A STUDY ON DAM INSTRUMENTATION RETROFITTING: GÖKÇEKAYA DAM

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ABSTRACT

A STUDY ON DAM INSTRUMENTATION RETROFITTING: GÖKÇEKAYA DAM

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Multi-purpose project requirements lead to construction of large dams. In order to maintain the desired safety level of such dams, comprehensive inspections based on use of a number of precise instruments are needed. The ideal dam instrumentation system should provide time-dependent information about critical parameters so that possible future behavior of the structure can be predicted. New dams are normally equipped with adequate instrumentation systems. Most of the existing dams, however, do not have adequate instruments or current instrumentation systems may not be in good condition. By implementing the modern equipment to existing dams, the uncertainty associated with the impacts of aging or unexpected severe external events will be reduced and possible remedial measures can be taken accordingly. This study summarizes the major causes of dam failures and introduces the instruments to be used to monitor the key parameters of a dam. The concept of the instrument retrofitting to an unmonitored dam is highlighted through a case study. A sample system is proposed for Gökçekaya Dam, with reference to an investigation of the current condition of the structure. The deficiencies observed during a site visit are listed and the corresponding rehabilitative repair measures are suggested. Finally, different alternatives of a new instrumentation system are introduced and compared in terms of technical and economical aspects.

Keywords: Dam Safety, Dam Monitoring, Dam Inspection, Instrument Retrofitting, Gökçekaya Dam

ÖZ

MEVCUT BARAJLARIN ÖLÇÜM SİSTEMLERİNİN GELİŞTİRİLMESİ ÜZERİNE BİR ÇALIŞMA: GÖKÇEKAYA BARAJI

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Çok amaçlı proje gereksinimleri nedeniyle büyük barajlar inşa edilmektedir. Arzu edilen baraj güvenlik seviyesini sağlamak için hassas ölçüm aygıtlarıyla kapsamlı kontrollerin yapılması gereklidir. Ayrıca ideal baraj ölçüm sistemi, yapının gelecekteki olası davranışını öngörebilmek için ölçüm parametrelerindeki zamana bağlı değişimi de vermelidir. Yeni inşa edilen bütün barajlarda gerekli ölçüm aygıtları bulunmaktadır. Ancak mevcut barajların ölçüm sistemleri bulunmamakta veya yeterli düzeyde olmamaktadır. Mevcut barajlara modern aygıtların takılmasıyla, barajlarda zamanla yıpranan elemanların ya da beklenmeyen dış etkenlerden kaynaklanan zafiyetlerin yaratacağı etkilerdeki belirsizlikler azaltılabilir; hatta belirlenen sorunların

giderilmesi için onarıcı çözümler uygulanabilir. Bu çalışmada, barajların yıkılmasına yol açan başlıca nedenler tartışılmış ve ölçüm aygıtları kullanılarak bu parametrelerin izlenmesi üzerinde durulmuştur. Ayrıca, mevcut barajların ölçüm sistemlerinin geliştirilmesinin sağlayacağı katkılar bir örnek çalışma ile tartışılmıştır. Bu bağlamda, Gökçekaya Barajı'nın mevcut durumu bir teknik inceleme gezisiyle değerlendirilmiştir. Baraj ve civarında gözlenen zafiyetler belirtilmiş ve gerekli görülen düzenlemeler önerilmiştir. Son olarak, Gökçekaya Barajı için alternatif ölçüm sistemleri ele alınmış ve hem teknik, hem de ekonomik açıdan karşılaştırmaları yapılmıştır.

Anahtar Kelimeler: Baraj Güvenliği, Barajların İzlenmesi, Barajların Tetkiki, Sonradan Eklenen Ölçüm Aygıtları, Gökçekaya Barajı

To My Mother and Father...

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CHAPTER 1

INTRODUCTION

1.1 General

Every year, many dams are built all around the world. Sophisticated contemporary design approaches for multi-purpose dams are relatively complicated that some simplifying assumptions are made. Possible weak zones of dam body, foundation, and appurtenant structures are determined according to these approaches. Simplifications in design procedure would introduce some uncertainties. Therefore, validity of design assumptions and current status of dam safety can be assessed via some instruments installed in and close vicinity of the dam. In all design studies of dams, the items to be monitored, instruments to monitor these items, type, quantity and installation locations of instruments need to be determined precisely. After the design, it is the task of the site staff to ensure the proper installation and protection of instruments, especially the embedded ones from damage that might have been caused by machinery during construction.

Type of demand would also dictate the degree of instrumentation. For example, greater hydropower projects are needed to meet growing energy demands. The key element of hydropower generation is the net head above the turbine. In order to increase the head, the dams should be taller than those built in the past. When the reservoir behind the dam gets deeper and larger, the hazard potential of the dam becomes higher. Therefore, in order to protect the downstream from the floods caused by dam breaches, dams should be examined and monitored periodically. Due to great uncertainties that exist in dam design and construction, such as foundation conditions, hydrologic data, nature of materials and so on, unexpected behaviors may be observed any time throughout the physical life of the structure. Economic, effective, and fast corrective actions should then be taken. Apart from the positive effects of monitoring the dam in its life time by instrumentation, the designer can also get invaluable data and experience from the behavior of the existing structure. He/she can then improve his/her knowledge and skills to produce a contemporary design.

Early dam examination practices consider only the visual inspection, done by walking on dam body and checking the structural integrity visually from inside of the galleries. Although this method with an experienced examiner gives valuable information about the physical condition of dam body, more complex items, such as seepage, movement, and state of internal stresses cannot be determined without using proper equipment. With the advancement in dam monitoring, a number of instruments are developed and quickly started to be used in new dams. Early ones have limited capabilities and readout of every instrument should be made manually. Development of electronic technology, such as vibrating wire, made the use of new types of instruments, which have greater accuracy, longer life and possibly smaller dimensions. Moreover, new instruments are equipped with their automated data loggers. By using computers, one can then easily follow the behavior of independent parts and/or the whole structure.

1.2 Scope of the Study

Recently developed and highly precise measuring equipment is used in all newly built dam projects, however most of the existing dams do not have adequate instruments or nothing at all. In most cases, the instruments installed to the dam are not working or giving irrelevant measurements. As the dams age, concrete body deteriorates, drains get clogged, grouts and cut-off walls lose their effectiveness and construction joints separate due to cyclic loading. These adverse effects of aging bring the dam to a more vulnerable condition against breaching. In order to protect the downstream, older dams should be rehabilitated by most effective and economical methods. As a first step, the current condition of the dam should be assessed properly.

New techniques can also be applied to existing, unmonitored dams. Instruments can be retrofitted to measure the critical parameters. Not all types of instruments can be retrofitted to an existing dam but also not all types are needed in most conditions. Retrofitted instruments cannot give information about the behaviors and events occurred before the installations but provide a basis for future unexpected events thus reduce the level of uncertainty. Some researchers believe that after a certain age, movements and seepage rates are balanced and further monitoring and inspection practices are meaningless. However, as mentioned before, the effects of unpredicted occurrences, such as earthquakes and especially the deterioration caused by aging should be carefully evaluated and necessary corrective actions should be planned and taken. Moreover, it is obvious that in some areas, the dam reservoir causes climatic changes and the current hydrologic data may differ from the design conditions. This situation may increase the duration and magnitudes of the expected floods. Furthermore, in the days of the design period of an existing dam, structural and geotechnical design techniques might have been incapable of precise modeling. In order to ensure the validity of assumptions made and the adequacy of the design, many parameters, such as the uplift pressures, seepage through the dam may be required and most of them can be obtained from the retrofitted instruments.

Many countries attempt to develop their own inspection standards for dams and their appurtenant structures. Creating an inspection standard includes an extensive training program for inspection personnel. It is obvious that if all or at least, most of the dams include instrumentation; inspections can be accomplished in a more effective and economical way since the personnel could be trained to make reliable comments about the outputs of the same system of instrumentation. Retrofitting of instruments can be very helpful for easy, economic, and effective inspection of older dams. Their safety levels can also be estimated almost as accurate as the newer ones. The aim of this thesis is to introduce the concept of dam instrumentation retrofitting. A case study is conducted for Gökçekaya Dam. Chapter 2 summarizes possible causes of failures of dams and items to be monitored. Chapter 3 provides information about general dam instrumentation and the instruments that can be retrofitted to existing dams. Visual observations performed during a visit to Gökçekaya Dam site are provided in Chapter 4. Rehabilitative measures proposed for Gökçekaya Dam are described in Chapter 5. The conclusions of the thesis and recommendations for further studies are presented in Chapter 6.

CHAPTER 2

POSSIBLE CAUSES OF FAILURES AND ITEMS TO BE MONITORED

2.1 General

Because of their complex natures, failures of dams are generally due to more than one reason. Singh (1996) stated that the dam failures can occur as a result of structural deterioration, extraordinary natural events or man-made activities. Dam failures are normally categorized into two types: Type-1, component failure of a structure that does not result in a significant reservoir release; and, Type-2, uncontrolled breach failure of a structure that results in a significant reservoir release (INDNR, 2003). Type 1 failures are defined as localized structural or component failures which may exhibit a wide variation from localized seepage to trash-rack failure. All of these deficiencies require an immediate action. Type 2 failures are expressed according to their importance in terms of the release of reservoir, which will lead to a significant loss of life and damage to properties. Generally, Type-2 failures are often observed due to the inadequacy in remedial measures taken to correct the Type-1 failures (INDNR, 2003).

Principal causes of failures and corresponding consequences in major failures are summarized in Table 2.1.

Table 2.1 Causes and consequences of major dam failure (Tosun, 2004)

Dam	Country	Type	Failure Date	Failure Reason	Loss of Lives
Puentas	Spain	Rockfill	1802	Foundation Failure	60
Southfork	USA	Earthfill	1889	Overtopping	2,200
Saint Francis	USA	Arch	1929	Structural Failure	450
Vega de Tera	Spain	Buttress	1959	Structural Failure	144
Malpasset	France	Arch	1959	Foundation Failure	421
Oros	Brazil	Earthfill	1960	Overtopping	1,000
Bab-ı Yar	Ukraine	Earthfill	1961	Overtopping	145
Hyokiri	Korea	-	1961	-	250
Panshet	India	Earthfill	1961	Overtopping	1,000
Q. la Chapa	Colombia	-	1963	-	250
Vaiont	Italy	Arch	1963	Overtopping	3,000
Baldwin Hills	USA	Earthfill	1963	Foundation Failure	3
Nanaksagar	India	Earthfill	1967	Overtopping	100
Pado	Argentina	-	1970	-	25
Henan	China	Earthfill	1975	Overtopping	230,000
Teton	USA	Earthfill	1976	Piping	14
Machhu II	India	Earthfill	1979	Overtopping	2,000
Belci	Romania	Earthfill	1991	Overtopping	48
Gouhou	China	Rockfill	1993	Piping	300
Tirlyan	Russia	Earthfill	1994	Overtopping	27

2.2 Possible Failure Modes

2.2.1 Overtopping

Overtopping in earth dams may generally occur due to spillway inadequacy. Major causes of this inadequacy are attributed to the design errors. Moreover, the spillway capacity decreases over time, due to blockage with debris or increase in roughness coefficient because of the excessive damage, such as cavitation. Another cause of overtopping may be excessive settlement of the embankment, which leads to reduction in the freeboard (FERC, 1999).

2.2.2 Seepage Induced Failures

The term "seepage induced failures" are more descriptive than piping induced failures. All dams show some seepage and it may not necessarily be critical if the velocity and amount is under control (INDNR, 2003). Piping is often referred to as one of the mechanisms of seepage failures, which starts at the exit point of seepage path and develops towards the upstream face. Increasing flow rate erodes the material to form a pipe and if uncontrolled, piping causes severe settlement and slope instability. The second mechanism is internal erosion; which, in contrast to piping, starts in a crack, generally caused by differential settlement and poor compaction, and develops towards the exit point. Dam Safety Inspection Manual (INDNR, 2003) indicates that seepage is the major failure cause of embankment dams. Uncontrolled seepage would normally lead to slope stability problems in embankment body. Washing out of foundation material, diminishes the bearing capacity and causes sliding of concrete dams due to saturation. Foundation damages generally cause differential settlement. In case of concrete dams, severe cracking and opening of joints are usually observed.

2.2.3 Earthquake Failures

Earthquakes mostly result in cracking, opening of joints and uneven foundation movements in concrete dams and liquefaction in embankment dams. In general, earthquake damages lead to severe increase in seepage in both foundation and body, and thus cause large settlements. As a result of such settlements, the freeboard reduces. When earthquakes are combined with the waves caused by landslides into the reservoir, a catastrophic overtopping failure may occur. Liquefaction, which could be devastating, is simply defined as the loss of bearing capacity of non-cohesive soils in seismic actions which then act as a liquid rather than solid. After the infamous San Fernando Earthquake (1971), the observations of Seed et al. (1975) indicated that Lower and Upper San Fernando Dams failed due to severe liquefaction of fill. The in-depth inspections also pointed out that the hydraulic fill material was greatly liquefied. Liquefaction was also the main reason of collapse of many buildings in Adapazarı and its surrounding during the 1999 Gölcük earthquake (Çetin et al., 2004).

2.2.4 Failures Caused by Body and Foundation Movement

Both concrete and embankment dams are adversely affected by the movement of the body and/or foundation. These movements are often related with the foundation conditions. Water-sensitive foundation conditions with weak or void zones and inadequate treatment of possible fault lines will lead to increase in seepage. Possible failure zones may then be washed-out. Foundation movements may also lead to the excessive and uneven settlement. The movements may be in all directions concerning the body and foundation. Body movements can be further divided into three as vertical, horizontal, and rotational.

2.3 Dam Safety Concept

Planning, design and construction phases of a dam require extensive elaborate surveys and studies. Even if the design and construction phases of a dam are carried out properly using sound material, periodic monitoring and inspection are required to assess the safety level of a dam throughout its lifetime (Yanmaz, 2006).

Dams exhibit a potential fatal risk to people and property at the downstream due to the immense amount of impounded water. The goal of dam safety is to minimize the risk of failure by promoting the application of competent technical judgement and by the use of contemporary techniques and materials in all phases of development and use (USBR, 1987).

The great majority of existing dams were designed and constructed during the last century using conventional design procedures (De Michele et al., 2005). Therefore, the adequacy of such dams with respect to current conditions needs to be checked. Conventional design procedures are deterministic such that they do not consider possible variations of parameters involved in the phenomenon concerned (Yanmaz and Çiçekdağ, 2001). With the application of the reliability theory, probabilistic dam design approaches have been proposed that enable the assessment of various reliability levels under different combinations of design parameters (Yanmaz and Günindi, 2008). Post analysis of hydrologic data should also be carried out to detect possible temporal variations from the original analysis. Yanmaz and Günindi (2008) investigated the effect of type of hydrologic model used in a flood frequency analysis. They observed that contemporary techniques used for frequency analysis, i.e. multi-variate flood frequency analysis, yielded relatively conservative results compared to the classical approach, which is carried out using uni-variate.

In design of new dams, evaluating and increasing the safety level of the structure should be the first aim of the designer. The probability of failure of a dam depends on many factors. These factors should be extensively evaluated in order to get a realistic probability of failure. Then the results of failure should be predicted accurately. Only after these studies, the required strength of structure and the soundness of materials can be decided. However, the competent design is not satisfactory in all times. The safety level of a dam changes continuously during the lifetime of the structure. That is why time-dependent probabilistic safety analyses should be carried out throughout the lifetime of the structure.

Some key elements and the parameters would provide information about the level of safety of a dam in a specified time interval. However, these parameters should be monitored and evaluated precisely. By using suitable instruments, one can consider the performance of the structure and possible repair needs.

The rehabilitative solutions should preserve the structural safety and focus on the prevention of loss of human lives at a reasonable cost. They intent to provide rehabilitative measures at a lowest cost while retaining the project benefits, provide protection of project facilities and public and private property, consider non-structural and combinations of structural and non-structural modifications to minimize the cost of rehabilitation and apply contemporary design standards and construction practices (USBR, 1987).

Items and parameters to be monitored in order to determine the current safety level of a dam are discussed in the following sections. The instruments to be used to monitor the aforementioned items and parameters are also introduced.

2.4 Dam Monitoring

2.4.1 General

Performance monitoring of individual items of a dam has an utmost importance in dam safety. The possible causes of dam failures generally result from the time-dependent deterioration of individual elements of a dam. Thus, the possible deficiencies should be monitored either by direct or indirect measuring techniques. Characteristics of items to be monitored should then be analyzed for determining the remedial actions to be taken. Monitoring should also be planned during the design phase of a dam. In planning, every item regarding with the failure mechanisms explained above should be carefully examined for the dam concerned. First, critical items that exhibit a hazard are determined. Then the items that will trigger the critical events are listed. Finally the items that should be monitored constantly in order to reduce the corresponding risks are selected. After obtaining the items that should be monitored, the second step is the selection of the most appropriate equipment, in view of the performance of monitoring and economy. Required monitoring equipment would differ from dam to dam. However, the following factors should be taken into account for establishing a basis for instrumentation system design guidelines.

2.4.2 Pore Water Pressure and Seepage

As mentioned before, all dams will leak some amount of water as seepage, which should be monitored. Most of the dams have remedial measures to reduce the amount of water entering foundation and body of embankment dams, such as cut-off walls, grout curtains for foundation protection and impervious upstream blankets and membranes for reducing the saturation of embankment. In addition to those, some other elements are also used to control the adverse effects of seeped water, such as zoned and filtered embankments to reduce the chance of seepage-induced erosion, pressure relief wells to reduce the uplift pressure, and chimney, foundation and toe drains to collect and route seeped water. As time passes, the effectiveness of these items decreases and the seepage may exhibit a risk for dam safety. Pressure measurements before and after the cut-off walls and inside the pressure relief wells give information about the repair needs. Monitoring and measuring the flow at drains and assessing the quality of seeped water would also be very helpful for checking the condition of filters and the determination of possible internal erosion.

