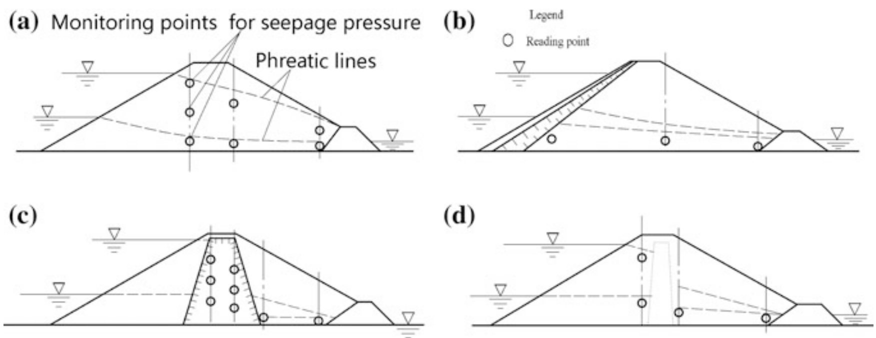


Fig. 18.15 Instrumentation layout of the uplift pressure in embankment dams. **a** Homogenous; **b** sloping core; **c** broad central core; **d** narrow central core

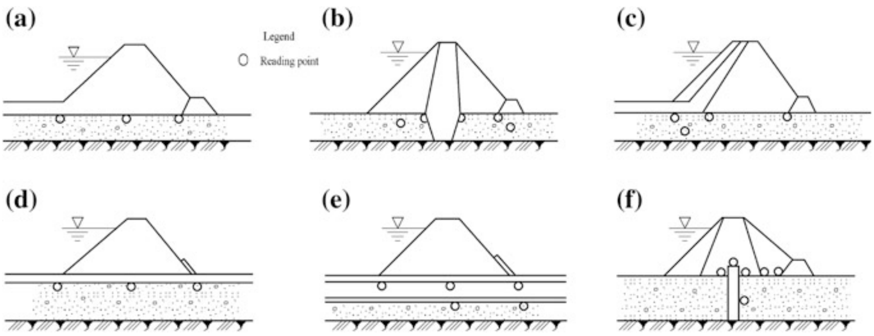


Fig. 18.16 Instrumentation layout of the uplift pressure under embankment dam

Figure 18.17 shows the layout of bypass seepage and groundwater monitoring.

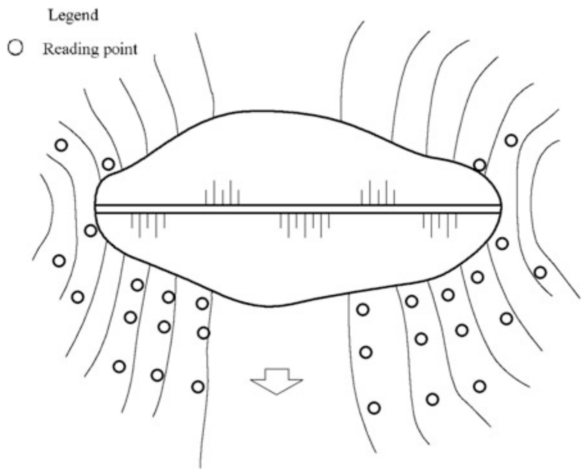
(e) Seepage pressure in rock slopes

The importance of seepage pressure to the slope stability has been well emphasized (Vide Chap. 14). If a reliable design concerning slope stability is guaranteed, it is essential that seepage pressure within the slope should be well monitored. It is most conveniently carried out by the observation of groundwater table directly in borehole, or of seepage pressure using piezometers that are devices sealed within the ground, generally in boreholes, which respond only to seepage pressure in the immediate vicinity.

(f) Instruments for uplift (seepage) pressure measurement

Being simple, reliable, inexpensive, and easy to operate, open standpipe piezometers are standard against which all other piezometers are calibrated. Open

Fig. 18.17 Layout for the bypass seepage and groundwater monitoring in an embankment dam



standpipe piezometers are also known as Casagrande-type piezometers and, in concrete dams, as pore pressure cells. They are observation wells with subsurface seals that isolate the strata to be measured. The seals are usually made of bentonite clay or cement grout, and care must be taken during the installation to ensure a good seal. Riser pipe joints should be watertight to prevent the leakage into or out of the pipe, which could deviate the water level in the pipe.

A common version of the open standpipe piezometer is a well point, which is a prefabricated screened section and riser pipe that is pushed into place. If the screened section is not adequately sealed, it will perform like an observation well rather than a piezometer.

The sensing zone (screened length or porous tip) of observation wells and open standpipe piezometers is susceptible to clogging, which can give rise to time lag or result in failure of the instrument. This can be diminished by a properly designed filter pack that meets filter criteria with the surrounding soil and properly sized perforations that are compatible with the filter pack.

The other evolved types of the piezometers available from various manufacturers are that of closed standpipe, twin-tube hydraulic, pneumatic, vibrating wire, bonded resistance strain gauge, hydraulic pore water pressure cell, etc.

2. Seepage discharge

The amount of seepage or leakage is directly proportional to the material permeability and seepage pressure. Periodic monitoring of the outflow from foundation drains, joint drains, and face drains serves as an indication of the adequacy with respect to foundation grout curtain and good functioning with regard to drains and reveals when and where remedial measures could be undertaken. Observations of leakage flow from contraction joints, lift joints, and cracks provide a means for judging the quality of construction, as well as for disclosing the necessity for remedial actions to preserve the integrity of the structure.

The main drainage sump may be utilized as a collecting and gauging point for all flows. Two types of gauges, namely V-notch weirs and critical depth meters, are available for the discharge measuring.

(a) Layout

For embankment dams, the commonly used configuration is to collect the seeping water from dam body or foundation into downstream gutter, and the measuring operation is undertaken at the exit of the gutter. For concrete dams, both the total discharge collection, and, the measuring operation, may be made at the sump in dam body.

(b) Instruments

Seepage and leakage are commonly measured with calibrated containers and weirs. Other flow measuring devices such as flowmeters also may be appropriate under special circumstances.

- Calibrated containers. They are preferable for the discharge smaller than 1 L/s, or where it is not permissible to use devices of long-term seepage collection and releasing. The collection time for seeping water into the container should be not shorter than 10 s;
- Weirs. They are applicable for the discharge of 1–300 L/s. Thin-plate weirs, such as rectangular weir, Cipolletti trapezoidal weir, and 90° V-notch weir, may be installed at the straight section of a gutter for the purpose of measuring, as is illustrated in Chap. 16 of this book.
- Flowmeters. They are suitable for measuring larger flow discharge.

18.4.3 Strain/Stress and Temperature Monitoring

A variety of mechanical and electrical strain gauges may be employed to measure the strains in concrete dams. Some of the instruments are designed to be embedded in the dam during the construction, and the others are surface-mounted following the construction. Strain gauges are often installed in groups so that the three-dimensional state of the strain can be evaluated.

Stress in concrete structures can be directly measured with total pressure cells or Carlson-type cells designed to have stiffness similar to the concrete. It also can be measured indirectly by overcoring.

For indirect stress measured via strain, the Young's modulus, creep coefficient, and Poisson's ratio of the concrete should be determined by the laboratory tests using concrete cylindrical samples from prototype dams.

In mass concrete, temperature variation is the primary cause of volumetric change and stress. In order to determine the effect of temperature on the stress and volumetric change, temperatures should be measured at a number of points within the structure, as well as at the boundaries. The resistance thermometer and thermocouple are normally employed to measure the temperature in concrete (Baker et al. 1953). Thermocouples are preferable for measuring temperature under certain conditions and at several locations. However, resistance thermometers are advantageous over thermocouples because they are more reliable, highly precise, and less complicated in measuring operation.

1. Strain/stress monitoring

The general requirements with regard to monitoring monoliths, sections, levels, and points should be met as follows:

- Be enable to detect the maximum stress (quantity and direction);
- Be convenient for the validation among observation, design, and experiment;
- Be conforming to the other monitoring facilities for comprehensive analyses of the structure;

- Be allowable of certain repeated installations of monitoring points to secure the reliable monitoring for important positions;
- Be facilitated for the installation construction and monitoring operation.

(a) Gravity dams

- Monitoring monoliths. Usually, at least one observation monolith is, respectively, selected from non-overflow and overflow dam monoliths, and additional monoliths may be desirable for important dams on complicated foundation;
- Observation sections. 1–2 monitoring sections may be set for each observed monolith;
- Observed levels. Apart from the level near foundation (lower than 5 m above base), several observation levels may be selected taking into account dam height and structure feature;
- Reading points. 3–5 strain meter groups or stress meters are commonly laid out for each observation level. The minimum distance of the points from dam surface is 3 m. Two additional points may be positioned at a distance of 1.5–2.0 m from the longitudinal joints. Additional points also may be laid out near the dam surface for measuring the surface stress, if necessary.

As the advancement in the understanding of stress state, in recent years, the stress monitoring for concrete gravity dams has been greatly simplified, and some low-gravity dams even neglect the stress instrumentation totally.

(b) Arch dams

- Observation sections. The crown cantilever section, and the abutment cantilever sections, may be selected for monitoring. For important dam, radial cantilever sections located at 1/4 crest length from the central line may be additionally selected for monitoring;
- Observed levels. Apart from the level near foundation (lower than 5 m above base), several levels may be observed taking into account dam height and structure feature;
- Reading points. Two points from up- and downstream surface in a depth around 1 m should be monitored, with additional point at the center. Denser points are desirable for thick dams or where the temperature monitoring is demanded.

(c) Instruments for concrete dams

Single strain meters, strain meter groups, or stress meters are prevalent in the strain/stress monitoring of concrete dams. To obtain both the quantity and direction of strain/stress, strain meters are often integrated into groups of multi-directional meters as “strain meter spider” (Carlson 1975) as shown in Fig. 18.18. For the purpose of measuring independently the volumetric effects due to temperature and moisture changes and chemical actions within a large structure, “no-stress” strain

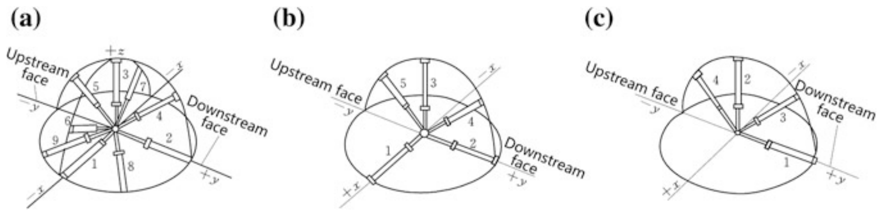


Fig. 18.18 Schematic of strain meter groups. **a** 9 directional meters; **b** 5 directional meters; **c** 4 directional meters

meters are installed in conjunction with strain meter groups. This can be accomplished by embedding an ordinary strain meter in typical mass concrete which is isolated from the deformations due to loading, but is responsive to the temperature, moisture, and growth volume changes in the structure.

The installation of multi-directional meters is usually carried out under the following circumstances:

- Gravity dams. 5 directional meters are commonly used, of which 4 strain meters are in the sectional plane, while the rest one is perpendicular to the section;
- Arch dams. 9 directional meters or 5 directional meters are commonly used, of which the latter has 4 meters in the sectional plane, while the rest one is perpendicular to the section;
- Near the dam base. Where the compression stress is the maximum, single stress meter may be installed;
- Boundary groups. They consist of several meters (from three to six) and may be placed at a depth of 0.1–1 m from a surface of structure, with each meter arranged in a plane parallel to the surface concerned;
- At the toe/heel of dam/foundation contact face, single strain meter, single joint meter, and reinforced concrete meter (R-C meter) may be installed, to check the real stress state in these areas.

(d) Embankment dams

Normally, 1–2 transverse sections may be selected for monitoring, and on each of them, 2–3 levels with reading points may be arranged. The points may be within anti-seepage device, downstream shell, contact face of anti-seepage device/dam shell, etc., as illustrated in Fig. 18.19.

Earth pressure cells are commonly employed in the stress monitoring for embankment dams. To obtain both the quantity and direction of stress, earth pressure cells are often integrated into group of multi-directional meters (2–3 for each group).

(e) Concrete faced rockfill dams

Typical face slab slices, such as the slab slices at portions of the mainstream, abutments, and in between (1/4 of crest length), are selected for monitoring, as

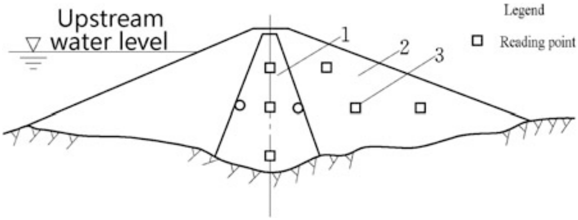


Fig. 18.19 Monitoring layout for the stress in an embankment dam. *1* core; *2* dam shell

shown in Fig. 18.20. The highest slab slice shall replace the middle one where they are not coincidence. The first and second abutment slab slices in close vicinity to the banks are not appropriate to be monitored. The strain sensors and reinforcement meters are laid out at portions lower than the normal storage level, which may be sparse at the upper portion and denser at the lower portion.

(f) Slopes

① Prestressed wire strand anchor cables. Dynamometers are customarily set to monitor the prestress. Conventional dynamometers for prestressed anchor are pressure transducer, vibrating wire pressure cell, strain gauge pressure cell, and liquid pressure cell. The Chinese design codes stipulate that the number of long-term observed anchors should be no less than 5 % than that of the total anchors installed.

② Stabilization piles and shear keys. Pressure cells are normally applied, which have various types according to their working principles. In the type selection of pressure cells, the major factors to be taken into account are the measuring range and precision, earthquake and impact resistance, tightness, etc.

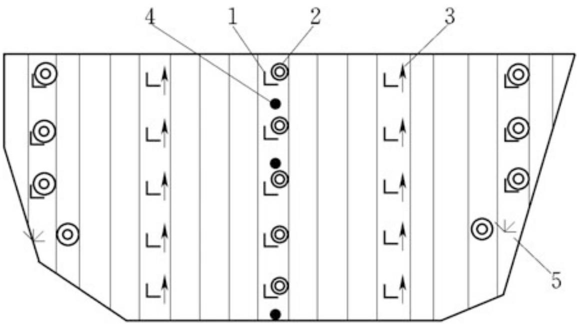


Fig. 18.20 Stress monitoring layout for a CFRD. *1* 2 directional meters; *2* “no-stress” strain meter; *3* reinforcement meter; *4* temperature sensor; *5* 4 directional meters

(g) Others

Reinforcement meters are required in the area of galleries, penstocks, and piers, for monitoring the stress in steel bars.

Earth pressures within fill against concrete structures are commonly measured with earth pressure cells. These are also known as total pressure cell consisting of two flexible diaphragms sealed around the periphery and with a fluid in the annular space between the diaphragms. Pressure is determined by measuring the incremental fluid pressure behind the diaphragm with pneumatic or vibrating wire sensors. Earth pressure cells should have similar stiffness as the surrounding soil to avoid error due to arching. Soil pressures against structures can also be measured with a Carlson-type cell, which consists of a chamber with a diaphragm on the end, and the deflection of diaphragm is measured by a Carlson-type transducer and converted to pressure.

2. Temperature monitoring

Temperature monitoring of dam and foundation is frequently demanded for reducing data from instruments, raising precision, or interpreting results. For example, movements and leakage changes of concrete dams are commonly related to the changes in temperature. Temperature is also commonly measured in the concrete dams under construction to evaluate the placement design (e.g., mix, rate, block, and lift size), to schedule grouting of construction joints, and to provide thermal loads. Temperature measured from seeping water may indicate the sources of seepage.

One of the common layouts would be to place thermometers spaced about every 8–15 m apart along an elevation in the monolith. For a small structure, a denser spacing may be desirable. A few thermometers should be placed near the structure faces to evaluate the daily and weekly temperature fluctuations. Another layout commonly exercised is to configure observation lines of thermometers parallel and transverse to the structure axis and in a vertical direction as well. The space of the reading points may be very small near the exposed surfaces and quite large in the interior regime with more uniform temperature. Near or crossing construction joints, it is conventional to position reading points close to the joint surface for temperature monitoring.

(a) Internal temperature

The monoliths, sections, and levels, for internal temperature monitoring, are commonly identical to that for strain/stress monitoring.

The temperature monitoring points for a gravity dam on one level are laid out at the space of 8–15 m, which is subject to be more concentrated around holes and near boundaries and less concentrated in the central portion of dam. Monitoring sections perpendicular to the penstocks are demanded. To facilitate the construction, the monitoring points are ordinarily laid out on the surface of lift joint. Figure 18.21 is a schematic layout of the temperature monitoring for a gravity dam.

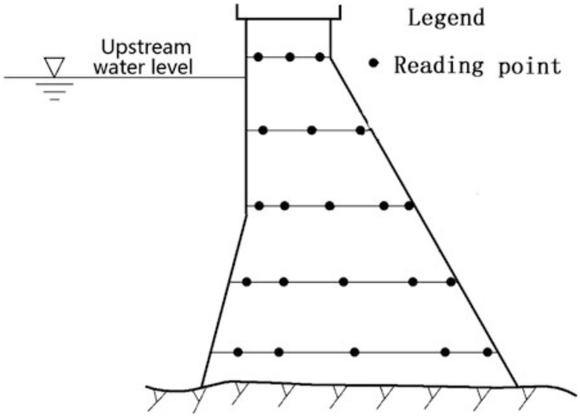


Fig. 18.21 Layout of the temperature monitoring for a gravity dam

For monitoring the crown cantilever section of an arch dam, 3–7 monitoring levels are normally laid out, and on each level, at least 3 monitoring points are set. The portions near crest and abutment are subject to more concentrated points. Figure 18.22 is a schematic layout of the temperature monitoring for an arch dam.

(b) Boundary temperature

Points for boundary temperature monitoring may be laid out at a distance of 5–10 cm from upstream surface and is commonly 1/10 ~ 1/15 of the dam height but may be doubled below the inactive storage level. On the downstream surface, temperature sensors are also laid out to evaluate the influence of sunshine.

(c) Foundation temperature

Temperature transducers should be laid out along the contact face of dam/foundation on the monitoring section. To understand the heat dispersion procedure, at the mid of contact face, a vertical monitoring line may be installed using several sensors, as shown in Fig. 18.23.