2.4.3 Body and Foundation Movement

Movement in concrete and embankment dams may be due to many different reasons. The most important reason for movement may be attributable to overstresses in dam body. All dams deform as a response to applied loads. Excessive movement may indicate developing problems (INDNR, 2003). In concrete dams, vertical movement is generally caused by the expected settlement of foundation, whereas lateral and longitudinal movements may result from a stability problem. Results of measurements of vertical, lateral, longitudinal, and rotational movements via surface monitoring system can be compared with the past recorded information. Observing time-dependent data is very helpful for estimation of future behavior and severity of causes of movements.

Another significant information that can be obtained for concrete dams, rather than the surface monitoring, is from the construction joints. These joints tend to open and close with respect to loading and temperature stresses. Constant monitoring coupled with temperature measurements should be used to determine the internal stresses caused by the fluctuating reservoir levels and other loading combinations. Cracks on the concrete that previously occurred should also be monitored with joint monitoring. The movement of these cracks gives information about the stress concentration and the further development of crack.

2.4.4 Mass Temperature

Temperature monitoring of the fresh-poured mass concrete during constructional stage would provide information about dehydration heat and the cooling requirements. Without proper cooling of the fresh concrete, cracks may develop and thus reduce the strength of concrete.

2.4.5 Seismicity

Finally seismic movements can be recorded and studied to find out the actual natural frequency of dam body and the intensity of forces that the dam deals with when an earthquake takes place.

2.4.6 Automation

Dam Safety Inspection Manual (INDNR, 2003) divided the risk factors for dams into four and one of them is “human factors”. Operational mismanagement is one of the elements of human induced risk factors, which creates a great risk especially in floods. For example, a delay in the manual operation of spillways may cause an “overtopping” which may then lead to a total destruction. Nowadays, automated systems take this responsibility. They constantly monitor the equipment, evaluate the inputs and trigger possible warning messages. Another advantage of automated systems is the continuous monitoring of individual elements in a dam, such as seepage, movement, and pore water pressure. The monitoring staff requirement is then reduced.

CHAPTER 3

DAM INSTRUMENTATION AND RETROFITTING

3.1 General

There are a number of companies producing various instruments to be installed on dams. With the advancement in technology, vibrating wire and electrical resistance instruments are chosen generally in large dam projects. Vibrating wire technology is simply based on the measurement of the change in oscillation frequency of a wire due to forces acting on the anchors moving freely in one direction in which the wire is connected. The well-known advantages of vibrating wire technology are their protection from moisture and forces by the help of the casing, longer life, stability, accuracy, and their suitability to automation. Use of most electrical resistance instruments are restricted by total length of cable. Therefore, the automation of such equipment is difficult compared to the vibrating wire ones (USACE, 1994). The selection of the type of instrument is generally based on considering the requirements of the project, condition of installation areas, and the cost of instruments. The optimum instrument selection is then determined for that project with the minimum total cost and maximum efficiency. Sezgin (2008) carried out a study to evaluate the instrumentation system of a newly constructed dam, Cindere Dam, and proposed a number of additional alternatives.

Since instrument retrofitting is defined as addition of recent-technology equipment to an existing dam, the present condition, required repair works, and the available spaces to install the equipment should be evaluated carefully. These necessary items should be studied in a technical manner, such as designing a new monitoring system. An old dam has also an unknown degree of failure risk which will threat the human lives and properties. So the data gathered from retrofitted instruments not only give information on the interested items, but also help in evaluation of possible deficiencies. Required repair actions can then be implemented.

3.2 Instruments for Pore Water Pressure Monitoring

The pore water pressure is the main reason of uplift forces, which acts on dam body and reduce the dam's stability condition. Many remedial measures are used to reduce the uplift. However, as dam ages, their effectiveness reduce drastically. Piezometers can be installed to dam foundation for both measuring the pore water pressure and examining the effectiveness of uplift reduction systems.

In early times, standpipe piezometers have generally been used in uplift measurement. They are simple filter units for measuring the water level (SISGEO, 2004-a). These types of piezometers consist of several elements, such as a polyethylene cylindrical filter, PVC tubes to provide the connection to surface, and a top cap which protects the piezometer from frost (SISGEO, 2004-a). The readouts are taken with water level detectors which give a visible or audible signal when come into contact with water. The reels of water level detectors are graduated such that the water depth in standpipe piezometer can be measured indirectly by using the bottom elevation of the piezometer.

Standpipe piezometers can also be automated with pressure transducers. However, they have to be positioned in the piezometer tube at a depth that will always be below the water level. The pipe of piezometers should be larger than the regular ones (SISGEO, 2004-a). There exist a number of limitations on the use of standpipes, such as long lag time on certain soil types, potential freezing problems, clogging possibility, and the possible damages due to settlement (INDNR, 2003).

The pipes of standpipe piezometers, different tips and filters to be used in certain soil types, and accessories, such as water level detectors and reels are presented in Figure 3.1.

Figure 3.1 Standpipe piezometers and accessories (SISGEO,2004-a)

Another type of pore water pressure monitoring equipment is pneumatic piezometer. These types of piezometers have some advantages, such as reliability, accuracy, reading simplicity, durability, and low cost (SISGEO, 2004-a). Piezometer operation requires a supply of pressurized inert gas (dry nitrogen). Water pressure is balanced with pneumatic pressure supplied from the gas cylinder of readout unit (SISGEO, 2004-a). Their disadvantages are attributable to long measurement time for relatively long tubes and limitation in reading high and subatmospheric pressures (Fell et al., 2005). In addition to those, INDNR Manual (2003) expresses the limited suitability of these types of piezometers for retrofitting. A typical pneumatic piezometer is shown in Figure 3.2.

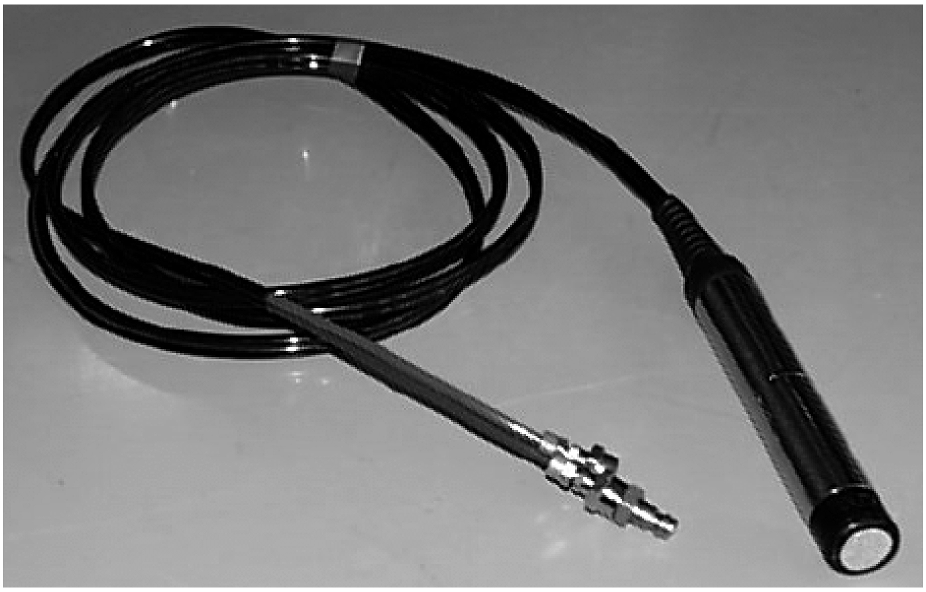

Figure 3.2 Pneumatic piezometer (SISGEO, 2004-a)

The third, most commonly chosen and the recommended type for retrofitting, is vibrating wire piezometers. With their short lag time and high accuracy, the vibrating wire piezometers can be installed in various different ways. Vibrating wire piezometers can be installed either in a borehole or directly embedded to the embankments. In addition to these, a special version of vibrating wire piezometers, named as drive-in piezometers, are offered, which are intended to be pushed directly into the soft soil (SISGEO, 2004-a). As a rule of thumb, in embankment dams, piezometers should be installed in both fill body and foundation; however, the only place to measure the pore pressure in concrete dams is the foundation. Observation wells in embankment dams can also be used as a borehole for piezometers. In Figure 3.3, different types of vibrating wire piezometers are shown.

Figure 3.3 Vibrating wire piezometer (SISGEO, 2004-a)

3.3 Instruments for Reservoir Water Level Monitoring

The conventional method to measure a reservoir level is to use a non-recording staff gauge. Water level measurements can be automated by retrofitting reservoir level sensors, which extend along the upstream face of dam.

3.4 Instruments for Seepage Monitoring

Since every dam leaks some water as seepage, the change in the amount of seepage should be carefully monitored and the necessary actions should be taken quickly. In concrete dam body, seepage water, which leaks from construction joints or cracks, is collected in galleries and then drained. In addition to that, seepage flow at foundation is drained by pressure relief wells for both embankment and concrete dams. Seepage flow from embankment body is also drained by toe drains. The seepage flow measurement is simply governed by measuring the flow rate. Generally, V-notch weirs or Parshall flumes are used for this purpose. The purpose of the weir is to transform the instantaneous water level into the corresponding values of flow (SISGEO, 2005-a). Weirs are simple and inexpensive tools to measure the seepage flow. They are normally installed in most dams. In the absence of them, retrofitting of a weir is a very simple operation. The primary way to determine the amount of flow is to measure the depth of flow and using the relevant calibration graph defined for that weir. The depth of flow on a weir can be measured by a pressure transmitter unit, by level transducer units or manually by staff gauge. Weirs can be automated by using a weir gauge and data can be continuously logged. The main component of a level transducer is a cylindrical weight suspended from the force transducer, which alters the tension on transducer by the change in buoyancy force due to water level fluctuations (Geokon, 2006). A definition sketch of a weir monitor is given in Figure 3.4.

Regular maintenance is required to clean the weir and the canal from sediment and the rim of calibrated mouth from deposit (SISGEO, 2008). The frequency of these cleaning procedures should be determined by considering the chemical and biological composition and the sediment load of the seepage water.

Figure 3.4 Weir monitor (Geokon,2006)

3.5 Instruments for Body and Foundation Movement

3.5.1 Vertical Movements

Body movement can be measured by various instruments. Vertical movements can be determined by providing fixed surface monuments for conventional surveying techniques. Surface monuments should be strong and durable enough to withstand the environmental and human induced effects. Moreover, the monuments should be rigidly anchored to the feature to be monitored. Although the surveying gives accurate results about vertical movements, namely settlement, surveying should be performed periodically by skilled personnel. So manufacturers design and produce settlement gauges, either hydraulic or electronic based, to automate the vertical movement monitoring. Vertical movements for old dams, generally caused by settlement, are normally low. Therefore, routine surveying may be considered satisfactory.

Surface alignment, namely differential settlement of a dam can be monitored by newly developed advanced GPS surface monitoring devices. Stewart and Tsakiri (2001) state that dam surface monitoring with highly precise GPS equipment is quicker and efficient than traditional surveying but the technology is still in developmental stage. Moreover, continuously operating GPS stations are still not economically feasible.

3.5.2 Horizontal Movements

Horizontal movements are divided into two as lateral movements and longitudinal movements. Horizontal movements are not a problem for concrete dams except foundation problems. So the surface alignment monitoring by any method is enough. However, in case of embankment dams, lateral and longitudinal movements result in cracks on embankment, which will yield to piping or internal erosion of embankment material. Horizontal movements are generally monitored by inclinometers, which are widely used in engineering practices to monitor the soil and the structural deformations (SISGEO, 2005-b). The first element of inclinometer is the inclinometer casing which is made of plastic or aluminum and embedded into the embankment. The movement of embankment causes some kind of deformations on the casing. A typical view of an inclinometer casing is given in Figure 3.5.

Figure 3.5 ABS and aluminum inclinometer casing (SISGEO, 2005-b)

Inclinometer casing can also be equipped with magnetic settlement targets so that the vertical movement of embankment or soil can be monitored. Although it is not a general application, inclinometer casing can be installed in horizontal direction on the foundation to monitor the differential settlement of ground.

The second element of an inclinometer is the inclinometer probe, which has four wheels for tracking the groove of casing. While following the grooves, the servo-accelerometer sensor group measures the deviation along the plane of probe wheels. Readout unit is the last element of an inclinometer system. Figure 3.6 shows a sample view of an inclinometer probe and readout unit.

Figure 3.6 Inclinometer probe and readout unit (SISGEO, 2005-b)

3.5.3 Rotational Movements

Rotational movements can be best monitored with the use of direct and invert pendulums in concrete dams. Direct pendulum is made of a steel wire anchored in the upper part of the structure and ballasted at the bottom by a proper weight (SISGEO, 2006). Invert pendulums work according to the same principle but the wire is tensioned by a float in a tank filled with fluid (SISGEO, 2006). Definition sketches for direct and invert pendulums are given in Figures 3.7 and 3.8, respectively. Pendulums can be automated by readout units and the data can be logged for further studies. As mentioned before, pendulums can be easily retrofitted to a concrete dam. However, it requires some kind of vertical opening, e.g. elevator shaft, in the body of the dam (ICOLD, 1992).

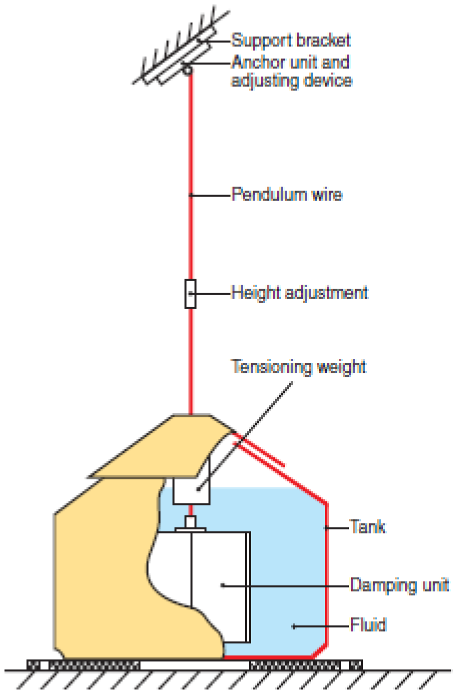

Figure 3.7 Direct pendulum (SISGEO, 2007)

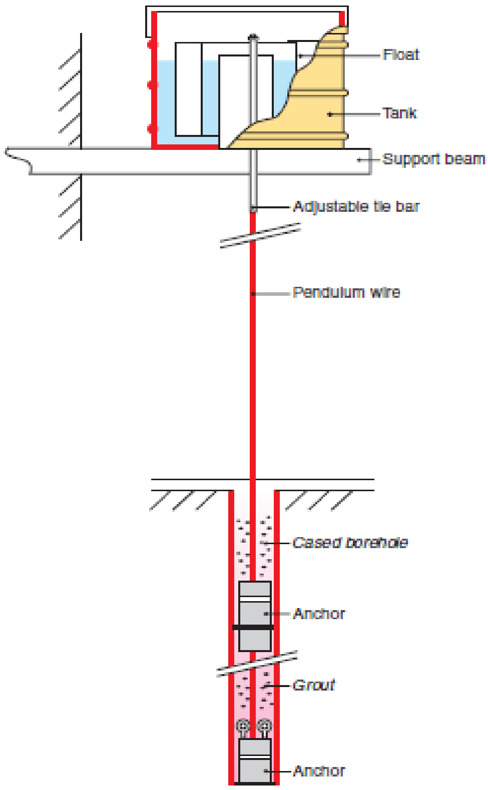

Figure 3.8 Invert pendulum (SISGEO, 2007)

As indicated by ICOLD (1992), if no opening can be assigned, other alternatives should be considered, such as tiltmeters, which can be used in both embankment and concrete dams. Tiltmeters are divided into two categories as portable and fixed ones, generally named as clinometers. Portable tiltmeters consist of a tilt plate, a portable tiltmeter, and a readout device. Tilt plate is a solid brass plate which is to be mounted firmly on dam body in either horizontal or vertical direction. Advantages of portable tiltmeters are that they are economical, easily installed, practical, durable, and accurate (SI, 2007). A view of portable tiltmeter, tilt plate, and readout device is given in Figure 3.9.

Figure 3.9 Portable tiltmeter (SISGEO, 2005-c)

However, when a continuous data logger is required, vibrating wire surface clinometer must be chosen rather than portable tiltmeters. The surface clinometer is permanently attached to the structure to be monitored. It can make measurements on horizontal or vertical surfaces and readings are taken by a readout datalogger or continuously and remotely by data loggers (Geokon, 2007). A view of a surface clinometer is presented in Figure 3.10.

Figure 3.10 Surface clinometer (SISGEO, 2005-c)

3.5.4 Crack and Joint Movements

Crack or joint movements could provide information about the behavior of the single concrete block under different loading conditions. Two types of jointmeters are produced by manufacturers as embedment ones and surface-mounted ones. Figure 3.11 shows a typical view of an embedment jointmeter. Embedment jointmeters are installed during concrete pouring and embedded to the body. So they are not suitable for retrofitting. Surface-mounted ones, such as crackmeters and 3D jointmeters should then be chosen. Vibrating wire crackmeters and jointmeters measure the distance between the two anchors at each block and the movement can be determined with respect to a datum, which is the initial reading at the time of installation (SI, 2006). Most of the vibrating wire crackmeter and jointmeter transducers also include temperature sensors so that the raw data can be calibrated with temperature induced movements.

Figure 3.11 Embedment jointmeter (SISGEO, 2003-a)

3.5.5 Foundation Movements

Invert pendulums are the best option for determining foundation movements as stated in previous sections. The instrumentation can be extended by using borehole extensometers (ICOLD, 1992). Borehole extensometers are used in a borehole in order to monitor the displacements at various depths (SISGEO, 2004-b). Extensometer assembly is inserted into the borehole and then grouted, fixing the anchors to the rock or soil but allowing free movement of each rod within its sleeve. Then, displacement caused by relative movement between the anchors and the reference head are measured (SISGEO, 2004-b). A typical borehole extensometer is shown in Figure 3.12.

Figure 3.12 Borehole extensometer (Roctest, 2005)

3.6 Stress and Strain Monitoring

Stress and strain monitoring can be assessed by total pressure (stress) cells and strain gauges. Stress cells are used to monitor total pressure in soil, rock and concrete at the contact between foundation and the structure (SISGEO, 2003-b). A typical view of total stress cell is given in Figure 3.13.

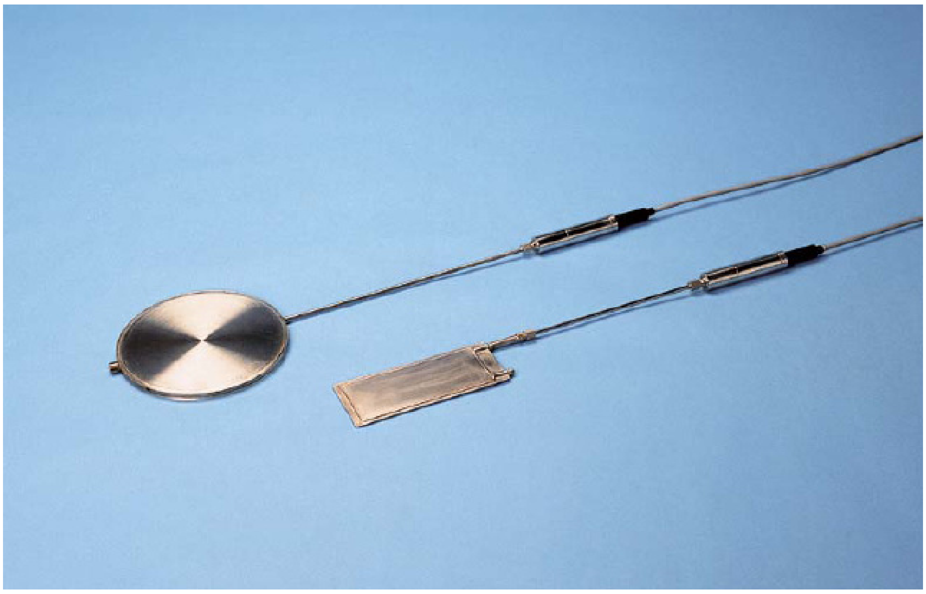

Figure 3.13 Total pressure (stress) cells (SISGEO, 2003-b)

Stress cells are generally embedded to concrete during construction or buried into the embankment. It consists of a deaered oil filled pad, either shaped in rectangular or circular for different applications, connected to a vibrating wire pressure transducer by a hydraulic tube and a data cable for connection to a readout unit (SISGEO, 2005-d).

Strain gauges measure the strain on reinforcement or concrete depending on the installation position. After obtaining the strain (deformation) on the member, the loading can be determined indirectly by using elastic modulus (Young's modulus) and the dimensions of the member. In elastic range, normal stress is directly proportional to normal strain. Vibrating wire strain gauges work with a principle of the movement of two end blocks relative to each other under deformation, thus altering the tension of the steel wire (SISGEO, 2005-d). Strain gages can also be installed in a group to monitor the deflection from different axes by using rosettes. Typical views of strain gages are given in Figures 3.14 and 3.15.

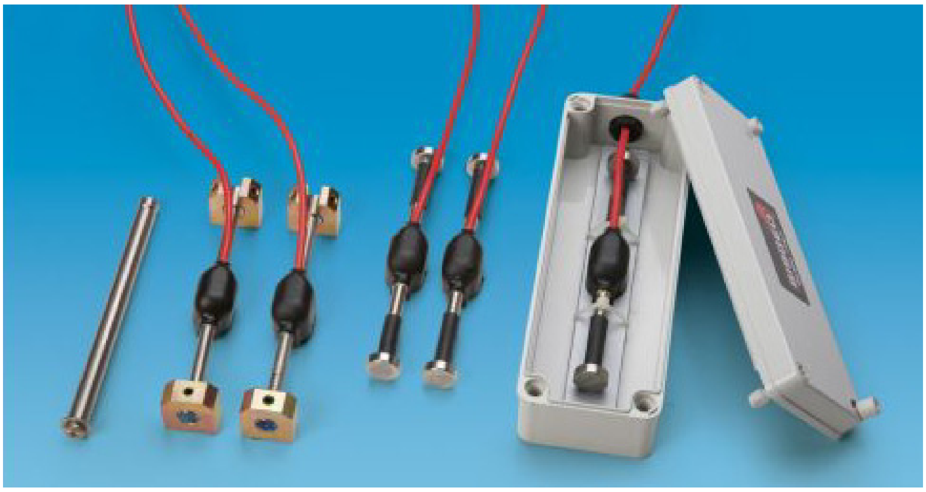

Figure 3.14 Different types of strain gauges (SISGEO, 2005-d)

Figure 3.15 Strain gauge welded to a reinforcement (SISGEO, 2005-d)

Since the nature of stress and strain monitoring requires the instruments to be embedded to body or foundation, it is impossible to retrofit such equipment. So stress and strain monitoring with instrumentation is eliminated from instrument retrofitting.

3.7 Seismic Monitoring

In areas with high seismic activity, strong motion accelerometers can be installed to monitor the earthquake acceleration. Common application is to mount a strong motion accelerometer to a suitable point on crest of a dam. However, in order to measure the effect of seismic activities extensively, it is more suitable to install three strong motion accelerometers. One accelerometer should be mounted on dam crest, the second one on dam body near foundation and the last one should be installed on the left or right abutment as a free-field accelerometer. Strong motion accelerometers cannot be connected to automated data logger systems. So they should be used with their own common triggering unit, which activates the accelerometers during seismic activities, and a recorder to store the earthquake accelerations.

3.8 Readout Units and Automation

Installing dam instrumentation without proper readout units is meaningless. Data readout can be assessed in two ways, manually by using portable datalogger or automatically by using data acquisition systems. Using portable datalogger is fast, simple and cheap way to monitor the interested parameters. They generally have a built-in memory to store an amount of readings and transfer them to personal computer to be processed. Although they have relatively lower initial cost than automated systems, they need skilled personnel to schedule and execute readouts. Therefore, their operation costs are much higher. Automated data acquisition systems generally consists of a data logger, a multiplexer, which increases the number of instruments to be connected to a single data logger and a software to analyze the raw data gathered from instruments.

3.9 Suitability for Retrofitting

In previous sections, basic dam instrumentation with different types of instruments has been introduced. Some instruments need to be installed during construction period, such as total pressure cells, which are instruments measuring the change in internal stress by recording the pressure change of liquid inside the instrument. The installation of these types of instruments is completed during the concrete pouring for concrete dams and filling and compaction period in embankment dams. The instrument should be embedded to the body; therefore retrofitting of such equipment is impossible. Installation of some instruments requires a borehole drilling. Generally, observation wells can be used for this purpose. On the other hand, some instruments, such as pendulums require a shaft or an opening in dam body. Before instrument selection, the suitable galleries, voids, and boreholes should be identified by studying the drawings and site surveys.

Basic types and characteristics of instruments that can be utilized in concrete dams are presented in Table 3.1. Possible locations for installation of these instruments and their suitability for retrofitting are also specified in this table. Suitability is indicated by “S” at the last column; whereas “NS” represents that the equipment is not suitable for retrofitting.

Table 3.1 Basic instruments for concrete dams (Yanmaz and Arı, 2008)

Category	**Instrument**	**Purpose**	**Location**	**Suitability to Retrofit**
Body and Foundation Movement	Pendulum	Rotation (tilt) measurement	Pendulum shafts	S
	Tiltmeter	Rotation (tilt) measurement	Suitable place on concrete	S
	Surveying monuments	Surface alignment monitoring	Crest and surface	S
	Jointmeter	Differential movement	Along joints	S
	Crackmeter	Differential and crack movement	Along cracks	S
	Borehole Extensometer	Foundation movement	Foundation	S

Table 3.1 Basic instruments for concrete dams (Yanmaz and Arı, 2008) (cont.)

Category	**Instrument**	**Purpose**	**Location**	**Suitability to Retrofit**
Pore Water Pressure	Piezometer	Pore water pressure measurement	Foundation	S
Uplift	Piezometer	Uplift measurement	Foundation-Concrete Interface	S
Stress and Strain	Total Pressure Cell	Foundation Pressure	Foundation	NS
Stress and Strain	Total Pressure Cell	Concrete body pressure	Within concrete mass	NS
Stress and Strain	Strain Meter	Foundation Strain	Foundation	NS
Stress and Strain	Strain Meter	Concrete body strain	Within concrete mass	NS
Concrete Temperature	Thermometer, Thermistor	Internal concrete dehydration temperature monitoring	Within concrete mass	NS
Concrete Temperature	Thermometer, Thermistor	Concrete temperature monitoring	Dam surface	S
Seepage	Flow Meter, Weir	Flow quantity	Drainage galleries	S
Seepage	Turbidity meter	Flow quality	Drainage galleries	S
Earthquake	Accelerometer	Earthquake acceleration	Crest of dam	S

A new monitoring system will be proposed to Gökçekaya Dam, Turkey, in the following chapters. Since Gökçekaya Dam is an arch dam, a definition sketch showing the typical instruments to be installed and the installation locations on an arch dam is shown in Figure 3.16.

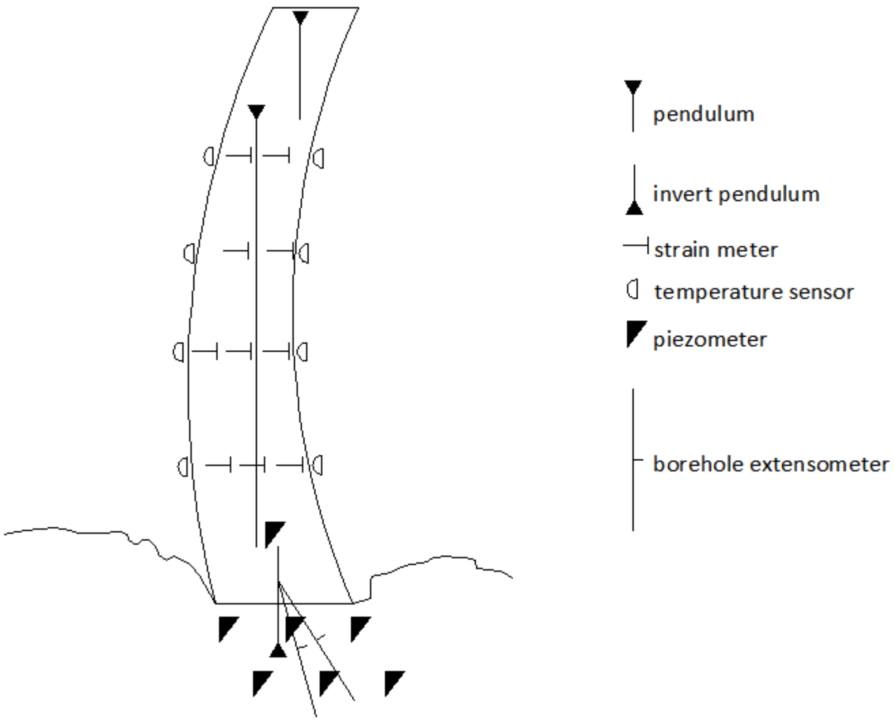

Figure 3.16 An arch dam with instruments (Yanmaz and Arı, 2008)

3.10 Case Histories

3.10.1 General

Previous applications and lessons learned from different cases will be of practical importance in retrofitting practices for existing dams. Several examples on instrumentation of existing dams worldwide are discussed briefly in the following sections. It should be noted herein that a complete instrumentation system retrofitting has not been applied in Turkey yet. To this end, the present study may be one of the pioneering works in this field in Turkey.

3.10.2 Morávka Dam (Czech Republic)

The Morávka Dam in Czech Republic was built in 1967. It is an embankment dam with a height of 39 m. The upstream facing is composed of multi-layer bituminous concrete. During the years of operation, the facing has been deteriorated. After extensive analysis, the facing has been rehabilitated, drainage system has been upgraded and monitoring system, especially seepage rate monitoring has been renewed. The new dam monitoring system has been further upgraded by automatic monitoring and data transfer (Kratochvíl and Glac, 2006).

3.10.3 Talvacchia Dam and Baitone Dam (Italy)

Talvacchia Dam and Baitone Dam are the examples of instrumentation retrofitting to existing dams from Italian practice. Talvacchia Dam has been constructed in 1960, whereas Baitone Dam has been built in 1930. Both of the dams are of gravity type (ICOLD, 1992).

Talvacchia Dam is 78 m high and has a crest length of 226 m. The dam has not experienced remarkable problems since the construction. Its initial instrumentation configuration was composed of a direct pendulum, a surface monitoring system, several thermometers, clinometers, jointmeters, piezometers and leakage monitoring equipment. In 1973, surface monitoring system and pendulum have been automated. Afterwards, in 1986, equipments monitoring dynamic and seismic responses are installed (ICOLD, 1992).

Baitone Dam is a masonry gravity dam with a height of 38 m. Hydrologic data has been collected in close vicinity to the reservoir. Dam leakage and surface movement have been monitored periodically. In addition to the existing monitoring system, one invert pendulum, three piezometers, and an automated data acquisition system have been supplemented (ICOLD, 1992).

3.10.4 Marunuma Dam and Kohmyo-Ike Dam (Japan)

Marunuma Buttress Dam is located 150 km far from Tokyo. The dam is 32 m high and has a crest length of 88 m. Due to freezing and thawing, the deck has been cracked and an increased leakage has been observed from the joints. The dam has been equipped with strain and joint monitoring instruments. Based on detailed inspections, the deck has been covered with a new concrete slab. With the introduction of new stress, strain and joint movement monitoring equipment, the safety level of the rehabilitated dam has been promoted (ICOLD, 1992).

Kohmyo-Ike Fill Dam has been used for irrigation in Osaka. The dam is 26.5 m high and has a crest length of 350 m. During the period of 60 years, seepage had become a severe problem. After intensive analyses, the upstream has been covered with silty clay and new piezometers and weirs have been installed. The measurements verified decrease in seepage after rehabilitation (ICOLD, 1992).

3.10.5 Upper Huia Dam (New Zealand)

Upper Huia Dam in New Zealand, which was constructed in 1920's, had problems with uplift. Experts had suspicion about the effectiveness of the cut-off wall and hence the overall safety of the dam. So they decided to install new piezometers to constantly monitor the pore water pressures beneath the dam. The most suitable positions were selected, boreholes were drilled, and the piezometers were installed. By using the automated system, the measured pore water pressure data gave invaluable information about the effectiveness of the cut-off wall and the necessary remedial works were then studied on that basis (Ahmed-Zeki et al., 2000).

3.10.6 Compuerto Dam and Chandreja Dam (Spain)

Compuerto Dam is 78 m high concrete gravity structure with a crest length of 273 m. A new monitoring system has been designed with some rehabilitative actions. Grout curtains has been extended and all existing drains have been cleaned and bored up to 10 m. Weirs has been installed to monitor the seepage rate. One of the ten weirs has also been equipped with an automatic limnimeter. Each monolith has been instrumented with two piezometers. Three direct and three invert pendulums have been installed. In addition to the invert pendulums, two extensometers have also been introduced. In order to monitor the rotation of the monoliths, seven clinometer bases and one clinometer have been added to the system. For monitoring contraction and expansion of the joints, fourteen thermometers have been installed to one of the monoliths. Finally, an automatic limnimeter has been installed for reservoir level monitoring (ICOLD, 1992).

Chandreja Buttress Dam has been built in 1953. The dam is 85 m high and 236 m long. The initial instrumentation was set for measurement of drainage flow and uplift monitoring. The new system has been proposed for further monitoring of uplift, surface movement, and joint movement. An array of piezometers has been installed under the buttresses. Angular collimation targets have been introduced for horizontal movement monitoring and an array of jointmeters has been implemented for monitoring the relative movements of joints (ICOLD, 1992).

3.10.7 Letten Pumped Storage Plant (Sweden)

The embankment dams at the Letten Pumped Storage Plant, built in 1957, showed a leakage from foundation material. As a result of dam safety assessment, the dams have been rehabilitated by installation of a drainage system in order to control leakage and upgrading the monitoring system (Bergman and Gustafsson, 2006).

3.10.8 Seeuferegg Dam (Switzerland)

Seeuferegg Gravity Dam is located in the Alps. Existing monitoring system consists of a direct pendulum and seepage rate monitoring weirs. In new instrumentation system, additional direct and invert pendulums have been introduced. A new geodetic network has been provided for surface monitoring. A number of standpipe piezometers have also been installed for uplift pressure monitoring (ICOLD, 1992).

3.10.9 Pacoima Dam (USA)

Pacoima Dam is a 113 m high arch dam. The dam has been built in 1928. After 1971 San Fernando Earthquake, vertical contraction joints have been opened resulting in horizontal cracks. Due to the movement of plates, a slight rotation of the dam body has also been noticed. No instrumentation had been provided for the dam initially. After rehabilitation, extensometers have been installed in abutments. A total of 20 piezometers were provided to monitor the uplift pressure. An inclinometer, an array of accelerometers, and a number of thermometers have also been installed at various locations on dam body (ICOLD, 1992).

3.10.10 Guri Dams (Venezuela)

Guri Project, which consists of several concrete and fill dams, had been equipped with more than 1900 instruments during construction. However, a significant number of instruments have been damaged due to aging. In order to satisfy the proper monitoring requirement of the project, a study has been conducted and new and contemporary instruments of various types and purposes have been installed to dams (Ramírez and Noguera, 2006).