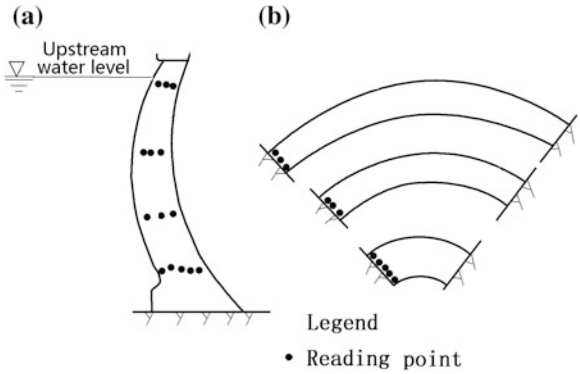


Fig. 18.22 Layout of the temperature monitoring for an arch dam. **a** Cantilever section; **b** arch section

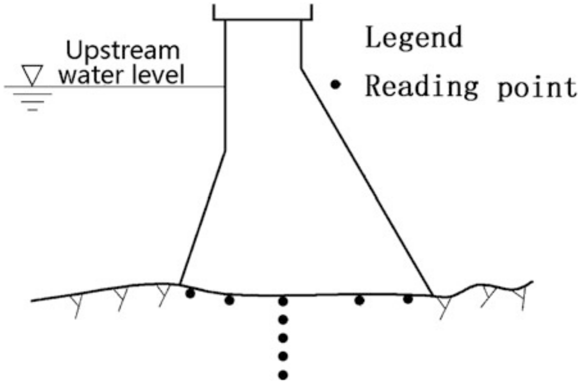


Fig. 18.23 Layout of the foundation temperature monitoring for a gravity dam

3. Internal monitoring system

For a medium-to-large hydraulic project, the internal monitoring system should be established which covers strain/stress, temperature, crack/joint movement, seepage/leakage, uplift, etc., for the purpose of overall safety surveillance and management. Figure 18.24 schematically shows such a system.

The process of breaking down the physical phenomena into their fundamental quantities of length, time, mass, and temperature enables engineer to better define the desirable types of measuring components (transducers) for the instrumentation system. In other words, the engineer should explicitly define and examine the non-electrical quantities that must be converted into usable electrical signals. Ultimately, these electrical signals must reliably and accurately represent the values of physical quantities being monitored.

A measurement is made by an element or transducer which produces a voltage, current, or frequency that represents the quantity or property being measured. This

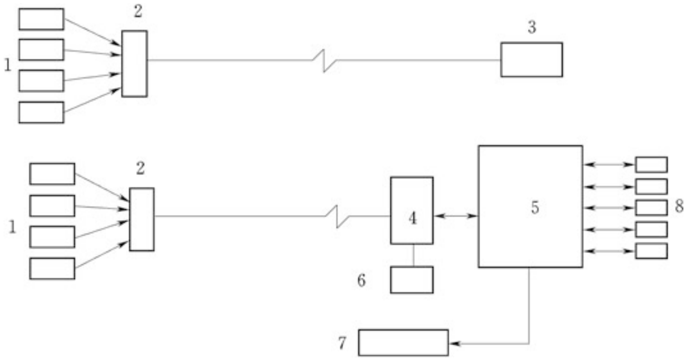


Fig. 18.24 Schematic of internal monitoring system. *1* transducer; *2* remote-controlled collective line box; *3* receiving instrument; *4* monitoring instrument; *5* computer; *6*,*7*,*8* peripheral equipments (e.g., power and power conditioning equipments)

is accomplished by varying inductance, light, capacitance, or resistance. The output must be digitized for the use by the computer system. It may be digitized at the source before transmission or later in the instrumentation system. Care must be exercised in the transmission of the signals if they are properly digitized at the system. Transmission of digital signals may give rise to high noise immunity and should be checked to assure that what is transmitted is what is received.

Distribution and acquisition of electronic signals from numerous sensors and instruments demand competent system architectures. A centralized acquisition system is generally recommended if the application is small, with signal sources close together. Larger application, typical of large dams, where instruments are geographically dispersed, may demand remote processing or distributed intelligence architecture of the data acquisition system. Wiring costs can be a large factor in determining the proper type of the system.

Most data acquisition system (DAS) manufacturers offer signal conditioning with multi-channel analog-to-digital (A/D) converters. Depending upon the type of input device, signal conditioning consists of amplification, bridge completion networks, thermocouple compensation, excitation voltage and current supplies, as well as filtering. The higher accuracy and precision are required, the higher of the system expenditure will be accompanied by the cost rise in maintenance and calibration.

18.4.4 Automated Measurement Techniques and Data Acquisition for Dams

The dam owner is responsible for the collection, storage, and presentation of all data associated with its operation and maintenance. There are two types of such data:

- Static data that do not change with time. They will normally be stored in the data books, dam safety reviews, and reports. Static data usually encompass all design and construction investigations. As much of the static data will never be changed, it may be reduced and stored on microfilm or electronic storage medium. Sufficient, easily accessible information should be kept on hand in data books to provide information for any situations which could arise.
- Dynamic data that change with time. They comprise data derived from dam safety inspection, monitoring, and operation and maintenance activities, which are accumulated in the corresponding reports. Most of the dynamic data are suitable for computer storage and presentation, particularly those arising from monitoring.

For data collection and management purposes, it should be well aware that

- The power and limitation of computer storage and retrieval systems (e.g., ease of access for the retrieval of information)
- Issues associated with compatibility of computer systems.

It should be made certain that the system used to collect and process the data has facilities to detect the occurrence of “obviously different” data, which can be resulted from the following:

- Data recording and transferring errors;
- Instrumentation malfunction;
- Abnormal behavior of the dam.

Instrumentation automation is increasingly becoming a valuable means of collecting monitored data (Choquet et al. 1998; Huang et al. 2004; ICOLD 2000b). Automation permits a greater volume of data to be collected in a given period of time. Where an instrumentation reading staff may take 4–6 h to read a set of plumb lines, the same readings can be taken in less than 10 min when collected by automated plumb line equipment. Therefore, it is now more economical, in terms of overall cost, to automate the reading of certain types of instrumentation than to continue reading them manually.

Automated data acquisition systems (ADAS) have evolved significantly since the 1990s and are currently installed on hundreds of dams in China. Figure 18.25 shows a schematic flowchart of the ADAS for dam safety monitoring, which can handle various tasks ranging from simple use of a data logger to collect data from a few instruments, to computer-based system that collect, deduce, present, and interpret data of a network with hundreds of different instruments. Most types of water level, water pressure, seepage, leakage, stress, strain, and temperature can be readily monitored and analyzed. However, some types of instrumentation such as probe inclinometers cannot be automated insofar.

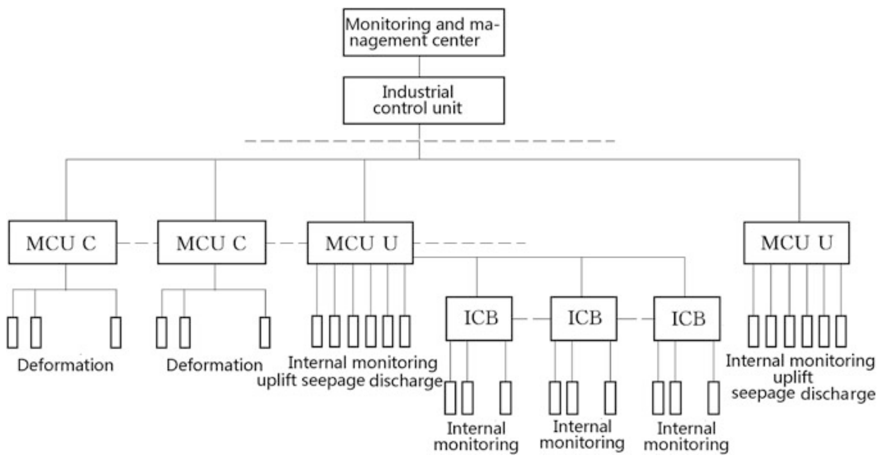


Fig. 18.25 Schematic flowchart of ADAS for dam safety monitoring. *MCU C* measure and control unit for capacitor equipments; *MCU U* measure and control unit for usual equipments; *ICB* intelligent conversion box

Advantages of ADAS are rapid notification of potentially hazardous performance and increased frequency of measurements being taken on demand. Disadvantages of ADAS are high initial expenditure and complex equipment, and the visual observations normally undertaken during the routine manual data collection will not be made.

18.5 Remedial Action

18.5.1 The Need for Remedial Action

There are a number of situations that may demand remedial actions for a dam, which can vary from a minor deficiency in the dam, to a moderate incident or major accident, or even dam failure.

Deficiencies threaten the safety of the dam and may be detected by surveillance, which are probably resulted from inappropriateness or deficiency in design or construction, changes to design criteria, time-based deterioration or breakdown of materials, maintenance-related problems, deficiencies in the dam operation and safety management program, and environmental impacts (e.g., landslide, erosion, earthquake, aging).

Incident is an event, which may evolve to a very serious situation endangering the dam. Examples of incidents are rapid change in seepage, overtopping of earth embankment, excessive beaching, excessive embankment erosion, spillway erosion or blockage, excessive cracking or deflection in the concrete dam and spillway, sliding or settlement of the dam, and malfunction of gates.

The failure of the dam means the physical collapse of whole or a part of the dam, or the uncontrolled release of any of its contents. Scenarios of failure include overtopping of embankment dams, collapse or erosion of spillways, internal erosion or piping through earth embankments or abutments, failure of release conduits, and overturning of concrete dams.

18.5.2 Requirements for Remedial Action

Remedial action is required in response to any deficiency, incident, or dam failure. The type of remedial actions necessitated and its urgency is determined by the nature of the event, which may be distinguished as follows:

- Preventative measures to stop situations worsening;
- Short-term actions such as activation of emergency action plans (EAP) inclusive evacuations and operation of warning systems, modification of operating procedures inclusive lowering of reservoir level and strengthening surveillance;
- Long-term actions such as structural changes and reinforcements, changes to operating procedures, and decommissioning.

A remedial action review should be undertaken which methodically evaluates the various options, covering the determination of the failure risk, the preparation of a failure impact assessment to determine the current population at risk and a consequence assessment to determine the other consequences, the development of possible solutions with respect to the benefits and implementation costs of each solution, and the justification for the adoption of the preferred remedial action.

18.5.3 Emergency Action Plans (EAP) for Dams

The standards and specifications used for design, construction, operation, maintenance, and inspection are intended to minimize the risk of dam failure (ICOLD 1987, 1995). However, as unusual circumstances may appear, the dam owner needs to identify conditions which could lead to failure situations and which may require emergency plans.

Emergency plan takes place at two levels: to prescribe the activities at the dam—known as the emergency action plan (EAP), which is prepared and operated by the dam owner, and to prescribe the activities below or beyond the dam—known as the counter disaster plan (CDP), which is prepared and operated by the appropriate local or central government departments with significant input from the dam owner (Cox 2007).

An EAP or CDP should indicate who is responsible for undertaking particular actions under emergency circumstances and must be tailor worked out to the conditions for each dam.

In the following text, the flood defense for embankment dams or river levees intended to provide temporary flood mitigation until permanent mitigation is taken as an example for a brief illustration of EAP.

During flood seasons, the embankment shall be patrolled continuously, to make it be certain that

- There are no indications of developing slides or sloughs;
- Wave wash or scouring action is not taking place;
- Overtopping action does not give rise to;
- No other conditions exist which might endanger the structure.

The problems that may arise during a flood season are varied and innumerable. It is impossible to enumerate all the categories of problems that field personnel must handle. The most valuable asset of field personnel under emergency conditions is their common sense and sensitivity to human relations, by which many problems can be solved quickly and efficiently. Physical problems with the embankment and related infrastructure can be identified early if a patrol team equipped with a good communication system has been well organized.

1. Overtopping

Overtopping of an embankment manifests after the water level exceeding its crest elevation, which will generally be resulted from

- Unusual hydrologic phenomena such as heavy rainfall that lead to a much higher water level than anticipated;
- Insufficient time in which to complete the embankment; or
- Unexpected settlement or failure of the embankment.

Overtopping should be prevented at any cost. Generally, emergency barriers are built about 0.5 m above the predicted water level. On an existing or completed barrier, predictions concerning water levels or settlements of the barrier will call for some form of capping to raise the barrier. Capping should be done with earth fill or sandbag.

2. Seepage

Seepage through the embankment is generally not a problem unless

- The downstream embankment slope becomes saturated over a large area;
- Seeping water is washing out solid material from the embankment.

Seepage is almost impossible to eliminate totally, and any attempt to do so may create a much more severe condition. If seepage gives rise to saturation and sloughing on the downstream slope, the embankment section should be flattened (Fig. 18.26). Material for flattening should be at least as pervious as the existing embankment shell, to avoid a pressure buildup.

3. Sand boil

A sand boil is the rupture of the top foundation stratum due to uplift pressure in the substratum. Even when an embankment is properly constructed and of such mass to resist the destructive action of flood water, water still may seep through a sand or gravel stratum under the embankment and break through the ground surface in the form of bubbling springs.

All sand boils should be watched closely. A sand boil that discharges clear water in a steady flow normally does not endanger the embankment. However, if the flow of water increases and the sand boil begins to discharge material, remedial actions should be undertaken immediately. One of the prevalent remedial actions for

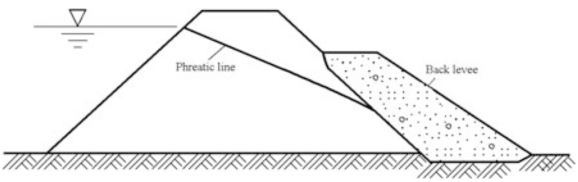


Fig. 18.26 Flattening of an embankment section

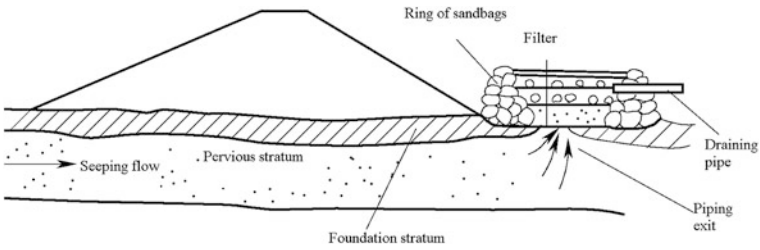


Fig. 18.27 Ring of sandbags around a sand boil

treating sand boils is to construct a ring of sandbags around the boil (Fig. 18.27), in this manner to build up a counter head of water within the ring sufficient to check the velocity of flow, thereby to prevent sand and silt from further moving. In general, the following principles may be observed in the construction of sandbag ring:

- The base width of the sandbag section on each side of the ring should be 1–0.5 times greater than the contemplated height;
- Weak soils near the boil should be enclosed within the ring, thereby preventing a breakthrough later; and
- The ring should be sufficient in size to permit sacking operations.

The height of the ring should only be that necessary to just stop the movement of soil in the water, and be not so high as to completely eliminate seepage—otherwise beyond the ring a new boil may erupt suddenly. Where many boils are found to exist in a given area, a ring levee of sandbags should be constructed around the entire area, and if necessary, water should be pumped into the area to provide sufficient water weight to counterbalance the upward pressure.

4. Erosion

Erosion of the embankment slope is one of the most severe problems that will be encountered during a flood fight. Emergency operations to control such erosion include the use of polyethylene sheeting and sandbag anchors.

18.6 Aging of Hydraulic Structures

There are already more than 86,000 reservoirs and 200,000 km river dikes in China, whose aging or deterioration is of great concern to personnel involved in their design, construction, and operation. These concerns extend throughout their entire life cycle until safe abandonment or decommission (ICOLD 1994).

The damages to hydraulic structures (incident and accident) may be generally classified into two categories: damages related to unusual events (flood and earthquake) and damages related to the environmental factors during service. The latter are regarded as aging after at least five years of operation. The aging of dams, as defined herein, is due to time-related changes in the properties of the materials of which the structure and its foundation are composed.

Good design may mitigate the effects of aging. Vigilance during construction may correct conditions contributing to aging. Surveillance during operation may identify aging processes which could have impact on dam safety.

For an understanding of aging, it is necessary to establish the relationship between causes and effects leading to the degradation in structural properties of the dam and/or foundation. These processes are referred to as "scenarios." The causes originate actions on the dam and/or foundation and may affect the material's properties. The consequences of deterioration may sometimes only be observed after several years of operation.

18.6.1 Nondestructive Examination

Each dam site and its environment are unique, with different characteristics governing its performance; therefore, it is important to establish and to maintain a strong database to assess the impact of aging scenarios on dam safety.

An up-to-date knowledge concerning the dam status is documented so that the anomalous behavior is detected in time to allow for appropriate intervention correcting the situation and avoid severe consequences. Indirect evaluation should be accomplished by monitoring the effects and consequences of aging, whereas direct evaluation of aging is realized by inspecting and testing data in structural properties.

Laboratory and in situ experiments, as an important direct evaluation method to obtain the knowledge concerning the dam status for the study of the anomalous behavior due to aging, will be discussed in the text hereinafter.

By laboratory tests, the physical and mechanical properties of rock, soil, and concrete may be studied on the samples extracted from prototype dams. These tests usually belong to destructive examination and have scale effects.

In situ tests may be destructive or nondestructive. Destructive examination comprises a large category including

- Hydraulic tests such as packer or pumping tests for porosity and permeability;
- Borehole sampling, borehole camera, and TV inspection;
- Leakage detection, chemical and physical test of water (reservoir and leakage), and other materials.

Nondestructive examination (NDE), also known as nondestructive testing (NDT), is intended to examine materials for flaws without harming the tested object (Bray and McBride 1992; Prassianakis and Grum 2011). As a widely applied industrial test technique, NDT provides an effective means for detection, while it protects the object's usability. NDT uses several methods including visual inspection, seismic exploration, and radiography.

To successfully apply NDT, a clear understanding of its limitations and manipulations of recorded data is essential, since experience has taught us that dependence on any one particular technique often leads to unacceptable mistakes.

1. Acoustic method

Sudden application of a point force to the surface of a homogeneous elastic body generates body waves and surface waves. The body waves are distinguished as compressive (*P*) and shear (*S*). There are two types of shear waves as vertically polarized (SV) and horizontally polarized (SH). In a homogeneous isotropic body, the velocities of these two waves are identical. In concrete, velocity V_p of *P*-waves is related to the Young's modulus, density, and Poisson's ratio, while velocity V_s of *S*-waves is related to the shear modulus and density. Where there are material boundaries or flaws affecting the wave transmission from the source, the waveforms are converted: Along boundaries, they are called "love" waves and those responding to the boreholes are called "tube" waves.