CHAPTER 4

CASE STUDY: GÖKÇEKAYA DAM

4.1 General Information about Gökçekaya Dam

Gökçekaya Dam is located on Sakarya River, 60 km northeast of Eskişehir and 50 km downstream of Sarıyar Dam. Figures 4.1 and 4.2 show satellite images covering the dam site and its close surrounding. The dam site lies between the second and third earthquake zones. An earthquake zone map showing the exact place of dam site can be observed in Figure 4.3. It is the first concrete arch dam of Turkey, whose construction took place between 1967 and 1972.

The purpose of the Gökçekaya Dam is electricity production. The dam is 159 m high from the foundation, 115 m high from the thalweg and it has a crest length of 479.66 m. A concrete volume of 650,000 m^3 was used in the construction of the body. The reservoir lake has a surface area of 20 km^2 with maximum volume of 910 million m^3. Gökçekaya Hydroelectric Power Plant (HEPP) has three units with an installed capacity of 278.4 MW (3*92.8 MW). The HEPP of Gökçekaya Dam generates annual electrical energy of 400,000,000 KWh (DSİ, 1974).

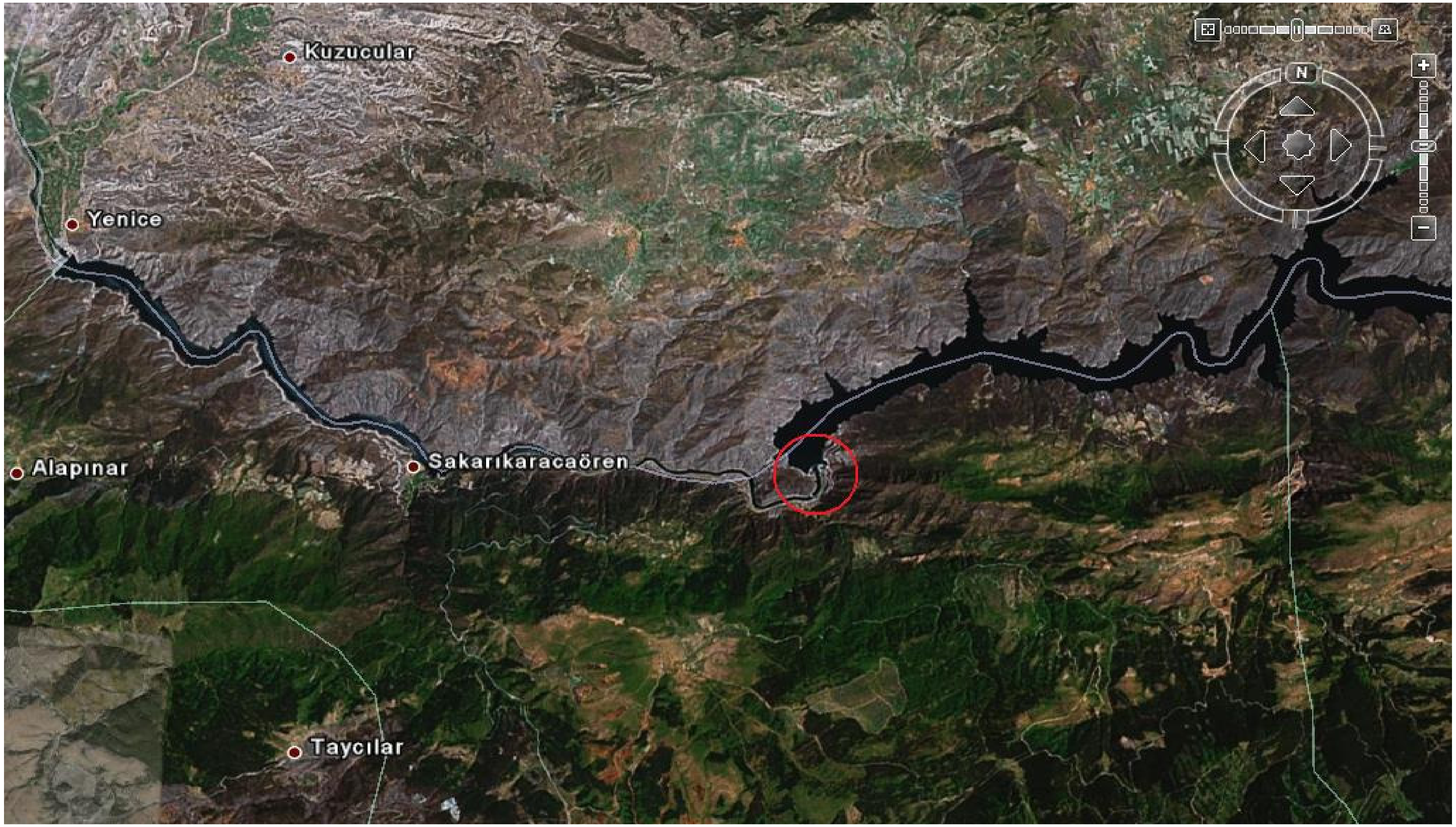

Figure 4.1 A satellite view of Gökçekaya Dam (Google Earth, 2008)

Figure 4.2 A closer view of Gökçekaya Dam (Google Earth, 2008)

In 1950, a number of studies had been prepared about the usage of Sakarya River and some possible axes for dam construction had been determined. After the completion of Sarıyar Dam, the suitable site for the construction of a new dam had been started. After numerous researches, an axis in Gökçekaya region was found to be suitable for all types of dam construction (DSİ, 1974).

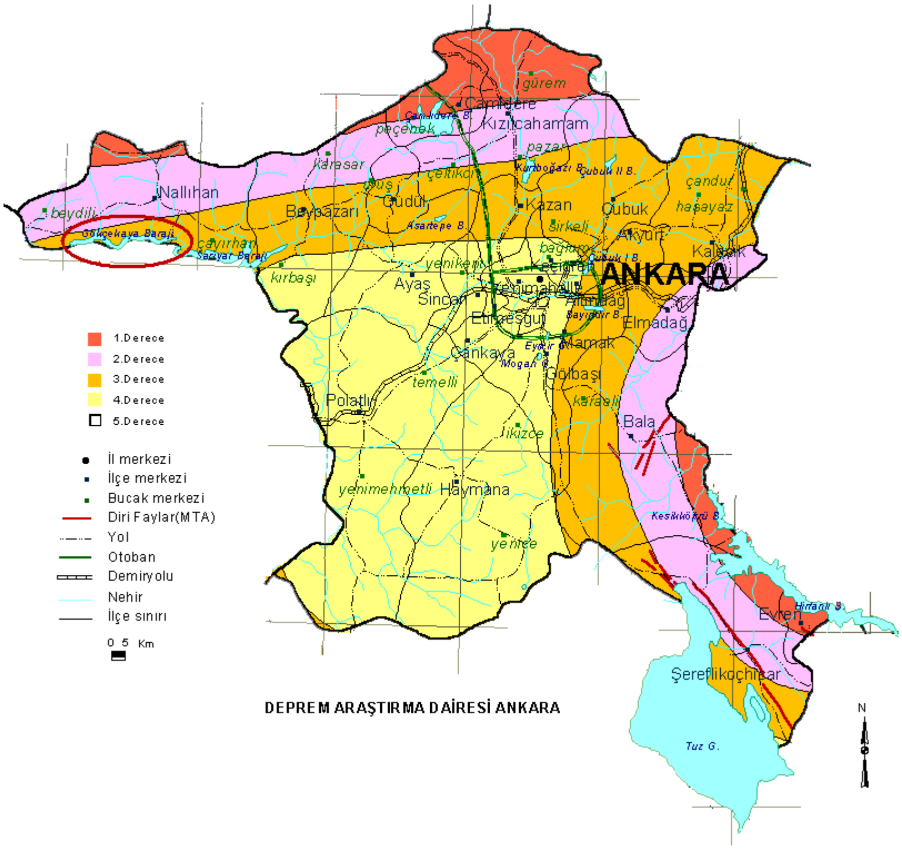

Figure 4.3 Ankara Province Earthquake Map showing Gökçekaya Dam

After extensive analysis of the application of different dam types, such as concrete gravity, rockfill and concrete arch, the most cost-effective solution was found as concrete arch. So Gökçekaya Dam had been constructed as double-curvature, variable-radius, and variable-center concrete arch (DSİ, 1974). Its spillway had been constructed on a separate valley at the right side of the dam (see Figure 4.4)

Hydrologic Information about Gökçekaya Dam is as follows (DSİ, 1974):

Catchment area of Gökçekaya Dam	:	44,650 km^2
Surface area of the reservoir	:	1,650 km^2
Average annual precipitation	:	450 mm
Average flow rate	:	70 m^3/s
Minimum recorded daily flow rate	:	20 m^3/s
Maximum recorded daily flow rate	:	790 m^3/s
Average annual runoff volume	:	$2,500 \times 10^6$ m^3

There were two important fault lines at the dam site, which directly affected the body construction. The first one, which is on the right side, directly passes through the dam body under blocks 0 and 1 on high elevations. The other one passes along the dam axis starting from the left thrust blocks and leaves the dam axis by passing under blocks 9 and 10 (see Figure 4.5). Treatments of these faults were completed by removing the debris up to a predefined level and then filling the fault by concrete (DSİ, 1974).

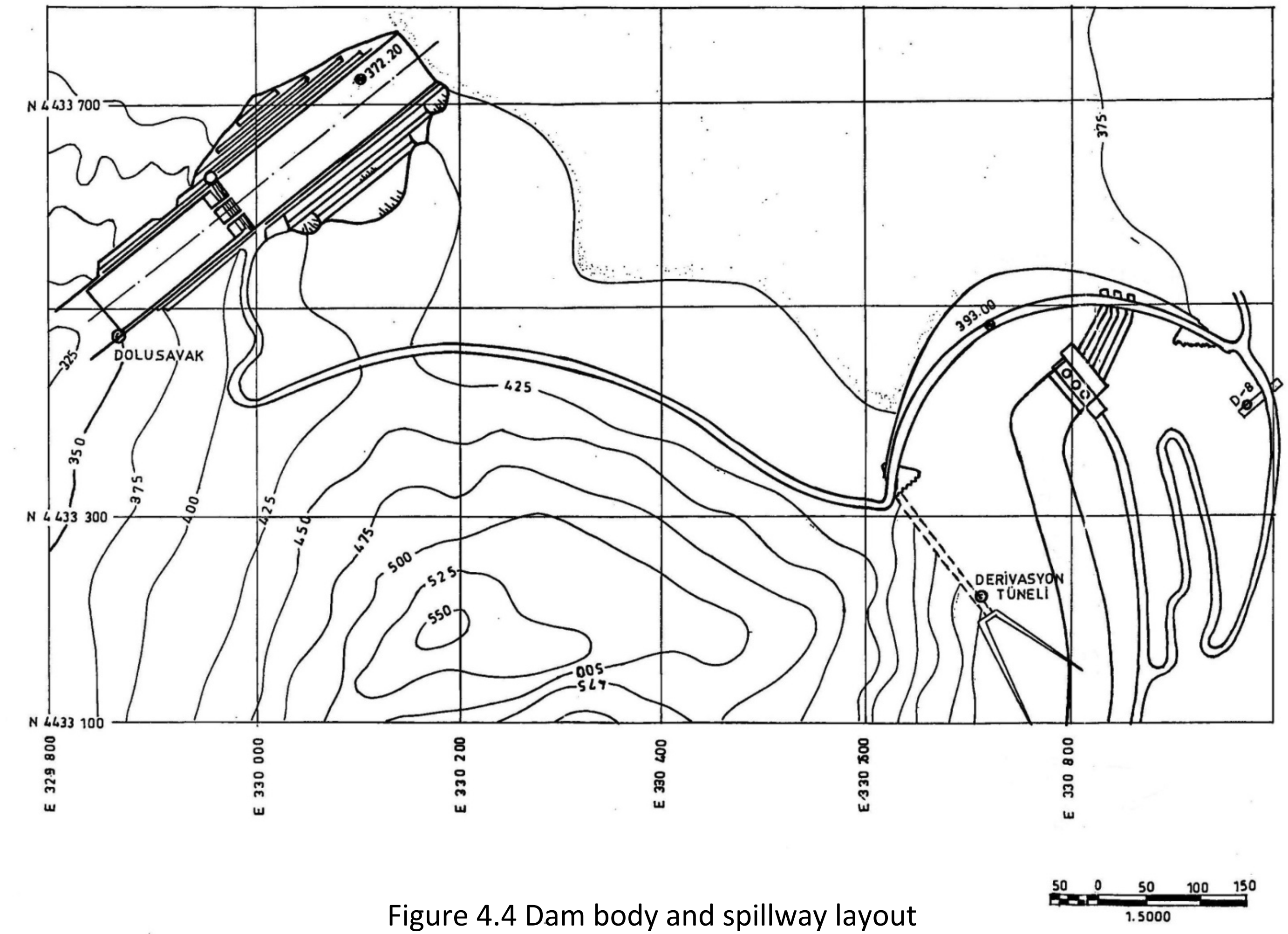

Figure 4.4 Dam body and spillway layout

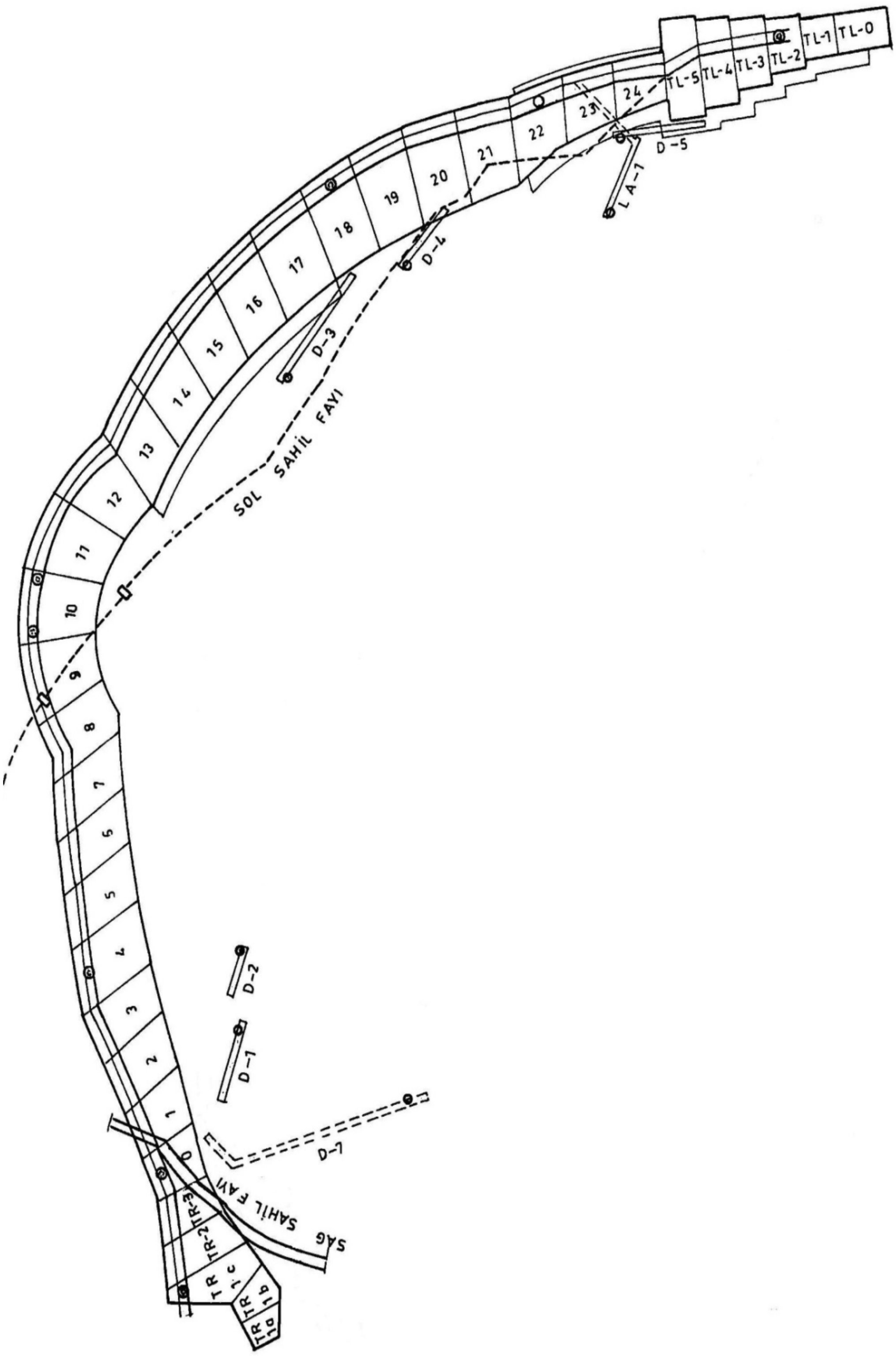

Figure 4.5 Block arrangements of Gökçekaya Dam

4.2 Site Investigation

4.2.1 General

Gökçekaya Dam site has been visited on 17 April 2008 together with the DSİ (State Hydraulic Works) officials. During the site visit, there was a repair work on hydro-electric power plant such that the electricity generation had been suspended. A general view of the dam from the downstream is shown in Figure 4.6.

Figure 4.6 General view of the dam

4.2.2 Left Abutment

The investigation has been started in an inspection gallery (LA-1) on the left abutment at relatively high level (see Figure 4.7). This gallery extends towards the left abutment and the access road to the dam crest passes just above. The inside of the gallery was full of debris and concrete wastes. On the walls and ceiling of the gallery, traces of leaked water could be easily noticed. Water was seeping through the ceiling, starting from the construction joints which were deteriorated as a result of seeped water. Some traces were dry, some others were leaking. Some calcium deposits were noticed, possibly related with the surrounding earth material (see Figure 4.8).