If the point force source is within the medium, the waves generated will depend upon the characteristic of the motion at the source. A pure spherical expanding source will generate *P*-waves, but any non-spherically symmetric disturbance will generate both *P* and *S*-waves. In reality, the necessity for finite source dimensions complicates the type of waves produced. Furthermore, inhomogeneities give rise to wave conversions, whether the source is at the surface or at the depth.

Acoustic method is based on the generation of elastic waves in the structure and the measurement of the time taken by the waves to travel from the source, through the materials to a series of geophones, which are usually laid out spaced one to ten meters apart along a straight line from the source (Li et al. 1998). Seismic technique involves frequencies in the range of 100–500 Hz generated by explosives and other energy sources (in rock mass or dam concrete), higher frequencies in the range of 10–30 kHz are used in "acoustic" technique with piezoelectric sources, whereas ultrasonic technique employs a frequency range in 0.5–25 MHz. With lower frequencies, the interaction effects of the waves with internal flaws would be so small that detection accuracy is poor. However, with lower frequencies, the damp would be lower which enables the waves to penetrate deeper into the structures.

Seismic tomography technique (ST) enables to obtain the wave distribution at the dam section, and in this way, the dynamic elastic properties and integrity of the dam may be evaluated. ST was used in Italy for the exploration of 27 concrete dams and 10 embankment dams, as early as the year of the 1990s.

2. Electromagnetic method

Electromagnetic method employs higher-frequency waves for detection, including electromagnetic induction and ground-penetrating radar (GPR), and the latter will be illustrated herein.

GPR is a prevalent geophysical method (Bertacchi et al. 1985; Butler et al. 1991; Daniels et al. 1997; McDowell et al. 2002; Xiao 2000) that generates and receives radar pulses to image the subsurface. This nondestructive method employs electromagnetic radiation in the microwave band (UHF/VHF frequencies) of the radio spectrum and detects the reflected signals from subsurface structures. GPR has been successfully exercised in a very wide range of tasks from mapping geological structure, to identifying defects (voids and cracks) in concrete.

High-frequency (usually polarized) radio waves transmit into the ground by antenna (T). When the waves hit a buried object or a boundary with different dielectric constants, the receiving antenna (R) records the variation in the reflected return signal (Fig. 18.28). The principles involved are similar to reflection seismology, except that electromagnetic energy is used instead of acoustic energy, and reflections appear at boundaries with different dielectric constants instead of acoustic impedances (Daniels 2004; Goodman 1994; Guo 2000; Hanssen 2001; Kampes 2006).

Travel time of an electromagnetic pulse may be calculated by

$$t = \sqrt{4z^2 + x^2}/v \tag{18.5}$$

where t = time for the travel, ns (1 ns = 10^{-9} s), and v = electromagnetic pulse velocity in the ground medium, m/ns.

The survey line spacing varies according to target type, size, and depth. It is typically ranged in 0.25–1.0 m as a standard for civil engineering targets, but may be increased up to 10–20 m for mapping geological layers. The measuring interval between individual GPR waveforms is normally set between 1 cm and 1 m, depending on the size of the target and the resolution required. As an empirical rule, to define discrete targets, it is necessary to record a minimum of 2.5 waveform traces in the distance that the dimension of the target occupies. Additional survey lines, or a multi-line grid, may be demanded to detect small or multiple targets, or in cases where a high degree of spatial accuracy in data analysis is necessary. Taking the ordinate as t (ns) and the abscissa as distance x(m), a GPR log obtained is shown in Fig. 18.29.

Cross-borehole GPR has been developed within the field of hydrogeophysics to be a valuable means for assessing the presence and amount of soil water.

The detected depth of GPR is restrained by the electrical conductivity of the ground, the transmitted center frequency, and the radiated power. As conductivity increases, the penetration depth decreases. This is because the electromagnetic

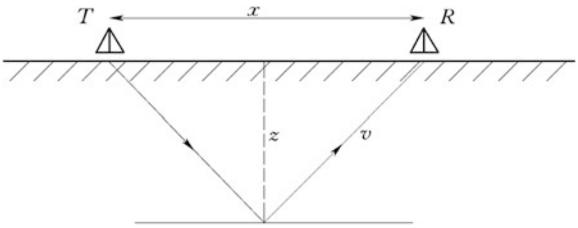


Fig. 18.28 Schematic diagram of ground-penetrating radar

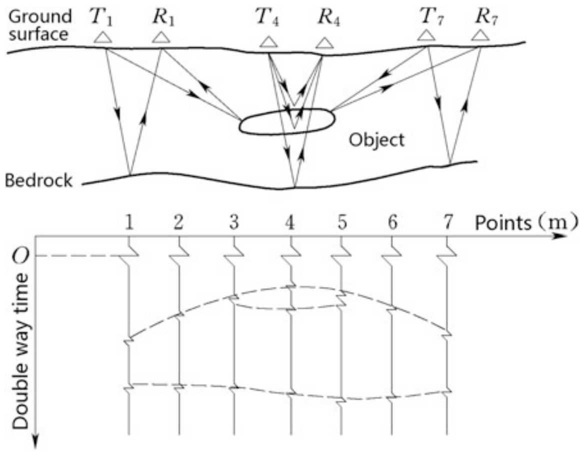


Fig. 18.29 A ground-penetrating radar log

energy will be more quickly dissipated into heat, resulting in a loss in signal strength at depth. Therefore, the most significant restraint of GPR is imposed by high-conductivity materials such as clay soils and soils that are salt-contaminated. Good penetration is achieved in dry sandy soils or massive dry materials such as granite, limestone, and concrete where the depth of penetration could be up to 15 m. In moist and/or clay-laden soils and soils with high electrical conductivity, penetration depth is sometimes only a few centimeters. Higher frequencies do not penetrate as far as lower frequencies, but give better resolution. Performance of GPR is also limited by signal scattering in heterogeneous conditions (e.g., rocky soils).

Disadvantages of currently available GPR systems are as follows: Interpretation of radargrams is generally non-intuitive to the novice; considerable expertise is necessary to effectively design, conduct, and interpret GPR surveys; and relatively high energy consumption can be problematic for extensive field surveys. However, recent advances in GPR hardware and software have done much to ameliorate these disadvantages, and further improvement can be expected with the ongoing innovations.

3. Spontaneous potential (SP) method

It is also termed as “self-potential method,” which measures the natural or spontaneous potential difference that exists between the borehole and the surface in the absence of any artificially applied electric current, by an electrode relative to a fixed reference electrode. SP is usually caused by charge separation in clay or other minerals, due to the presence of semipermeable interface impeding the diffusion of ions through the pore space of materials, or by natural flow of a conducting fluid through the materials. SP is often measured down boreholes for formation evaluation in the oil and gas industry, and they also may be undertaken along the earth’s surface for mineral exploration or groundwater investigation.

The measuring network is usually laid out along the flow direction of groundwater, and the probes of Cu–$CuSO_4$ are embedded. The potential difference of every dual points measured enables to draw potential profiles or potential contours. The leakage zone has low potential, while the exit area of seepage flow has higher potential. Accordingly, the incipient fault of leakage may be analyzed concerning the position, embedding depth, development tendency, etc. Quantitatively, the relative abnormal value η of the potential curve may be used in the judgment, which is defined as the ratio of abnormal value to normal value: When η = 1.3–1.5, concentrated leakage has formulated, whereas $\eta > 1.5$ indicates the occurrence of piping.

4. Resistivity method

The method is employed for the study of discontinuities, by utilizing direct currents or low-frequency alternating currents to investigate the electrical properties (resistivity) of subsurface materials (Edwards 1977; Johansson and Dahlin 1996). Resistivity measuring operation uses two current electrodes and two potential electrodes.

Electrical resistivity method is an excellent tool for groundwater exploration. Electric current is injected into ground through two electrodes, and the resultant potential is measured through another pair of electrodes. These electrodes are placed on ground at predefined locations. The data are then interpreted with available software, and the results are in the form of resistivity and the thickness of different layers. In the application to dams, the exploration network may be laid out parallel to the dam axis, and the results may be interpreted in a manner of deep resistivity profiling or resistivity contours.

Figure 18.30 shows the resistivity contours of the Baicaoping Embankment Dam (H = 42 m, China) along the dam axis. Above the altitude of 25 m, the contours are nearly horizontal, which means homogeneity of the core material along the horizontal direction. This had been verified by 5 borehole samplings with $\gamma_d = 1.2-1.4\,\mathrm{t/m^3}$. Vertically, the contours may be divided into three groups:

- Above the altitude of 14 m enclosed by the contour of 50 Ωm, they are on the high side of resistivity, which means that the construction quality is good with dry density $\gamma_d = 1.5-1.6\,\mathrm{t/m^3}$ by borehole samplings;
- From the altitude of 14 m to the dam base limited by 50 Ωm, there are two lower resistivity areas. The right one has the lowest resistivity at a depth of 25 m, which is the dam closure portion with poor construction compaction; the left one is at a depth of 15 m, which is mainly induced by the filling of frozen soil according to the construction log. The boreholes of 4, 7, and 8 all obtained over saturated soil samples;
- The zone under the resistivity 50 Ωm, where the density of contours increases at a faster rate, reflects the characteristics of sand–gravel foundation. The borehole 9 also revealed that the contour line 50 Ωm is the boundary of dam and foundation.

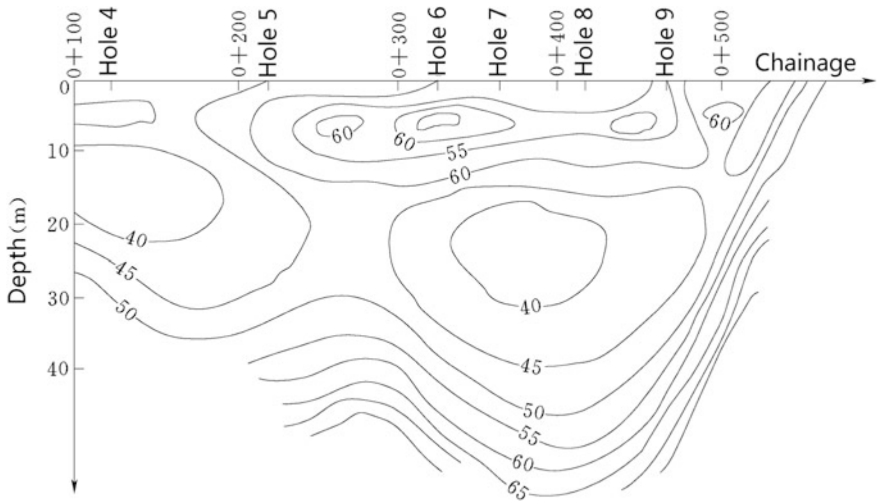


Fig. 18.30 Resistivity contours of the Baicaoping Embankment Dam, China, $H = 42$ m

5. Induced polarization method

Induced polarization (IP) means electrical surveying using geophysical imaging technique to identify the subsurface materials (Edwards 1977). The method is similar to electrical resistivity tomography, in that an electric current is induced into the subsurface through two electrodes, and voltage is monitored through two other electrodes. The method makes use of the capacitive action of the subsurface to locate zones where clay and conductive minerals are disseminated within their hoist rock: Under the similar target body condition, the higher the water content, the slower the decay of the induced polarization field. In the application to dams, a measuring network should be laid out along the dam axis, to detect the distribution of soft stratum within foundation or dam body.

Time-domain IP method measures the voltage decay or chargeability over a specified time interval after the induced voltage is removed. The integrated voltage is employed for the measuring operation.

Frequency-domain IP method uses alternating currents (AC) to induce electric charges in the subsurface, and the apparent resistivity is measured at different AC frequencies.

18.6.2 Aging Diseases of Hydraulic Structures

Aging of hydraulic structures should be examined from two aspects (Combelles 1991; ICOLD 1983, 1994; Jansen 1988; Sinniger et al. 1991):

- As an objective process of exhaustion of given properties over the course of time with respect to a set of factors including aesthetic, functional, physical, technoeconomic, environmental, social;
- As a process of accumulation over the course of time with respect to irreversible physicochemical changes in materials composing the object, which ultimately lead to the deterioration of the object and exhaustion of its given properties.

1. Aging of foundation rock and soil

Three scenarios of foundation rock and soil aging may be summarized in the following (Lin et al. 2009; Unal and Unver 2004).

(a) Loss of strength

It is a phenomenon associated with mechanical failure by shear leading to the progressive deterioration of the foundation. Loss of strength generally takes place in clay soils due to saturation and excessive strain. When clay soils are strained over their peak strength, they manifest a significant loss of strength.

(b) Deformation

When the foundation consists of very weak rocks or deformable soils, they are ordinarily removed or measures are taken to improve the material left in place. Otherwise, differential settlements due to consolidation may appear, resulting in the cracking of the dam body. In areas of dry climates, dry and low-density soils collapse when they become wetted, which also may lead to dam cracking.

(c) Uplift pressure

Uplift pressure builds up where the steady-state seepage is obstructed or where a sudden excess in seepage cannot be accommodated. In rock with erodible or soluble joint infilling under reservoir pressure, water penetrates the joints and progressively builds up the uplift pressure. Water seeping under the dam may transport fine grains which may progressively clog the draining filter or draining holes and build up the pressure at the downstream toe. Water seeping through poorly graded material may move material grains (internal erosion or piping) leading to subsidence or sinkholes.

2. Aging of concrete structure

The durability of concrete depends on the durability of its constituents, cement paste, and aggregates. A concrete with strong paste may not be durable if combined with poor aggregate and vice versa. One of the most important parameters is the "porosity" of the paste, which is a function of the amount of water relative to the cementitious materials. Excessive water can dilute the cement paste leaving a medium more porous, which is more vulnerable to deleterious substances and physical processes (Dolen 2005; Dolen 2006; Durcheva and Puchkova 1995; Tsulukidze 1962). The climate is a significant environmental factor influencing the long-term durability of concrete structures. One of the reasons that the ancient structures have survived is because they were constructed in areas of relatively dry and mild climates.

The four primary scenarios of concrete deterioration in concrete dams may be summarized in the following:

- Sulfate attack (SA). The chemical and physical destruction of the cement paste by aggressive, sulfate-laden waters;
- Alkali–aggregate reaction (AAR). The chemical reaction between alkali compounds in cement with certain amorphous silica-bearing aggregates, responsible for concrete "growth" by expanding silica gel;
- Freezing/thawing deterioration (FT). The physical destruction of primarily cement paste by ice formation within the cement pores;
- Corrosion of reinforcing steel. Primarily related to structures constructed accidentally with insufficient cover. However, when other mechanisms deteriorate surrounding concrete, corrosion may ultimately become the primary means of deterioration.

(a) Sulfate attack

Effects of erosion water environment include internal sulfate attack (SA), acid attack, and chloride (corrosion) attack (Neville 2004). SA is a chemical degradation of cement paste caused by high concentrations of sulfates in soils and groundwater, which is attributable to chemical interactions between sulfate ions and constituents of the cement paste. The disintegration appears to be resulted from chemical reactions with cement hydration products and the formation of a secondary compound, ettringite, accompanied by a large volumetric expansion and cracking of the concrete. SA was also known as "cement corrosion" in the early 1900s and is very common in the white "alkali flats." Early observations in these scenarios identified certain cement brands as being more resistant to deterioration than others.

(b) Alkali–aggregate reaction (AAR)

Alkali–aggregate reaction is a chemical reaction between certain specific mineralogical types of aggregates (either sand or gravel) and the alkali compounds (generally lower than 2 % of the cement composition) of cement in the presence of moisture (Pacheco-Torgal et al. 2008; ICOLD 1991a, b; Roy and Morrison 2000). Typical manifestations of concrete deterioration through AAR are expansion, cracking, exudations of jellylike or hard beads on surfaces, reaction rims on affected aggregate particles within the concrete, and sometimes pop-outs. The cracking may allow for moisture being more readily absorbed by the silica gel, which in turn accelerates AAR or further freezing–thawing damage.

Many new cases of expansion instances are being identified worldwide including that caused by alkalies in cement reacting with certain "glassy" siliceous aggregates such as opals, chalcedony, cherts, andesites, basalts, and some quartz, which is termed as "alkali-silica reaction (ASR)," and with certain specific carbonate aggregates, which is termed as "alkali-carbonate reaction (ACR)," as well as that caused by internal sulfate attack (SA). In these ongoing expansion cases, it appears that the alkalies are also supplied by certain minerals, such as feldspars in the aggregates, and the reaction and associated expansion may possibly continue indefinitely.

An ICOLD paper stated that nearly 10 % of concrete and masonry dams suffered from the damages due to aging undergo expansion. Recent studies suggest that this figure is higher, as many cases of dam expansion have been reported recently and it also should be noted that the phenomenon can manifest even if it is not directly apparent (ICOLD 1991a, b). Over 100 cases found to be seriously affected by AAR in terms of dam safety, and operations were discussed at the "USCOLD Conference on Alkali Aggregate Reactions in Hydroelectric Projects and Dams" in 1995.

(c) Freezing/thawing (FT) deterioration

Freezing/thawing deterioration was identified early in USBR history under the general term of durability of concrete without specific causes or solutions. It is the deleterious expansion of water within the cement paste leading to the destruction of the concrete matrix, which appears in the chilly regions solely.

Water present in the cement paste expands about 9 % upon freezing. When confined within a rigid, crystalline microstructure, the expanding ice crystals can exert pressures far exceeding the tensile strength of the paste, causing cracking and ultimately disintegration of the concrete (Shang and Song 2006). For this form of deterioration to take place, the concrete must be nearly saturated when it undergoes the freezing. Water conveyance structures are more likely suffered from repeated cycles of freezing/thawing actions. Areas subject to cyclic freezing, such as in spillways, and particularly those in fluctuating water surface levels, or in splash or spray zones, are the most susceptible to FT deterioration.

Freezing/thawing deterioration is most pronounced in highly porous concrete with a high W/C and those concretes without purposely entrained air bubbles, such as the concretes commonly used in the early twentieth-century construction.

(d) Corrosion of reinforcing steel

Corrosion of steel bars in concrete is an electrochemical process. The damages to reinforced concrete of thin hydraulic structures (e.g., buttress dams, aqueducts, embedded conduits, sluices, inverted siphons, culverts, tunnel linings, bridges) resulting from the corrosion of embedded steel bars manifest in the forms of expansion, cracking, and eventual spalling of the cover. In addition to the loss of cover, a reinforced concrete structures may suffer structural damages due to the loss of bond between steel and concrete and the loss of the cross-sectional area of reinforcement—sometimes to the extent that structural failure becomes inevitable (Bentur et al. 1997; Bertolini et al. 2014; Mehta and Monteiro 2006).