Figure 4.7 Left abutment inspection gallery (LA-1)

Figure 4.8 Seeping water through the ceiling of gallery

As a result of low seepage flow, collector on the base of the gallery was dry. At the exit of the gallery, a triangular weir exists to measure seepage flow; however it was full of debris and wastes. So the possible future measurements cannot be taken correctly. The condition of the aforementioned weir is shown in Figure 4.9. Some diagonal cracks were also discovered on the walls of the aforementioned inspection gallery as shown in Figure 4.8.

Figure 4.9 Accumulated debris at the weir

4.2.3 Dam Body

A relatively large leakage has been noticed on the downstream face of the dam, which was indicated by a red circle on Figure 4.10. It is believed that the water leaked through that construction joint for a long time because of some algae formation along the joint on the face of dam body.

At the dam crest, there was no equipment to measure and log the reservoir level. A graduated stick which was possibly mounted during the construction was observed at the upstream face. A view from the graduated stick of Gökçekaya Dam is given in Figure 4.11.

Figure 4.10 Crack on the face of the dam

Figure 4.11 Reservoir level measurement

On the dam crest, a number of surface monuments exist which were in good condition. A sample view from one of the surface monuments is presented in Figure 4.12. On downstream face of the dam, a number of surface markers exist at different levels whose surrounding area was painted to white in order to find them easily. However, the white paint seems deteriorated greatly. DSİ officials noted that no systematic dam surface monitoring have been taken by conventional surveying techniques on this dam.

Figure 4.12 A surface monument

The inspection gallery of Gökçekaya Dam begins from the highest body elevation at side abutments. By following the foundation line, the gallery continuously lowers to the thalweg level of the dam by steep stairs. At the entrance of the gallery from the power plant access (Block 18), a crack on the base of the gallery, perpendicular to the dam axis was observed as shown in Figure 4.13. These types of lateral cracks are generally treated as an indication of differential settlement due to foundation movement. The triangular weir (B-18) had a flow with some debris accumulation (see Figure 4.14). The stairs were wet as shown in Figure 4.15 and the measurement ports of strain-meters were all corroded and were out of order (see Figure 4.16). On both the upstream and downstream walls along the construction joints, water leakage had formed a lime accumulation (see Figures 4.17 and 4.18). The ceiling had these traces of water leakage on several places as shown in Figure 4.19. Leakage from construction joints can be evaluated as a sign of joint movement. A red mud was also noticed on the side canal, which was observed in several drain holes on the stairs of the dam gallery. The traces of the aforementioned mud in two different drain holes and in drain canal are shown in Figures 4.20, 4.21, and 4.22, respectively.

Figure 4.13 Crack on the base of the gallery

Figure 4.14 Triangular weir (B-18)

Figure 4.15 Wet stairs

Figure 4.16 Corroded strainmeter ports

Figure 4.17 Calcium deposits along a construction joint

Figure 4.18 Calcium deposits

Figure 4.19 Calcium deposits at the ceiling of gallery

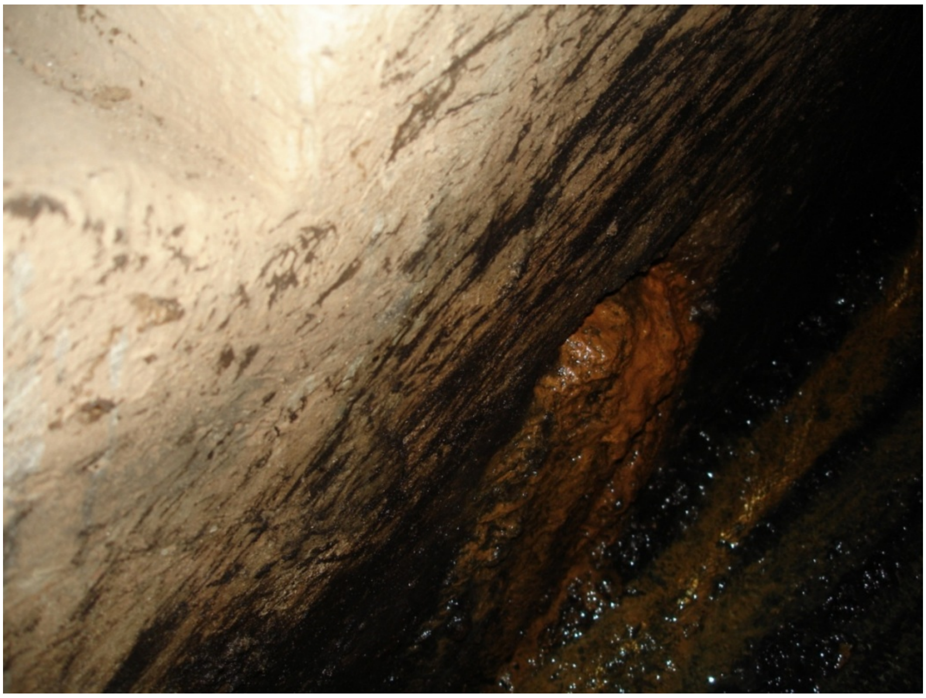

Figure 4.20 Traces of red mud

Figure 4.21 Red mud in a drain hole

Figure 4.22 Traces of red mud in the drainage channel

4.2.4 Right Abutment

On the right abutment, around the right thrust block, eleven drain wells exist as shown in Figure 4.23. They have been used as a standpipe piezometer. One of them is out of order due to a subsidence. A table of readout of these drain wells recorded on 04.06.2001 by DSİ officials implied that three of these wells had no water, possibly due to clogging.

Figure 4.23 Standpipe piezometers on the right thrust block

4.2.5 Spillway Site

The spillway of Gökçekaya Dam is a separate structure, on the right side of the dam. It has a length of 62 m having a crest elevation of 376.50 m. Three radial gates decrease the net length of the spillway to 48 m.

These gates are supported by two intermediary piers. The energy dissipation of the spillway is achieved by a free jet, directed to the air by a deflector bucket. The face, chute, and the deflector bucket of the spillway of Gökçekaya Dam can be seen in Figure 4.24. Radial gates are operated by a chain mechanism rather than using a modern hydraulic system. The operating mechanism and the access bridge of the spillway of Gökçekaya Dam can be observed in Figure 4.25.

Figure 4.24 Gökçekaya Dam Spillway

The chute and deflector bucket sections of the spillway were heavily covered by plants as seen in Figure 4.24, which would significantly decrease the capacity of the spillway during operation. DSİ officials stated that the gates have not been opened since 1982. Therefore, the probability of failure of the mechanism during an emergency situation may be high (see Figure 4.25). Such an operational failure would create severe situation which may lead to a catastrophic event. An unexpected overtopping may occur as a result of this operational deficiency. It would, therefore, not only create a flood wave through the downstream, but also greatly erode the concrete body and the foundation of the dam. Moreover, the increased water level also leads to a greater seepage head and thus results in increased foundation seepage rate. Furthermore, appurtenant structures, such as three cranes on the crest of dam and the HEPP would be seriously damaged.

Figure 4.25 Spillway gate mechanism

Deficiencies observed in and around the dam body will be interpreted and possible remedial actions to be taken and instrumentation alternatives with cost calculations will be proposed and discussed in Chapter 5.

CHAPTER 5

REHABILITATIVE RECOMMENDATIONS

5.1 General

Items exhibiting problems regarding the dam safety were mostly determined during the investigations at Gökçekaya Dam site. The information gathered from DSİ officials was also helpful. In this study, some remedial actions to be taken in order to rehabilitate the dam will be introduced and discussed according to their technical and economical feasibility. Being an aged structure, Gökçekaya Dam has confronted several problems up to date. Lack of routine inspection, monitoring, and maintenance also quickened the overall deterioration. In order to be consistent on rehabilitative and instrumental recommendations; not only the present situation, but also the history of the dam should be considered.

5.2 Deficiencies of Gökçekaya Dam

One of the important deficiencies observed in the dam body will be introduced in this section. On the basis of an unpublished DSİ report, the following information had been obtained about this occurrence.

According to dam management, on 15 August 1983 at six o'clock, a mud flow coming from sluiceway drainage valve, which clouded one-third of the downstream lake, had been observed. After one week, again a mud flow had been started from various drain holes in the floor of the gallery. The inspection by experts started on 31 August 1983. The first evidences were listed by the inspection team as the observed mud flow from drain holes 16 and 26, mud residue on the stairs of inspection gallery, opening of some construction joints around drain holes 32 and 44, the non-functional monitoring equipment, no flow measurement on drain canals and the clogged drains. Since the original drawings of the dam body are not available, the exact locations of the aforementioned drain holes are not known. The mud observed in galleries and in drain holes was clay containing some organic materials with light-brown color. Physical and chemical studies on both the mud found in drain holes and the material from the reservoir bottom showed identical properties, so the origin of the mud was assumed to belong to the reservoir bottom.

During the investigation of Gökçekaya Dam, the residue of the mud, pointed out by the DSİ inspection team twenty five years ago, has been still observable in drain holes in the gallery. Several drain holes have also been observed to be clogged. Ineffective drain holes at the gallery may lead to pore water pressure built-up by increasing the uplift pressure at the foundation level which decreases the dam safety.

It is also noticeable that the drain canals and measurement weirs have been affected by accumulated debris and algae formation. Chemical and biological composition of seepage water and the sediment load greatly influence the accumulation frequency of such drain canals and weirs. Corrosion of some weirs has also been observed. With the present situation, the weir readouts would significantly deviate from actual flow values.

It was learned from an unpublished document, recorded by DSİ in 2001 that, a total number of 17 weirs were installed on Gökçekaya Dam body, spillway, and buttress walls. Inside the dam body, there exist a total of 7 weirs; two of them are rectangular weirs and the others triangular. These weirs are placed in galleries of blocks 0, 4, 10L, 10R, 18, 22 and in right thrust block 1C. The weir in the right trust block was marked as cancelled, whereas the others are still operable. The other ten weirs are placed on side slopes, spillway, and diversion tunnel. One of these ten weirs is faulty, one is inaccessible and the other four weirs are dry. The weir D-8 was marked as faulty in a readout table which should be further evaluated whether the problem can be eliminated or not. Also the dry weirs should be observed whether or not they are dry in all seasons. After an evaluation of the weirs by the helps of drawings and recent readouts, the active and usable weirs have been determined in order to be rehabilitated and instrumented. These are weirs 0, 4, 10L, 10R, 18 and 22 in dam body, weir TL-2 in left thrust block, and weirs LA-1, D-4, D-3 and D-2 in inspection galleries.

Around the Gökçekaya Dam body, the only place to measure the pore water pressure is the standpipe piezometers, mounted around the right thrust block. The exact reason of the installation of such a group of standpipe piezometers is unknown but it is assumed that the main reason might be the monitoring of seepage due to the right fault line. There exist a total number of eleven piezometers in that group and by referring to a readout table prepared by DSİ which belongs to year 2001, a piezometer was out of order due to a subsidence and also three of them did not contain any water. In addition to those on the right thrust block, active five standpipe piezometers also exit around the spillway. After detailed studies on these standpipe piezometers, four wells at the left thrust block, namely wells 143, 145, 147, and 149 have been selected to be retrofitted, with reference to their location and condition.

Cracks and opening of construction joints generally indicates the potential problems and should be monitored as mentioned before. The causes of unusual behaviors should be eliminated. As indicated before, a large leakage on the downstream face of Gökçekaya Dam has been noticed. The water seeps throughout the horizontal construction joint of block number 1. After examining the dam related documents and drawings, it was detected that the right abutment fault passes along the foundation of block 1 and continues to the upstream of the dam. The reason of the horizontal construction joint opening at block 1 might be the movement of fault. During investigation in the gallery, it was observed that several vertical joints were also exhibited unexpected opening and the water leakage had left noticeable traces of calcium and lime deposits on both the upstream and downstream walls of the gallery. Moreover, the joints on the ceiling have also opened and leaked a significant amount of water.

Spillways play a big role on the safety of a dam but generally their inspection and possible repair demands are omitted. However, when a flash flood comes, defective mechanisms and/or deteriorated spillway basins will decrease significantly the discharge capacity of a spillway which may lead to a more catastrophic event like overtopping. During the investigation of the spillway of Gökçekaya Dam, it was observed that the chute and the deflector of the spillway have been heavily covered by debris, some plants, and even a number of trees. Naturally, these roughness elements will greatly decrease the design discharge capacity of the spillway and also disturb the flow regime which promotes damages.

DSİ officials stated that the spillway and the mechanism of the spillway gates have not been used since 1982. It seems that they are suspicious about whether or not the gates will operate properly during a subsequent flood. This is again very risky in view of the safety against dam overtopping possibility.

5.3 Rehabilitative Repair Works

In previous section, deficiencies observed in Gökçekaya Dam have been introduced. Despite of being a milestone in dam history of Turkey; the remaining economical life of Gökçekaya Dam is still around 40 years. This value is roughly estimated. Since Gökçekaya Dam is located downstream of Sarıyar Dam, the upstream sedimentation rate was assumed to be lowered, which may promote the remaining life of the reservoir. Due to lack of routine inspection and necessary repair works, the expected safety performance of Gökçekaya Dam has been reduced. So considering the present situation, it is essential to perform some rehabilitative repair works. Last but not the least, monitoring the performance of the accomplished work and the whole structure by retrofitting modern instruments is logical and indispensible. In this section, the possible remedial actions to be taken to correct the aforementioned deficiencies are introduced.

It has been stated before that the drain holes in the gallery had been drilled during construction stage to reduce the uplift pressure by releasing the pore water pressure beneath the dam body. But as also mentioned before, these holes are greatly clogged with the sediment of seepage water caused by possibly washing out of foundation fissures. It is obvious that clogged drain holes cannot perform their function, thus increase the overall failure risk of the dam. In order to relieve the uplift pressure built-up at the foundation of the dam, drain holes, especially the clogged ones should be cleaned from accumulated debris, gravel, and sediment by using pressurized water. If the application of pressurized water cannot completely clean the drain hole because of the solidification of the accumulated debris, they should be drilled again by suitable drillers cautiously by considering the depth of drain hole during construction and not disturbing the treated foundation. The cleaning process is also required for installation of piezometers. Retrofitting of such equipment deactivates a drain hole which seems to create a decrease in overall discharging capacity at a first glance. However, considering the improved capacity of remaining drain holes, the effect of blocked ones due to equipment installation is negligible.

Seepage in concrete dams generally originates from the released water from foundation in order to reduce the uplift and the leaked water from construction joints and cracks. The seepage is naturally a function of reservoir level at a given time, so it should be monitored precisely and then evacuated to the downstream. Seepage flow rate monitoring is generally assessed by use of weirs. However, measurement with weir is affected by the flow regime and dimensional change in canals in both the upstream and downstream of the weir. In order to accomplish correct weir readout, the canals and the weir should be free of dirt, debris, and algae formation. Thus, a systematic cleaning application should be designated. As explained before, intense accumulation of sediments and algae formations have been observed in weirs of Gökçekaya Dam. So, before installing any seepage flow measuring instrumentation, the weirs and canals should thoroughly be cleaned. The cleaning is generally done by pressurized water; however some solvents which are suitable for concrete should also be used in order to dissolve the algae formation from the walls of canals and weirs. It should be noted that the cleaning of drain holes should be completed before the cleaning process of weirs and canals. Finally, after cleaning the weirs and canals, seepage rate can be monitored precisely. The instrumentation options for seepage monitoring will be explained in the following sections.

In Gökçekaya Dam, many joints indicate a significant leakage due to the relative movement between blocks. Especially, the leakage through the horizontal joint on Block 1 can be observed easily. Leakage through the joints exhibit a great risk to the overall safety level of the dam. Therefore, after an extended inspection, the condition of joints should be inspected and repaired urgently by means of proper and economical ways. Ohio Department of Natural Resources (OhioDNR, 1999) states that the two main objectives to repair a crack are to provide structural bonding and stop water. For structural bonding, epoxy injection can be used. However, they should only be applied to cracks which are not active, whereas since the urethane sealants are flexible, they can be used for cracks that are still active (OhioDNR, 1999). After extensive inspections of cracks, they should be treated by first applying urethane sealant for watertightness by divers and then epoxy injection for structural stability. After these rehabilitative applications to cracks and opened joints, they should be constantly monitored by using vibrating-wire jointmeters for determining the performance of repair works and observing the future behavior of these joints. Instrumentation options for joint movement monitoring will be discussed in following sections.

The condition of the spillway and the gate mechanism has already been discussed. In order to decrease the risk of overtopping due to poor physical conditions at the spillway site, some urgent repair works should be carried out on the spillway and the gate mechanism. First of all, plants and debris covering the spillway chute and deflector bucket should be removed carefully from the concrete face. In order to end the root activities of plants completely, some chemicals, which will make them ineffective, can also be used. Secondly, after the cleaning works of removed plants and debris, the concrete face should be cleaned with pressurized water in order to remove the spalled concrete particles and stone chips.

After cleaning works, the cracks or voids on concrete body of the spillway chute and the deflector bucket should be filled and leveled by using a proper material, such as structural epoxy or specialized repair concrete. The best method can be selected considering the structural and economical requirements as a result of detailed inspection of the condition of concrete after the cleaning works. It will be a good practice to accomplish these inspections and repair works in a scheduled manner, especially after floods.

The gates are probably the most critical elements of controlled spillways. Their periodical inspections, to be made by skilled personnel, should be carefully tracked in order to keep the gates and the mechanism in reliable and maintainable condition. In Gökçekaya Dam, the condition of the gates is unknown and there has been no inspection since 1982. Employment of skilled personnel is of worthy to control the conditions of the gates and the gate mechanisms. After detailed inspection, the necessary repair works should follow. The details of the required mechanical repair works are out of this study. After the inspection and repair works, a strict maintenance schedule should be established for the future activities. These considerations are out of the scope of the thesis.