When designed, a concrete hydraulic structure is usual to state the minimum concrete cover for the reinforcement steel bars, which is normally stipulated by the corresponding design specifications. It is conventional to postulate that when the embedded steel is protected from air by an adequately thick cover of a low-permeability concrete, the corrosion of steel and other problems associated with it would not arise. However, this may not be entirely true from the high-frequency evidences—even some properly built reinforced concrete structures show premature deterioration due to steel corrosion. For example, a 1995 investigation organized by the Ministry of Water Resources (China) reported that of 200 hydraulic structures investigated (including 50 aqueducts, 33 embedded conduits, 36 sluices, 9 inverted siphons, 32 culverts, 18 tunnels, 8 gates, 14 bridges) in China that suffered from aging deterioration and required immediate repair, corrosion of reinforcing steel is responsible for 74 (accounting for 37 % of the total) (Luo, Luo, 1995).

The corrosion types may be classified according to the deferent criteria related to the corrosion mechanisms, final damage appearances, environmental factors, etc.

By the corrosion mechanism, the electrochemical potentials to form the corrosion cells may be generated in two ways:

- Composition cells may be formed when two dissimilar metals are embedded in concrete, such as steel bars and aluminum conduit pipes, or when significant variations exist in surface characteristics of the steel bars;
- In the vicinity of reinforcing steel, concentration cells may be formed due to differences in the concentration of dissolved ions, such as alkalies and chlorides.

As a result of aforementioned mechanism, one of the two metals becomes anodic and the other cathodic, among them the fundamental chemical changes occur. The transformation of metallic iron to rust is accompanied by an increase in volume that, depending on the state of oxidation, may be as large as 600 % of the original steel. This volume increase is believed to be the principal cause of concrete expansion and cracking.

Although the corrosion processes are not induced by environmental factors directly, they may cause the deterioration of the cover concrete and accelerate the ingress of aggressive species, making the pore solution in contact with the steel more corrosive. Among all the environmental factors, chloride icons (Glass and Buenfeld 2000) and carbon dioxide (Yoon et al. 2007) are mainly responsible for most corrosion of steel in concrete structures.

Chlorides can promote the corrosion of embedded steel rebar if present in sufficiently high concentration. The pore solution in concrete is an electrolyte containing various icons (e.g., sodium, potassium, calcium, hydroxyl) and is physically absorbed in the pores of the concrete. The chemical composition of the pore solution influences the conductivity of the concrete and the corrosion process. As the pH value of the pore solution in a well-cured new concrete is about 13.5, any steel in contact with such a pore solution should be in a passive state, so there is no problem with a young reinforced concrete structure if there is no chloride. Unfortunately, this high pH value can not always be kept. Under some circumstances for example, when concrete has high permeability, alkalies and most of the calcium hydroxide have been either carbonated or leached away, the pH value of concrete in the vicinity of steel may be reduced lower than 9. As a result, the passive film on the steel in contact with the pore solution with low pH value would no longer be stable, and rapid corrosion would occur on the surface of the "active steel."

Carbonation, or neutralization, is a chemical reaction of carbon dioxide in the air with calcium hydroxide and hydrated calcium silicate in the concrete and is another main cause responsible for the corrosion of reinforcement steel bars. Atmospheric CO_2 dissolved in the pore solution will react with the alkali hydroxide and the $Ca(OH)_2$. As a result, the pH value of the pore solution is brought down below 9, the passivity of steel is destroyed, and the corrosion reaction of steel in concrete is dramatically enhanced.

In addition to aforementioned two major factors—chloride and carbonation, temperature and moisture, as well as some other factors such as steel type, pore solution of concrete, and permeability of concrete, will influence the corrosion of reinforcement steel bars in concrete:

- Since the electrochemical reactions are mainly responsible for the reinforcement corrosion, moisture should be an essential substance in the corrosion. Normally, if there is no water in concrete, there should be no corrosion problem with the reinforcement.
- An increase in temperature will lead to the rate increase of all electrochemical processes, consequently the increase in corrosion rate, too (Pour-Ghaz et al. 2009).
- Stress corrosion cracking and hydrogen-induced embrittlement are caused by the combination of particular corrosion media and stresses for some types of steels (Villalba and Atrens 2008). Different types of steels possess different microstructures and compositions, so they usually exhibit different corrosion performances in concrete. Generally, high-strength steel is one of the materials that are sensitive to stress corrosion and hydrogen-induced embrittlement. Unfortunately, the reinforced concrete structures and rock slopes, particularly when the prestress technique is employed, demand high-strength steel.
- The pores can facilitate the ingress of CI, CO_2, O_2, H_2O and some other detrimental species from the environment. Therefore, higher porosity and large pore sizes lead to more severe corrosion (Hussain and Ishida 2010).
- The permeability directly affects two of the basic corrosion processes: the supply of depolarization reagents and the removal or accumulation of corrosive products. Therefore, it has significant influence on the steel corrosion in concrete.

3. Aging of embankment fill

(a) Cored and homogenous embankment

The long-duration process of consolidation of the embankment material after the completion of construction and during the first filling is the main cause of continuing deformations. This consolidation process may be influenced by environmental and operational actions (temperature, precipitation, earthquake, blasting, crest traffic, water level fluctuations).

Embankment deformation depends upon the mineral type, shape, hardness, grain-size distribution, moisture, and density of the compacted material, which leads to

- Differential deformation in adjacent sections of embankment, which may give rise to fissures leading to the internal erosion;
- Settlement at the contact with a concrete structure, which may initiate cracking, leaking, and erosion;
- Loss of freeboard.

Harmful effects of deformations can be prevented by appropriate positioning, and using, placing, and compacting, of materials. Monitoring of vertical and horizontal displacements will allow for the evaluation of deformations.

Loss of strength of embankment materials is associated with

- Wetting of improperly compacted embankment soil. Materials compacted on dry side without sufficient compaction effort to reduce the void volume will experience considerable particle reorientation leading to the large settlement and/or differential settlement, which, in turn, may lead to cracking and piping and manifest high pore pressure.
- Reduction in cohesion when wetted of some specific soils. Reservoir seepage, abutment groundwater, and rainfall may affect downstream shell stability by wetting.
- Change in the state of stress. Embankment heightening may stress the existing material beyond its peak strength, and a lower residual strength is onset. Cycles of drying/wetting of high-plasticity clays may result in slope instability, which is usually in the form of shallow downstream sliding. As the clay dries, capillary stresses lead to tensile cracking. When water is again available to the crack, material sloughs off into the crack and there is a loss of strength in the swelling clay at the crack face. A progression of these events would reduce the effective dam width and manifest other detrimental scenarios such as
- Pore pressure mounting. It is generally associated with progressive opening of transverse cracks in the core or through the whole fill.
- Adverse deformation. It may be associated with differential foundation settlement, differential embankment compression, embankment arching, contact with concrete structures, or drying out of the upper portion of the core during prolonged dry periods (Hjeldnes and Lavania 1980; Lo and Kaniaru 1990). It also may be associated with dissolution of dispersive clays, a defective layer in the core, poor material placement, or low permeability in downstream dam shell.

Good design and construction practices may preclude excessive pore pressure mounting by eliminating foundation overhangs, by shaping the abutment and structure contact, by using ductile material in the upper part of the dam, and by adequate downstream draining and careful material placing. Periodic inspections and displacement measuring, supplemented by seepage and pore pressure monitoring, are appropriate for the detection of these scenarios.

Internal erosion may manifest within the embankment core or the downstream shell with soils susceptible to cracking, piping, or other types of erosion (Fry 1997). Quite frequently, it occurs at the embankment contact with the foundation. Internal erosion generally finds its origin in design and construction inadequacies, but may progress unnoticed for a long time.

Detection of internal erosion depends on visual inspection, water flow monitoring, pore pressure readings, and turbidity measuring. Drilling and geophysical investigations are helpful to capture the internal erosion.

(b) Upstream surface erosion

Surface erosion, although a very common aging scenario, has not yet been regarded as an important contributor to embankment failures. Erosion on the downstream slope and crest may be due to heavy rainfall directly or surface water runoff, brief crest overtopping, and wind-driven wave splash over a parapet wall. Erosion on the upstream slope may be attributable to wave action on the riprap with too small blocks or inadequate bedding, breakdown of riprap, or freezing/thawing displacement. Surface erosion is readily detected by routine visual inspection, and a timely repair is emphasized.

Seepage through CFRDs is the aging scenario that had been mostly reported for dams constructed of dumped rockfill with little or no roller compaction, whose excessive settlement led to the damage of slab joints and even slab buckling. The high permeability of rockfill leads to large leakage, although internal erosion was not an urgent problem. Remedial measures are only undertaken when the seepage became too large or limitations on operation are found to be too restrictive.

(c) Others

Analyses of data on the deterioration of earth- and rockfill dams revealed other aging scenarios concerning the behaviors and materials of some specific dam elements.

① Loss of bond between concrete structure and embankment. It is associated with differential movements in the bonding zone due to settlement of the embankment material attributable to inadequate compaction and/or foundation treatment, or internal erosion. Settlements may lead to arching in the fill and reduction of the effective stress. Seepage develops through cracks in the fill or along the concrete structure promoting internal erosion if the filter drainage system is not able to tackle the severe erosion actions.