5.4 Application Options and Related Costs of Instruments

5.4.1 General

Instrument application design of a dam requires extensive structural, hydraulic and geotechnical analysis, prediction of weak zones, determination of best suited instruments, cost optimization and experience. Thus, the amount of required data is high. However, considering an existing dam, the possibility of the loss of design information, such as the assumptions, applied methods and drawings was very high. So, an experienced designer should intensively inspect the present condition of the dam. In conservative approach, it is believed that after some time, the items to be monitored are stable in terms of measured values and further monitoring, especially instrument retrofitting is unnecessary, which lead to an additional cost. However, recent studies showed that the aging of dams results in more damage than expected. In order to monitor and predict faulty elements and conduct a rehabilitative future repair works, it is necessary to install modern and precise equipment to an aged dam also. In the following sections, the alternatives of instrument retrofitting will be discussed for Gökçekaya Dam, with reference to available technical information, data, and economy. The proposed basic instruments are piezometers, automated weirs, surface jointmeters, surface clinometers, and finally strong motion accelerometers. In this study, all equipment is to be planned considering a central automated data acquisition system in order to obtain contemporary results. The calculations have been achieved by considering several items, such as types of instruments, installation locations, the cable lengths to data logger from the individual instrument, required cable sag, and the length of the tray carrying cable safely. The location of the multiplexers and dataloggers are selected as the entrance of the gallery from the hydroelectric power plant for easy access. Furthermore, all length calculations have been completed accordingly. The required sag of the cables has been chosen according to the approach used by DSİ as 15% of the total length of cable (USBR 1987). The equipment costs have been taken from the cost table presented in Appendix A. After computing the details and the quantity of the individual instrument items, cabling, cable tray and instrument costs have been determined and total costs of each option are attained by summing them up.

5.4.2 Piezometer Options

In Gökçekaya Dam, three options for piezometer applications were proposed. Definition sketches are presented to demonstrate the instrumentation configuration for piezometers in Figures 5.1-5.3 In Option 1, the total number of required vibrating wire piezometers was determined as 35. On the right thrust block, TR-2 and TR-3 blocks (Figure 4.5) were selected and one of their drain holes was considered to be utilized for piezometer installation. Similarly, on the left thrust block, 4 blocks, from TL-2 to TL-5, were chosen to be monitored. Since this option concentrates on the amount of data gathered, all blocks from 0 to 24 were equipped with piezometers to monitor the change of pore water pressure between the sequential blocks. In this option, the existing standpipe piezometer holes on the right abutment were also evaluated. By considering their condition and location, wells 149, 147, 145 and 143 have been added to the monitoring system by installing piezometers. In the computations, the average depth where piezometers are embedded is taken as 10 m. The possible costs of drilling and backfilling are ignored.

In Option 2, the existing right abutment drain holes have been excluded because of the consistent results of manual well reading up to date and the closeness of the piezometers on blocks, TR-2, TR-3, and 0. The other installations in dam body have been taken as the same as Option 1. Total number of instruments in this option is 31. The Option 3 was suggested as a lower-cost alternative to Option 1 and Option 2.

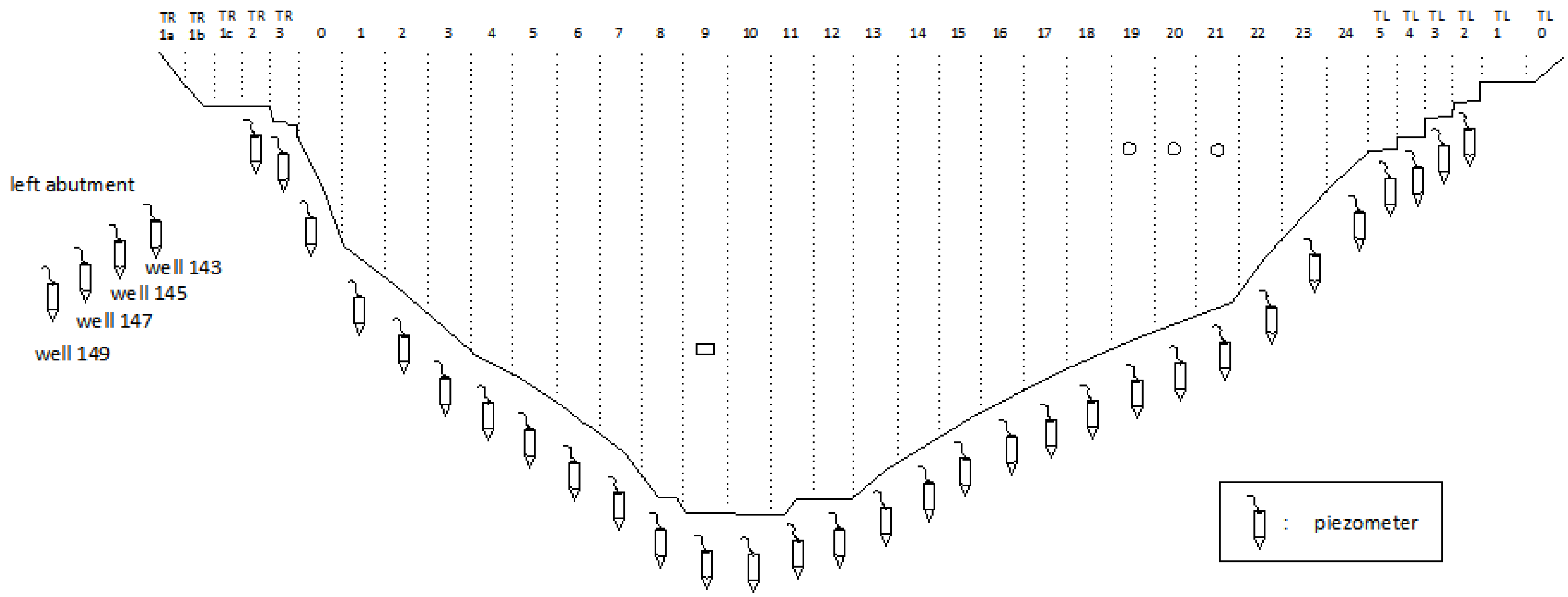

Figure 5.1 Piezometer-configuration in Option 1

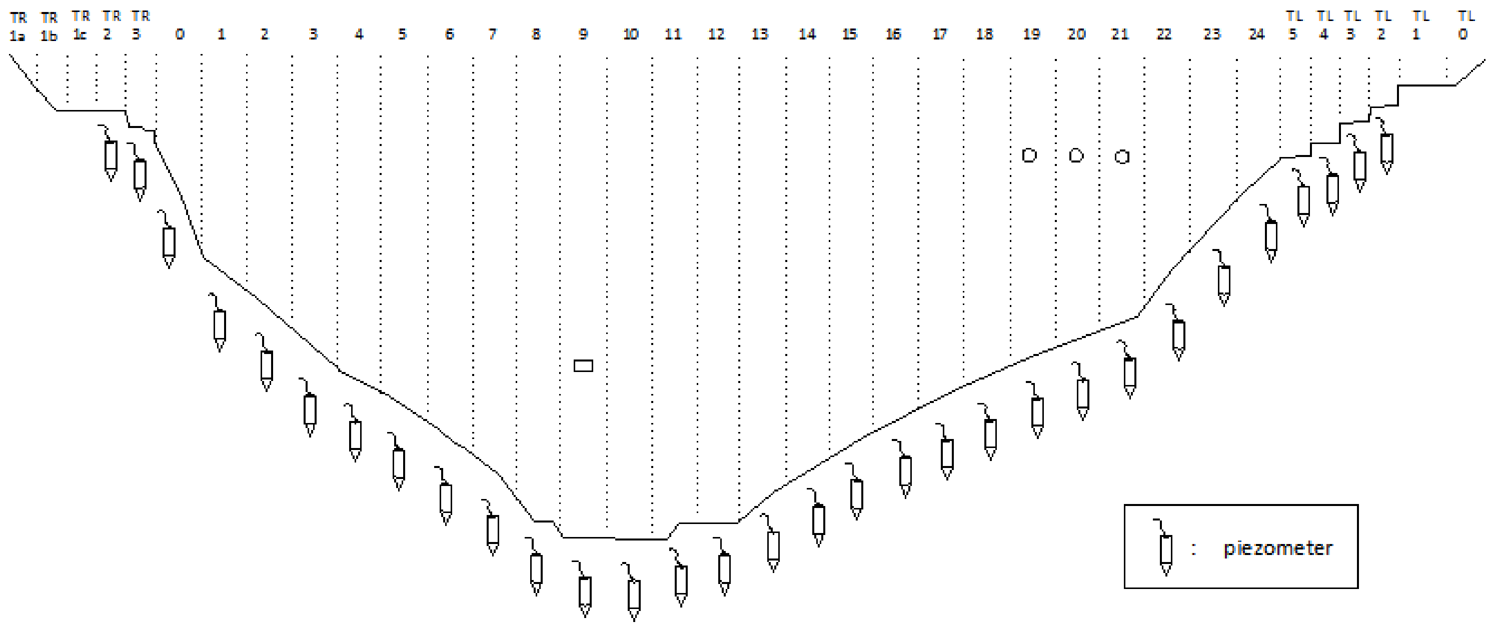

Figure 5.2 Piezometer-configuration in Option 2

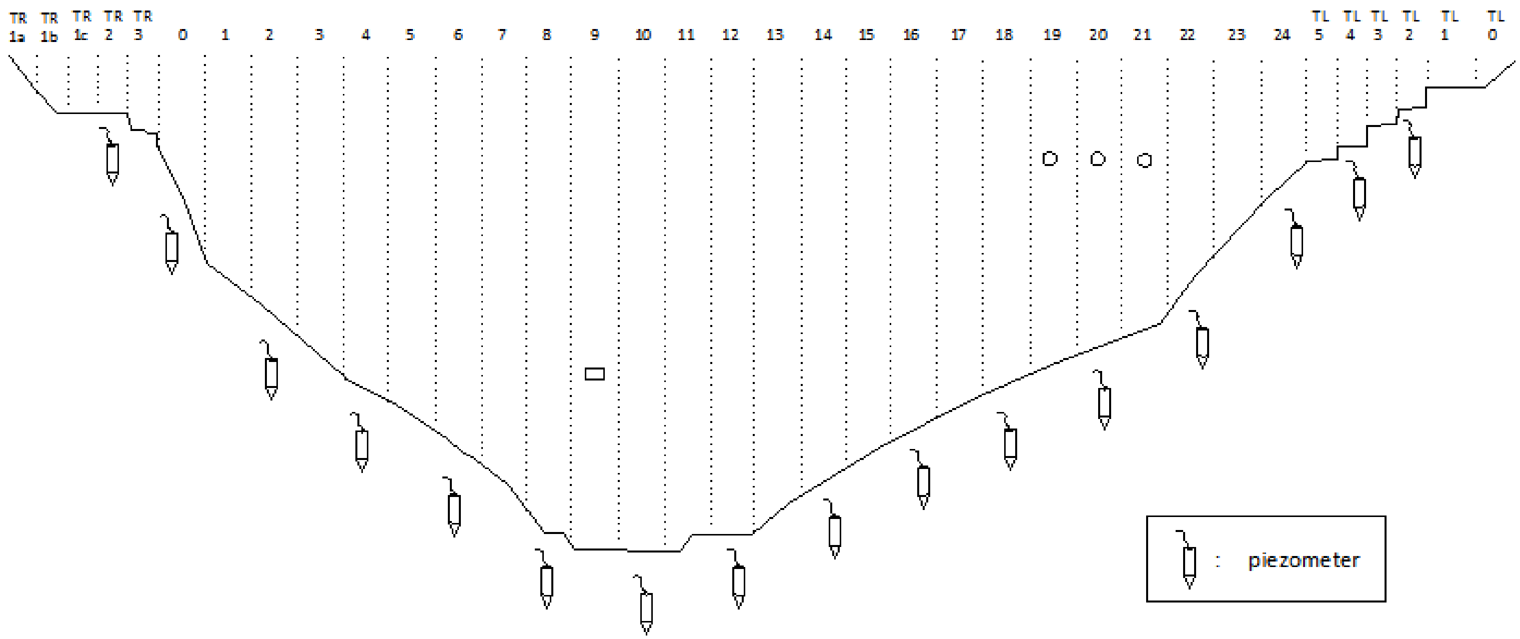

Figure 5.3 Piezometer-configuration in Option 3

In Option 3, the piezometers on the existing right drain holes were similarly discarded as in Option 2. Moreover, unlike the previous options, only one piezometer for two blocks was utilized. So the blocks with even numbers have been equipped with piezometers. These cost-cutting actions reduce the total number of instruments to 16.

In Table 5.1, the costs of three piezometer options in European currency unit (€) are compared according to their total costs. Cost items are divided into three as the cost of instrument, cost of cable, and the cost of cable carrier. The low-cost option has a total cost of nearly the half of Option 1. Detailed tables concerning cost calculations are presented in Appendix B.

Table 5.1 Comparison of the costs of piezometer options

Option	**Cost of Instrument (€)**	**Cost of Cable (€)**	**Cost of Cable Carrier (€)**	**TOTAL COST (€)**
1	14,000.00	32,848.11	13,925.60	**60,773.71**
2	12,400.00	24,821.61	11,390.60	**48,612.21**
3	6,400.00	13,081.65	11,390.60	**30,872.25**

5.4.3 Seepage Monitoring Options

Gökçekaya Dam consists of a number of seepage measurement weirs, either triangular or rectangular. These weirs had been made of steel and embedded to the concrete bed of the seepage canals. The measurements had to be conducted by measuring the flow depth above the weir manually and determining the corresponding flow rate by using calibration curves. After the evaluation of past reading logs, it is clear that neither the monitoring has been properly scheduled nor the readings had the required precision. However, the logs gave invaluable information about the condition of weirs, such as accessibility to weir area and the quality of information that can be gathered from individual weirs. By evaluating these data, the weirs to be instrumented have been determined. In dam body, weirs 0, 4, 10L, 10R, 18 and 22; in left thrust block, weir TL-2 and in abutments, weirs LA-1, D-4, D-3 and D-2 are selected. The places of these weirs can be seen in Figure 4.5. For seepage measurement in Gökçekaya Dam, two options were suggested. No matter which option is selected, the collector canals should be cleaned from debris and algae accumulation in order to monitor the flow rate of seepage accurately.

In Option 1; it is proposed that existing weirs should have been replaced by parshall flumes since they have some advantages over conventional weirs, such as being maintenance-free and not creating a head loss like weirs. The selected parshall flumes in this option should also be automated by vibrating wire weir gages in order to fully adapt to automated data logging system. The installation places of these flumes could be the same place with the existing weirs. So the weirs should be cut though the concrete lining of the canal and then the flumes can be installed. Like the piezometers, cables have been routed to the ceiling of gallery and then carried via cable carrier trays to the multiplexer units.

Option 2 also focuses on the economic point of view of the dam instrumentation. So this alternative suggests that the existing weirs need to be repaired if necessary and then cleaned from debris and algae accumulation. Afterwards, by using the vibrating wire weir gages, the existing weirs have been equipped and monitored continuously by data logger device. For cable routing, similar approach as that of Option 1 has been applied.

In Table 5.2, the costs of two seepage monitoring options are compared with each other according to their total costs. Although Option 2 is economical, the total cost did not change as expected due to the vibrating wire weir gages' and the cables' high cost compared to the parshall flumes. Detailed tables concerning cost calculations are presented in Appendix C.

Table 5.2 Comparison of the costs of seepage monitoring options

Option	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	TOTAL COST (€)
1	25,850.00	8,739.32	16,265.60	**50,854.92**
2	21,065.00	8,739.32	16,265.60	**46,069.92**

5.4.4 Joint Movement Monitoring Options

Gökçekaya Dam consists of 25 main body blocks and 10 thrust blocks. During the construction, several embedded jointmeters had been installed along the construction joints. However, the condition of these jointmeters is unknown since no readouts giving relevant information are present. Furthermore, as seen in Figure 4.16, the readout ports have been out of order due to heavy corrosion. In addition to those, a horizontal joint opening and leakage has been observed on Block 1 (see Figure 4.10) and several leakages on vertical joints inside the gallery have been observed (see Figure 4.17) during the aforementioned site visit. As mentioned before, joint movement in concrete dams indicate several defects, such as abnormal loading or foundation problems.

After extensive studies on dam drawings obtained from DSİ and photos taken during the site investigation, some repair works, subsequent instrument retrofitting to monitor the performance of dam structure, and the positive outcomes of rehabilitative repair works have been recommended. Two options for instrument retrofitting have also been suggested.

In Option 1, along the inspection gallery, every joint was equipped with two surface mount jointmeters. One jointmeter has been proposed for the downstream side, whereas the other one for the upstream side of the joint as seen in Figure 5.4. The reason behind this layout is that the hydraulic loading on an arch dam creates either tension or compression on the different zones of the shell and two piezometers on both the upstream and downstream faces help to monitor the movement precisely. Moreover, after the repair works on the leaking joint on Block

1, the downstream face of the horizontal joint has been suggested to be instrumented with surface mount jointmeters in order to verify the performance of the repair works done. After the computations, a total number of 65 surface mount jointmeters are required in this option. Cables have been conveyed along the ceiling of the gallery by cable trays and a total length of 10.12 km of cable was required.

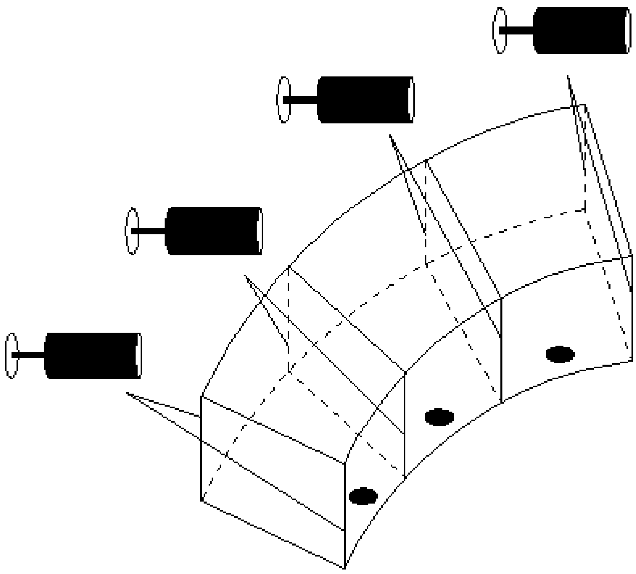

Figure 5.4 Jointmeter arrangement in Option 1

Option 2 suggests a more economical layout than Option 1 by minimizing the data loss by proper placing of 34 surface mount jointmeters. In this option, a construction joint has been equipped with only one surface mount jointmeter on either downstream or upstream face. In addition to this, if a joint has been monitored on its downstream face, the neighboring joints have been instrumented on the upstream face as shown in Figure 5.5. This layout has created an alternating plan and thus reduced the required number of instruments. As a result of fewer instruments, total length of cables has been reduced to 5.59 km.

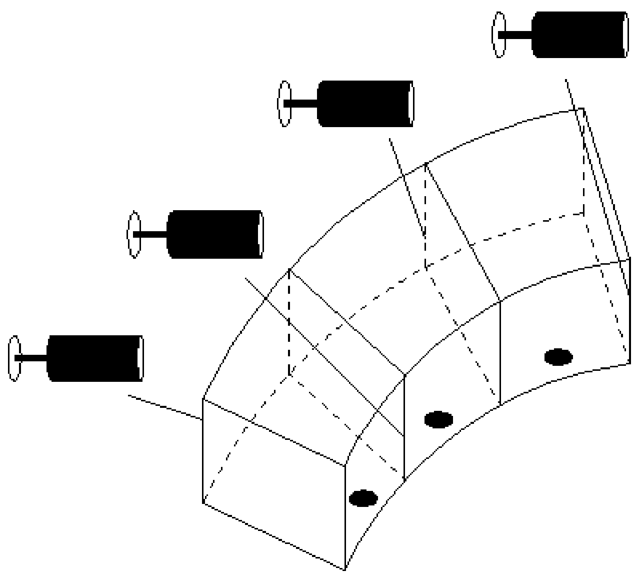

Figure 5.5 Jointmeter arrangement in Option 2

In Table 5.3, the costs of two joint movement monitoring options are compared to each other according to their total costs. Detailed cost-computation tables are presented in Appendix D.

Table 5.3 Comparison of the costs of joint movement monitoring options

Option	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	TOTAL COST (€)
1	35,750.00	49,595.33	23,821.20	**109,166.53**
2	18,700.00	27,363.00	12,430.60	**58,493.60**

5.4.5 Rotation (Tilt) Monitoring Options

Body movement in concrete dams, especially in arch dams, clearly indicates the foundation problems, which dominate failures of various types of dams. During the construction of Gökçekaya Dam, two faults under the dam body had been treated by several grouting processes. But in August 1983, a mud flow had been observed and DSİ officials stated that the mud had come from the fault zone by washing out. They estimated that the fault treatment had not been successfully established in some sections (Tanrıverdi et al., 1983). Because of the possibility of the deterioration of cement-based fault treatment over time, it is a good practice to monitor the foundation deformation of Gökçekaya Dam, indirectly by measuring the body movement.

Generally, concrete dams are equipped with direct and invert pendulums to measure the rotation of dam, but in case of Gökçekaya Dam, no pendulums had been installed. In addition to this, no vertical shafts had been provided. So the only way to monitor the rotation is to install surface mount clinometers. When considering the gallery layout of Gökçekaya Dam, two options of surface mount clinometers have been proposed.

Option 1 recommends installing three surface mount clinometers inside the gallery. The dam body was first divided into two and the location of the first clinometer was determined as block 11. The remaining two clinometers were installed to the blocks 4 and 21 (see Figure 5.6). The reason behind the selection of blocks 4 and 21 was the equality between their distances to the thrust blocks and the height from the crest of the dam. Finally, the total required length of the cable has been determined as 441 m.

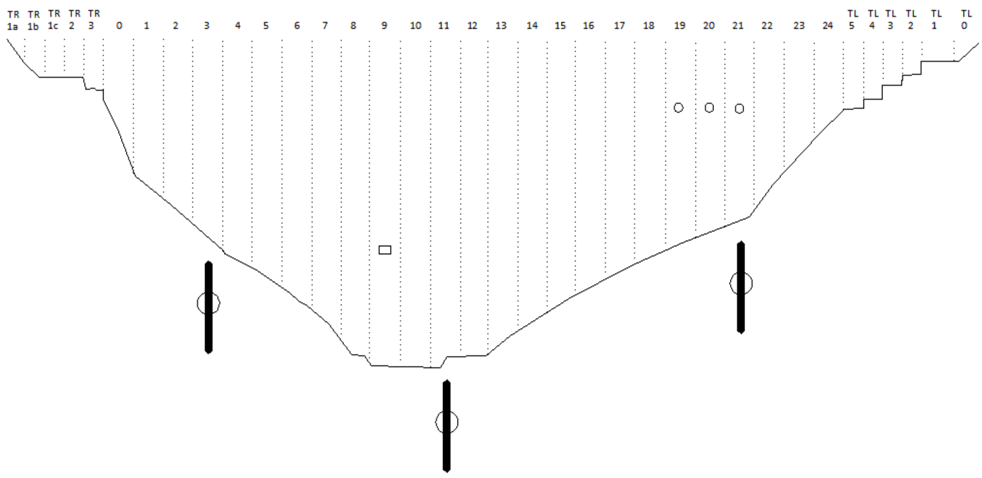

Figure 5.6 Tiltmeter arrangement in Option 1

Option 2 suggests installing only one clinometer on the block 11 as shown in Figure 5.7. As a result of being an economical alternative, this layout reduces the total cost around one third. In Table 5.4, the costs of two rotation (tilt) monitoring options are compared to each other according to their cost items and total costs. Detailed tables concerning cost computations are presented in Appendix E.

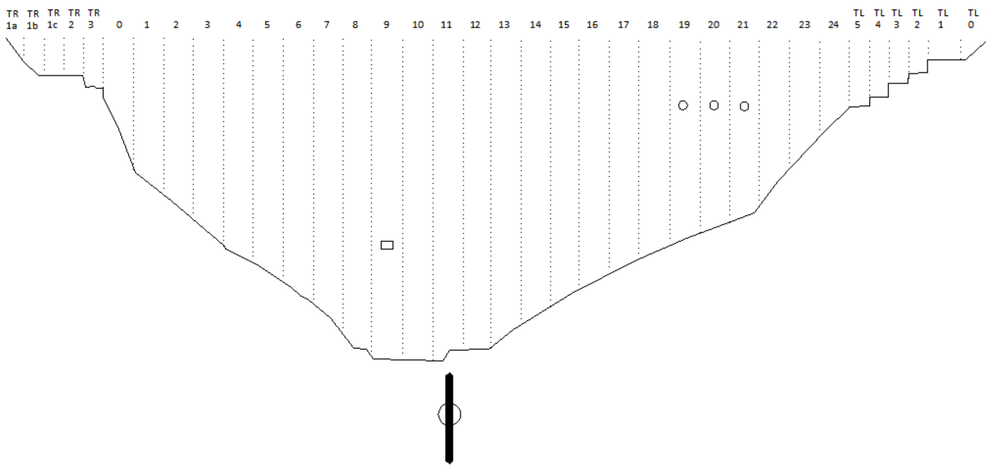

Figure 5.7 Tiltmeter arrangement in Option 2

Table 5.4 Comparison of the costs of rotation (tilt) monitoring options

Option	**Cost of Instrument (€)**	**Cost of Cable (€)**	**Cost of Cable Carrier (€)**	**TOTAL COST (€)**
1	2,685.00	2,163.84	7,397.00	**12,245.84**
2	895.00	566.32	2,587.00	**4,048.32**

5.4.6 Earthquake Acceleration Monitoring Options

Gökçekaya Dam has been placed in the third earthquake zone on the Ankara – Eskişehir province border. However, the dam site is only 30 km far from the first earthquake zone and 80 km from Düzce, the epicenter of the 12 November 1999 Earthquake with a magnitude of 7.2. Gupta (2002) investigated possible sources of earthquakes in dam environments. His results covering more than 90 earthquakes indicated that these earthquakes were triggered by reservoirs with the recorded largest event at Koyna Dam reservoir, India, in 1967 having a magnitude of 6.3. Gupta (2002) also stated that the depth and the volume of the reservoir are the most important factors in reservoir induced seismicity. One of the recent studies implied that reservoir induced earthquakes have been occurring over 44 years in Koyna region (Gupta et al., 2007). Considering the left and right faults at foundation and distance to the North Anatolian Fault Line, Gökçekaya Dam site may be evaluated as a seismically active region. Moreover, the depth and the volume of reservoir dictate that Gökçekaya Dam is prone to reservoir induced seismicity.

Earthquake performance and behavior under seismic loading can only be evaluated with proper measurement of earthquake acceleration. So, it has been proposed that Gökçekaya Dam need to be equipped with strong motion accelerometers in order to assess the acceleration during an earthquake. Strong motion accelerometers have not been monitored by automated data loggers like the other instruments. So they require a common triggering unit and a recorder. Common triggering unit activates the strong motion accelerometer at a time, in case it senses any earthquake trigger (Sezgin, 2008). In Gökçekaya Dam, two options for installing strong motion accelerometers have been suggested.

In Option 1, three accelerometers have been proposed. Two accelerometers are proposed to be placed on the dam body and the remaining one to the left abutment as a free-field accelerometer. A typical distance for free-field accelerometer in dam applications was defined as twice the height of dam for concrete dams and half of this if the modulus of elasticity of foundation is equal to or higher than the modulus of elasticity of the dam concrete (Darbre, 1995). The first accelerometer is placed to the dam-foundation interface at Block 10, whereas the second one is placed on the crest of Block 10. The third accelerometer is installed as a free-field accelerometer, 300 m from the left abutment.

Option 2 was suggested as consisting one accelerometer which is placed on the crest of Block 10. No matter which option has been selected, the required number of common triggering unit and data recorder is only one. In Table 5.5, the costs of two earthquake acceleration monitoring options are compared to each other according to their cost items and total costs. Cost of common triggering unit and data recorder are added to the cost of instruments equally. Detailed tables concerning cost calculations are presented in Appendix F.

Table 5.5 Comparison of the costs of earthquake acceleration monitoring options

Option	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	TOTAL COST (€)
1	57,547.00	4,194.13	11,499.80	**73,240.93**
2	35,041.00	1,052.62	4,856.80	**40,950.42**

5.4.7 Evaluation of Alternatives

Totally 48 combinations are possible with three options for piezometer, two options for seepage monitoring, joint movement monitoring, rotation monitoring and earthquake acceleration monitoring. Options are designated by two-character option codes. The first character defines the instrument group as P for piezometers, W for seepage monitoring, J for joint movement monitoring, T for rotation (tilt) monitoring, and A for earthquake acceleration monitoring. The second character identifies the option numbers described in detail in previous sections. Alternative codes are introduced in Appendix G. The total numbers of instruments in alternatives are also presented in Appendix G. The highest number of instrument belongs to Alternative 1 as 117 and the lowest is Alternative 48 with 63 instruments. Total numbers of instruments are used to determine the required number of multiplexers, which are used to increase the quantity of instruments to be monitored by automated data acquisition system. For every 8 instruments including temperature sensors, one multiplexer should be added to the automated data acquisition system. These calculations made the computation of the automation cost possible, which consists of required number of multiplexers, a data logger device and software. The costs of these items are also given in Appendix A. After determination of instrument and automation cost, total costs of alternatives have been computed and presented considering the combinations of different options. A constant cost of 15,000 € has been added to alternatives in order to represent the cost of portable datalogger which should be available in all automated systems for determining the possible faults and the cost of some additional consumables, such as cable splicing kits. The highest total cost of the proposed system has been determined as 419,060.92 € and the most economical alternative has a cost of 262,238.51 €. Total costs of all alternatives are presented in Appendix G. Cost computations are carried out considering initial costs only. As the operation, maintenance, installation, and workmanship costs are almost the same for all alternatives, these cost items are omitted.

A proper decision-making for the selection of a suitable instrumentation system is normally based on consideration of the achievement to be expected in reduction of future risks as a result of implementation of the proposed instrument retrofitting and rehabilitative actions to be taken. Therefore, the net benefit of such a retrofitting project would be the difference between the failure cost of the dam and the total cost of instrumentation and the aforementioned rehabilitative actions.

The cost of failure of Gökçekaya Dam cannot be determined unless all relevant information covering the dam site and downstream reaches is available. However, it is quite clear that this cost is much greater than the initial investment cost of such a dam, which was 582 million TL as of 1967 prices (EMO, 1967). The equivalent project cost in U.S. Dollars was obtained by considering a change rate reflecting the year in 1967, i.e. 1 U.S. Dollar = 11 TL. This cost was then converted to 2008 prices by using an official inflation rate of 545.67% in USA between 1967 and 2008 (FTF, 2008). It was eventually obtained as 288.71 million U.S. Dollars or 216.54 million Euros using an average change rate of 0.75 in December 2008. According to a criterion given by USBR (1987), the instrumentation cost is approximately 1% of the total project cost which comprises approximately 2.17 million Euros. When compared to the highest-cost alternative, this value is almost 5 times the cost of retrofitted instruments to be proposed for Gökçekaya Dam.

As a final remark, since instrumentation retrofitting cost is much smaller than the initial investment cost and dam failure cost, selection of a reasonable alternative may be governed by the performance expectation from the monitoring system. The alternative having the highest ability to monitor dam performance may then be selected. To this end, it is recommended to choose either alternative 1 or 5 since they have maximum number of instruments with desired performance of monitoring the critical parameters.

5.5 Further Instrumentation

Recent developments in electronic technology lead to the production of high accuracy GPS (Global Positioning System) devices especially in elevation calculations. These advanced GPS units can be used on dams for dam surface monitoring. Together with the internal movement monitoring, such as tiltmeters, clinometers and jointmeters, surface monitoring will give the behavior of the dam body in more detailed manner. In Gökçekaya Dam, the surface monuments are in good condition but the lack of routine measuring and inexperienced staff makes these monuments obsolete. The initial cost of installing such a GPS system for surface monitoring would be high but the system's automatic data collection and processing abilities would remove the skilled operator necessity.

Foundation movements in Gökçekaya Dam had not been monitored in any way. These types of movements are best monitored by extensometers. Borehole extensometers are very suitable for both concrete dam foundation monitoring and retrofitting. In this study, installation of extensometers is eliminated because of the necessity of drilling boreholes for them. However, after application of one of the previously proposed instrumentation systems, the trend of the gathered joint and body movement data during a specified time can be evaluated and the possible demand of such a monitoring system for foundation movement may be determined.

The quality and the diversity of the data obtained from instruments can be improved by constant monitoring of the reservoir water level. Reservoir levels are monitored precisely by using reservoir level gauges mounted on the upstream face of the dam body and then calibrated.

CHAPTER 6

CONCLUSIONS

Growing multi-purpose project demands lead to requirement of large dams that may exhibit a higher risk at the downstream. Hence they must be monitored, inspected, and repaired periodically in order to ensure the required safety level throughout their physical life. Almost all newly built dams have proper and adequate instrumentation for progressive monitoring. So relevant preventative measures could easily be taken without delay to offset any deficiency detected from instruments. However, when we consider the old dams, generally, instruments do not work properly or more critically, no monitoring equipment had ever been installed. The idea behind instrument retrofitting is to install latest-technology monitoring equipment to old dams in order to assess the destructive effects of aging and thus take the necessary corrective actions on time.

In this study, major causes and mechanisms of dam failures have been discussed, the types of monitoring equipment were introduced, and the dam monitoring instruments, which are suitable for retrofitting, were described. Gökçekaya Dam, which was completed in 1972, is considered in a case study in order to demonstrate the equipment that may be installed. First, the numerous deficiencies on the dam body and appurtenant structures have been partially identified through a site visit. Then, these evidences were interpreted according to the investigation of an event occurred in 1983, which gave further information about the history of the dam. After evaluation of results, a number of rehabilitative repair works and a contemporary dam monitoring system have been proposed with cost analysis of different alternatives.

Alternatives are formed by the combination of various options of different types of instruments. Cabling costs were found to be of the order of instrument costs. The highest-cost belongs to Alternative 1 with 419,060.92 €, whereas the Alternative 48 has the lowest cost of 262,238.51 €. It should be noted that these alternatives consist only the initial instrumentation costs and the costs of rehabilitative measures as well as repair, maintenance, installation, and workmanship costs should also be added to determine the overall costs. As already discussed in detail, the cost of instrumentation retrofitting is normally much smaller than the failure cost of a dam. Therefore, the reduction in future risks as a result of implementation of a retrofitting system would highlight the benefits to be gained.

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APPENDIX A

In Table A.1 the complete price list of all proposed instruments are given in European Currency Unit (EUR, €) with reference to IIC (2007).

Table A.1 Unit prices of instruments

Equipment Type	Unit price per item
VW Piezometer	400.00 €
V-Notch Weir and Automatic VW Readout Unit	2,145.00 €
Automatic VW Weir Readout Unit	1,915.00 €
Parshall Flume and Automatic VW Readout Unit	2,350.00 €
Surface Mount VW Jointmeter	550.00 €
Accelerometer	11,253.00 €
Surface Mount Clinometer (Tiltmeter)	895.00 €
Instrument Cable	4.90 €/m
Cable Carrier Tray	26.00 €/m
Multiplexer	4,425.00 €
Data Logger	15,104.00 €
Software	16,300.00 €
Common Triggering Unit and Data Recorder for Accelerometers	23,788.00 €
Handheld Data Readout	5,627.00 €

APPENDIX B

Required cable lengths of Piezometer Option 1, 2 and 3 are presented in Table B.1, Table B.3 and Table B.5, respectively. Also, detailed cost calculations of Piezometer Option 1, 2 and 3 are given in Table B.2, Table B.4 and Table B.6, respectively.