This scenario may be detected by inspection with regard to vertical displacements and seepage.

② Aging of geosynthetic material. Geosynthetic materials are installed for the purpose of water barriers (geomembranes), material separators, filters (geotextiles), and drainage and stabilization (geogrids, geotextile wraps). The common aging agents for geosynthetic materials are ultraviolet light, stress, temperature, moisture extraction, oxidation, chemical and biologic substances.

The construction operation including the storing, handling, and installing can give rise to the most damaging (ICOLD 2010).

Monitoring of the aging process may be carried out by physical and mechanical tests on the test samples from a project field test section, from time to time.

③ Aging of asphalt concrete facing. The bituminous material coating and binding together the aggregates in asphalt concrete breaks down from exposure to oxygen and ultraviolet light. Abrasive debris and wave action remove the bituminous coating from aggregates, allowing the aggregates for weathering. These aging scenarios will deteriorate the asphalt concrete as water barrier. Flames used in contact with the asphalt concrete for heating previously placed material cause severe deterioration of the bituminous material.

Asphalt concrete also has been used for construction of an internal water barrier (core). So far, no examples of aging have been found, apart from only a very poorly designed and constructed asphalt concrete core being suspicious of aging.

④ Aging of soil–cement. Soil–cement slope protection may be deteriorated, attributable to the effects of freezing/thawing, abrasive debris, and wave actions. These may be hastened due to insufficient cement in the mixture, poor mixing, insufficient compaction, or lack of bond between layers. Soil–cement placed in stair-step fashion is sufficiently massive that repairs would only be necessary if large segments of protection are lost. However, aging damage to the soil–cement placed by the plating method demands immediate repairs.

18.6.3 Mitigation of Aging

1. Analysis

The first step in the mitigation is the analysis of aging process and its consequences on dam safety and project operations (Bangert et al. 2003). In most cases of deterioration, the specific causative process can be identified by concrete materials' experts, with the help of petrographers augmented by field and laboratory testing. The role of nondestructive testing techniques accompanied by the analysis and presentation using tomography methods is a fast advancing field and can help to develop an "internal" view of the structural condition.

The effects of aging processes on structural integrity and dam safety can sometimes be obvious, whereas in other cases such as AAR, detailed modeling that comprises finite element analysis is necessitated identifying the mechanisms and forecasting the potential effects such as cracking in the future.

A related key aspect is the long-term susceptibility to the corrosion and performance of high-strength anchors. They are increasingly used to improve the stability of structures where construction joints are deteriorating or where criteria have escalated. Corrosion of reinforcing steel is clearly a major problem although it is often more of a maintenance issue and not always related to dam safety.

Understanding the predominant processes at the work can provide a basis to plan an appropriate structural management program. In some of such cases, the process may have a finite lifetime, while in others, it will continue indefinitely. In the latter case, long-term budget provisions should be made accordingly rather than a one-time fix. Economic analysis for the options of remedial actions should thus be done on a "life cycle" basis (Yang 2007). In some cases, these considerations may indicate that there is only a finite economical remaining service life and plans need to be made for a replacement structure.

2. Prevention and mitigation

No material is inherently durable. As a result of interactions with its environment, the microstructures and consequently the properties of materials change with time. A material may be assumed to have reached its end of service life when its properties under given conditions have deteriorated to such an extent that the continuing utilization of the structure is unsafe or the costs of ongoing repairs make the structure uneconomical.

In addition to having to deal with the physical and chemical processes of deterioration, the determination of acceptability of such structures is often made more difficult by changing criteria such as escalation in design floods (PMFs) and earthquakes (MCEs) due to improved knowledge and society's evolving demands for public safety.

The economic value of the structures for water supply, energy generation, or flood control is frequently substantial, and thus, life extension and rehabilitation of existing dams and appurtenant works is an important activity. It is vital that the industry community understands what is the process of aging, how to assess its extent, and which remedial actions can be taken, to cost-effectively extend the service life of these structures.

It is ideal that prevention and mitigation of aging may be achieved through the design, construction, and operation of high quality. A well-planned and well-performed surveillance program is crucial for the early detection of aging scenarios. Provisions of convenient access to all vital areas of the structure will enhance the surveillance and regular maintenance.

(a) Concrete structures

For a durable concrete, the following are the main contributions (in chronological order) (Ahmaruzzaman 2010; Saraswathy and Song 2007; Yoon et al. 2002):

- Development of special cements to improve concrete quality, such as low-heat and sulfate-resisting cements;
- Development of internal vibrator to consolidate concrete—this equipment significantly reduces the water content of concrete and makes it less permeable;

- Determining the causes of and the solutions to AAR and FT attack, using methods such as petrographic mineralogical examination and long-term testing to identify the parameters, which affects the durability of concrete under these conditions; and
- Incorporating fly ash in concrete construction—fly ash may improve concrete workability, reduce the porosity of the cement paste, and improve its resistance to ST and ASR.

Because water, oxygen, and chloride ions play important roles in the corrosion of embedded steel and the cracking of concrete, it is obvious that permeability of concrete is the key to control the various processes involved in the corrosion phenomena. The permeability of concrete is mainly determined by the porosity of concrete and its pore size distribution, which are dependent on the ratio of W/C. Sometimes, the permeability could vary by as much as two orders of magnitude as W/C increases from 0.4 to more than 0.7. Other factors such as hydration process and the mineral mixture can also significantly influence the porosity and permeability.

Corrosion inhibitors, such as calcium nitrite, also can be added to the water mix before pouring concrete (Soylev and Richardson 2008). The nitrite anion is a mild oxidizer that oxidizes the soluble and mobile ferrous ions present at the surface of the corroding steel and causes it to precipitate as an insoluble ferric hydroxide. This causes the passivation of steel at the anodic oxidation sites.

Coatings and cathodic protection for reinforcing bars provide common approaches to prevent corrosion (Manning 1996). The former comprise two types: anodic coatings (e.g., zinc-coated steel) and barrier coatings (e.g., epoxy-coated steel), and the latter involves the suppression of current flow in the corrosion cell, either by supplying externally a current flow in the opposite direction or by using sacrificial anodes. Due to their complexity, they are finding limited applications, and long-time performance of these protections is still under investigation in many countries. In addition, they are costlier than producing a low-permeability concrete through quality, design, and construction controls.

(b) Embankment dams

Excessive differential deformations may be prevented by the appropriate selection, zoning, and compaction, of materials.

Cracking of dam core may be avoided by

- Placing ductile material at locations subject to tensions;
- Shaping abutments with smooth transitions from area to area;
- Battering adjoining appurtenant structures to permit a full compacting and bonding of core;
- Adopting a zoning scheme that does not place materials with large differences in deformation modulus adjacent to each other;
- Widening excavations and flattening slopes to prevent arching;
- Flaring the core near and at the abutment interfaces.

Internal erosion may be prevented by installing an upstream self-healing transition zone and a downstream filter zone of the core, avoiding internally unstable materials, treating fissured bedrock, and dispersive soils.

If instrumentation is to be installed within the dam core, it should be so designed that it does not become an "Achilles heel" for future degradation of the dam.

Slope protection with riprap of appropriate durability, size, grading, thickness, slope inclination, and proper placement is required. The riprap thickness with lower-quality rock should be raised, and bedding layer to prevent the underlying embankment from erosion is installed.

Where the only available rockfill may experience excessive deformations, which could be unacceptable for a concrete face slab, alternative use of an asphalt concrete face slab, geomembrane face, or central asphalt core is recommended.

The use of asphalt concrete requires careful selection of bitumen and aggregates, complete mixing, temperature control, and compaction.

The use of geosynthetic materials requires careful studies on their composition and manufacture and characterization of the environment into which the materials will be installed and handled, to avoid accelerated aging aging (ICOLD 1991a, b). Periodic retrieval of coupon samples from a field test section for the aging testing and evaluation is demanded.

Soil–cement requires careful selection of the soil to be mixed with cement and water. Design mixes are tested in laboratory to assure its engineering properties.

3. Rehabilitation

Many methods of treating surface deterioration have been available (ICOLD 2000a), including chemical treatment of the upstream face; upstream concrete coating using epoxy/gunite/shotcrete/sealant materials; geomembranes attached to the upstream face to provide a waterproof; downstream drainage layers covered with gunite or shotcrete and reinforced to the downstream face; grouting or other internal treatment; and concrete buttresses on the downstream face.

Remedies for more invasive forms of deterioration such as AAR or other swelling or internal mechanisms tend to be more structural in nature since no complete "cure" is available once the structure has been constructed. Some solutions available are anchoring to improve shear strength on lift joints or as an attempt to restrain the structure against the aging, stress relief by slot cutting, or other structural modifications.

The repair and replacement costs associated with concrete damaged by the corrosion of reinforcing steel have become a major maintenance expense. It is now prevalent to provide a waterproof membrane, or a thick overlay of an impervious concrete mixture on newly constructed, or thoroughly repaired surfaces of reinforced concrete structures that are large and possess flat configuration, if possible. Typically, concrete mixtures used for the overlay are of low slump, very low water-to-cement ratio (made possible by adding a super-plasticizing admixture), and high cement content. Portland cement mortars containing polymer emulsion (latex) also show excellent impermeability and have been exercised for the overlay installations.

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