Table B.1 Required cable lengths of piezometer option 1

Table B.1 lace	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
TR-2	12.00	294.60	45.99	352.59	15.30
TR-3	12.00	289.40	45.21	346.61	15.30
0	12.00	279.40	43.71	335.11	15.30
1	12.00	253.50	39.83	305.33	15.30
2	12.00	236.50	37.28	285.78	15.30
3	12.00	220.00	34.80	266.80	15.30
4	12.00	203.00	32.25	247.25	15.30
5	12.00	188.00	30.00	230.00	15.30
6	12.00	171.00	27.45	210.45	15.30
7	12.00	155.00	25.05	192.05	15.30
8	12.00	138.50	22.58	173.08	15.30
9	12.00	124.50	20.48	156.98	15.30
10	12.00	112.00	18.60	142.60	15.30
11	12.00	99.50	16.73	128.23	15.30
12	12.00	87.50	14.93	114.43	15.30
13	12.00	73.50	12.83	98.33	15.30
14	12.00	56.50	10.28	78.78	15.30
15	12.00	41.50	8.03	61.53	15.30
16	12.00	27.00	5.85	44.85	15.30

Table B.1 Required cable lengths of piezometer option 1 (cont.)

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
17	12.00	13.00	3.75	28.75	15.30
18	12.00	15.00	4.05	31.05	15.30
19	12.00	29.00	6.15	47.15	15.30
20	12.00	44.00	8.40	64.40	15.30
21	12.00	61.50	11.03	84.53	15.30
22	12.00	79.00	13.65	104.65	15.30
23	12.00	94.50	15.98	122.48	15.30
24	12.00	112.00	18.60	142.60	15.30
TL-5	12.00	121.00	19.95	152.95	15.30
TL-4	12.00	130.00	21.30	163.30	15.30
TL-3	12.00	139.50	22.73	174.23	15.30
TL-2	12.00	143.50	23.33	178.83	15.30
Well-149	16.00	339.60	53.34	408.94	15.30
Well-147	16.00	332.10	52.22	400.32	15.30
Well-145	16.00	339.60	53.34	408.94	15.30
Well-143	18.00	347.10	54.77	419.87	15.30
Total	**438.00**	**5391.30**	**874.40**	**6703.70**	**535.60**

Table B.2 Detailed costs of piezometer option 1

Place	**Cost of Instrument (€)**	**Cost of Cable (€)**	**Cost of Cable Carrier (€)**	**Total Cost (€)**
TR-2	400.00	1,727.69	397.87	**2,525.57**
TR-3	400.00	1,698.39	397.87	**2,496.26**
0	400.00	1,642.04	397.87	**2,439.91**
1	400.00	1,496.09	397.87	**2,293.97**
2	400.00	1,400.30	397.87	**2,198.17**
3	400.00	1,307.32	397.87	**2,105.19**
4	400.00	1,211.53	397.87	**2,009.40**
5	400.00	1,127.00	397.87	**1,924.87**
6	400.00	1,031.21	397.87	**1,829.08**
7	400.00	941.05	397.87	**1,738.92**
8	400.00	848.07	397.87	**1,645.94**
9	400.00	769.18	397.87	**1,567.05**
10	400.00	698.74	397.87	**1,496.61**
11	400.00	628.30	397.87	**1,426.18**
12	400.00	560.68	397.87	**1,358.56**
13	400.00	481.79	397.87	**1,279.67**
14	400.00	386.00	397.87	**1,183.87**
15	400.00	301.47	397.87	**1,099.35**
16	400.00	219.77	397.87	**1,017.64**
17	400.00	140.88	397.87	**938.75**
18	400.00	152.15	397.87	**950.02**
19	400.00	231.04	397.87	**1,028.91**
20	400.00	315.56	397.87	**1,113.43**
21	400.00	414.17	397.87	**1,212.05**
22	400.00	512.79	397.87	**1,310.66**
23	400.00	600.13	397.87	**1,398.00**
24	400.00	698.74	397.87	**1,496.61**

Table B.2 Detailed costs of piezometer option 1 (cont.)

Place	**Cost of Instrument (€)**	**Cost of Cable (€)**	**Cost of Cable Carrier (€)**	**Total Cost (€)**
TL-5	400.00	749.46	397.87	**1,547.33**
TL-4	400.00	800.17	397.87	**1,598.04**
TL-3	400.00	853.70	397.87	**1,651.58**
TL-2	400.00	876.24	397.87	**1,674.12**
Well-149	400.00	2,003.81	397.87	**2,801.68**
Well-147	400.00	1,961.54	397.87	**2,759.42**
Well-145	400.00	2,003.81	397.87	**2,801.68**
Well-143	400.00	2,057.34	397.87	**2,855.21**
Total	**14,000.00**	**32,848.11**	**13,925.60**	**60,773.71**

Table B.3 Required cable lengths of piezometer option 2

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
TR-2	12.00	294.60	45.99	352.59	14.13
TR-3	12.00	289.40	45.21	346.61	14.13
0	12.00	279.40	43.71	335.11	14.13
1	12.00	253.50	39.83	305.33	14.13
2	12.00	236.50	37.28	285.78	14.13
3	12.00	220.00	34.80	266.80	14.13
4	12.00	203.00	32.25	247.25	14.13
5	12.00	188.00	30.00	230.00	14.13
6	12.00	171.00	27.45	210.45	14.13
7	12.00	155.00	25.05	192.05	14.13
8	12.00	138.50	22.58	173.08	14.13
9	12.00	124.50	20.48	156.98	14.13
10	12.00	112.00	18.60	142.60	14.13
11	12.00	99.50	16.73	128.23	14.13
12	12.00	87.50	14.93	114.43	14.13
13	12.00	73.50	12.83	98.33	14.13
14	12.00	56.50	10.28	78.78	14.13
15	12.00	41.50	8.03	61.53	14.13
16	12.00	27.00	5.85	44.85	14.13
17	12.00	13.00	3.75	28.75	14.13
18	12.00	15.00	4.05	31.05	14.13
19	12.00	29.00	6.15	47.15	14.13
20	12.00	44.00	8.40	64.40	14.13
21	12.00	61.50	11.03	84.53	14.13
22	12.00	79.00	13.65	104.65	14.13
23	12.00	94.50	15.98	122.48	14.13
24	12.00	112.00	18.60	142.60	14.13
TL-5	12.00	121.00	19.95	152.95	14.13
TL-4	12.00	130.00	21.30	163.30	14.13
TL-3	12.00	139.50	22.73	174.23	14.13
TL-2	12.00	143.50	23.33	178.83	14.13
Total	**372.00**	**4032.90**	**660.74**	**5065.64**	**438.10**

Table B.4 Detailed costs of piezometer option 2

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
TR-2	400.00	1,727.69	367.44	**2,495.13**
TR-3	400.00	1,698.39	367.44	**2,465.83**
0	400.00	1,642.04	367.44	**2,409.48**
1	400.00	1,496.09	367.44	**2,263.53**
2	400.00	1,400.30	367.44	**2,167.74**
3	400.00	1,307.32	367.44	**2,074.76**
4	400.00	1,211.53	367.44	**1,978.96**
5	400.00	1,127.00	367.44	**1,894.44**
6	400.00	1,031.21	367.44	**1,798.64**
7	400.00	941.05	367.44	**1,708.48**
8	400.00	848.07	367.44	**1,615.51**
9	400.00	769.18	367.44	**1,536.62**
10	400.00	698.74	367.44	**1,466.18**
11	400.00	628.30	367.44	**1,395.74**
12	400.00	560.68	367.44	**1,328.12**
13	400.00	481.79	367.44	**1,249.23**
14	400.00	386.00	367.44	**1,153.44**
15	400.00	301.47	367.44	**1,068.91**
16	400.00	219.77	367.44	**987.20**
17	400.00	140.88	367.44	**908.31**
18	400.00	152.15	367.44	**919.58**
19	400.00	231.04	367.44	**998.47**
20	400.00	315.56	367.44	**1,083.00**
21	400.00	414.17	367.44	**1,181.61**
22	400.00	512.79	367.44	**1,280.22**
23	400.00	600.13	367.44	**1,367.57**
24	400.00	698.74	367.44	**1,466.18**
TL-5	400.00	749.46	367.44	**1,516.89**
TL-4	400.00	800.17	367.44	**1,567.61**
TL-3	400.00	853.70	367.44	**1,621.14**
TL-2	400.00	876.24	367.44	**1,643.68**
Total	**12,400.00**	**24,821.61**	**11,390.60**	**48,612.21**

Table B.5 Required cable lengths of piezometer option 3

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	**Horizontal**	**Sag**	**Total**	
TR-2	12.00	294.60	45.99	352.59	27.38
0	12.00	279.40	43.71	335.11	27.38
2	12.00	236.50	37.28	285.78	27.38
4	12.00	203.00	32.25	247.25	27.38
6	12.00	171.00	27.45	210.45	27.38
8	12.00	138.50	22.58	173.08	27.38
10	12.00	112.00	18.60	142.60	27.38
12	12.00	87.50	14.93	114.43	27.38
14	12.00	56.50	10.28	78.78	27.38
16	12.00	27.00	5.85	44.85	27.38
18	12.00	15.00	4.05	31.05	27.38
20	12.00	44.00	8.40	64.40	27.38
22	12.00	79.00	13.65	104.65	27.38
24	12.00	112.00	18.60	142.60	27.38
TL-4	12.00	130.00	21.30	163.30	27.38
TL-2	12.00	143.50	23.33	178.83	27.38
Total	**192.00**	**2129.50**	**348.23**	**2669.73**	**438.10**

Table B.6 Detailed costs of piezometer option 3

Place	**Cost of Instrument (€)**	**Cost of Cable (€)**	**Cost of Cable Carrier (€)**	**Total Cost (€)**
TR-2	400.00	1,727.69	711.91	**2,839.60**
0	400.00	1,642.04	711.91	**2,753.95**
2	400.00	1,400.30	711.91	**2,512.21**
4	400.00	1,211.53	711.91	**2,323.44**
6	400.00	1,031.21	711.91	**2,143.12**
8	400.00	848.07	711.91	**1,959.98**
10	400.00	698.74	711.91	**1,810.65**
12	400.00	560.68	711.91	**1,672.60**
14	400.00	386.00	711.91	**1,497.91**
16	400.00	219.77	711.91	**1,331.68**
18	400.00	152.15	711.91	**1,264.06**
20	400.00	315.56	711.91	**1,427.47**
22	400.00	512.79	711.91	**1,624.70**
24	400.00	698.74	711.91	**1,810.65**
TL-4	400.00	800.17	711.91	**1,912.08**
TL-2	400.00	876.24	711.91	**1,988.16**
Total	**6,400.00**	**13,081.65**	**11,390.60**	**30,872.25**

APPENDIX C

Required cable lengths of Seepage Monitoring Option 1 and Option 2 are presented in Table C.1 and Table C.3, respectively. Also, detailed cost calculations of Seepage Monitoring Option 1 and Option 2 are given in Table C.2 and Table C.4, respectively.

Table C.1 Required cable lengths of seepage monitoring option 1

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
0	2.00	279.40	42.21	323.61	56.87
4	2.00	203.00	30.75	235.75	56.87
10L	2.00	116.00	17.70	135.70	56.87
10R	2.00	108.00	16.50	126.50	56.87
18	2.00	15.00	2.55	19.55	56.87
22	2.00	79.00	12.15	93.15	56.87
TL-2	2.00	143.50	21.83	167.33	56.87
LA-1	2.00	139.50	21.23	162.73	56.87
D-4	2.00	81.50	12.53	96.03	56.87
D-3	2.00	75.00	11.55	88.55	56.87
D-2	2.00	289.00	43.65	334.65	56.87
Total	**22.00**	**1528.90**	**232.64**	**1783.54**	**625.60**

Table C.2 Detailed costs of seepage monitoring option 1

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
0	2,350.00	1,585.69	1,478.69	**5,414.38**
4	2,350.00	1,155.18	1,478.69	**4,983.87**
10L	2,350.00	664.93	1,478.69	**4,493.62**
10R	2,350.00	619.85	1,478.69	**4,448.54**
18	2,350.00	95.80	1,478.69	**3,924.49**
22	2,350.00	456.44	1,478.69	**4,285.13**
TL-2	2,350.00	819.89	1,478.69	**4,648.58**
LA-1	2,350.00	797.35	1,478.69	**4,626.04**
D-4	2,350.00	470.52	1,478.69	**4,299.21**
D-3	2,350.00	433.90	1,478.69	**4,262.59**
D-2	2,350.00	1,639.79	1,478.69	**5,468.48**
Total	**25,850.00**	**8,739.32**	**16,265.60**	**50,854.92**

Table C.3 Required cable lengths of seepage monitoring option 2

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
0	2.00	279.40	42.21	323.61	56.87
4	2.00	203.00	30.75	235.75	56.87
10L	2.00	116.00	17.70	135.70	56.87
10R	2.00	108.00	16.50	126.50	56.87
18	2.00	15.00	2.55	19.55	56.87
22	2.00	79.00	12.15	93.15	56.87
TL-2	2.00	143.50	21.83	167.33	56.87
LA-1	2.00	139.50	21.23	162.73	56.87
D-4	2.00	81.50	12.53	96.03	56.87
D-3	2.00	75.00	11.55	88.55	56.87
D-2	2.00	289.00	43.65	334.65	56.87
Total	**22.00**	**1528.90**	**232.64**	**1783.54**	**625.60**

Table C.4 Detailed costs of seepage monitoring option 2

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
0	1,915.00	1,585.69	1,478.69	**4,979.38**
4	1,915.00	1,155.18	1,478.69	**4,548.87**
10L	1,915.00	664.93	1,478.69	**4,058.62**
10R	1,915.00	619.85	1,478.69	**4,013.54**
18	1,915.00	95.80	1,478.69	**3,489.49**
22	1,915.00	456.44	1,478.69	**3,850.13**
TL-2	1,915.00	819.89	1,478.69	**4,213.58**
LA-1	1,915.00	797.35	1,478.69	**4,191.04**
D-4	1,915.00	470.52	1,478.69	**3,864.21**
D-3	1,915.00	433.90	1,478.69	**3,827.59**
D-2	1,915.00	1,639.79	1,478.69	**5,033.48**
Total	**21,065.00**	**8,739.32**	**16,265.60**	**46,069.92**

APPENDIX D

Required cable lengths of Joint Movement Monitoring Option 1 and Option 2 are presented in Table D.1 and Table D.3, respectively. Also, detailed cost calculations of Joint Movement Monitoring Option 1 and Option 2 are given in Table D.2 and Table D.4, respectively.

Table D.1 Required cable lengths of joint monitoring option 1

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
TR-1c / TR-2	1.00	314.60	47.34	725.88	28.63
TR-2 / TR-3	1.00	289.40	43.56	667.92	28.63
TR-3 / 0	1.00	279.40	42.06	644.92	28.63
0 / 1	1.00	253.50	38.18	585.35	28.63
1 / 2	1.00	236.50	35.63	546.25	28.63
2 / 3	1.00	220.00	33.15	508.30	28.63
3 / 4	1.00	203.00	30.60	469.20	28.63
4 / 5	1.00	188.00	28.35	434.70	28.63
5 / 6	1.00	171.00	25.80	395.60	28.63
6 / 7	1.00	155.00	23.40	358.80	28.63
7 / 8	1.00	138.50	20.93	320.85	28.63
8 / 9	1.00	124.50	18.83	288.65	28.63
9 / 10	1.00	112.00	16.95	259.90	28.63
10 / 11	1.00	99.50	15.08	231.15	28.63
11 / 12	1.00	87.50	13.28	203.55	28.63
12 / 13	1.00	73.50	11.18	171.35	28.63
13 / 14	1.00	56.50	8.63	132.25	28.63

Table D.1 Required cable lengths of joint monitoring option 1 (cont.)

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
14 / 15	1.00	41.50	6.38	97.75	28.63
15 / 16	1.00	27.00	4.20	64.40	28.63
16 / 17	1.00	13.00	2.10	32.20	28.63
17 / 18	1.00	5.00	0.90	13.80	28.63
18 / 19	1.00	15.00	2.40	36.80	28.63
19 / 20	1.00	29.00	4.50	69.00	28.63
20 / 21	1.00	44.00	6.75	103.50	28.63
21 / 22	1.00	61.50	9.38	143.75	28.63
22 / 23	1.00	79.00	12.00	184.00	28.63
23 / 24	1.00	94.50	14.33	219.65	28.63
24 / TL-5	1.00	112.00	16.95	259.90	28.63
TL-5 / TL-4	1.00	121.00	18.30	280.60	28.63
TL-4 / TL-3	1.00	130.00	19.65	301.30	28.63
TL-3 / TL-2	1.00	139.50	21.08	323.15	28.63
Crack on Block 1	10.00	293.50	45.53	1047.08	28.63
Total	**41.00**	**4207.90**	**637.34**	**10121.50**	**916.20**

Table D.2 Detailed costs of joint monitoring option 1

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
TR-1c / TR-2	1,100.00	3,556.81	744.41	**5,401.22**
TR-2 / TR-3	1,100.00	3,272.81	744.41	**5,117.22**
TR-3 / 0	1,100.00	3,160.11	744.41	**5,004.52**
0 / 1	1,100.00	2,868.22	744.41	**4,712.63**
1 / 2	1,100.00	2,676.63	744.41	**4,521.04**
2 / 3	1,100.00	2,490.67	744.41	**4,335.08**
3 / 4	1,100.00	2,299.08	744.41	**4,143.49**
4 / 5	1,100.00	2,130.03	744.41	**3,974.44**
5 / 6	1,100.00	1,938.44	744.41	**3,782.85**
6 / 7	1,100.00	1,758.12	744.41	**3,602.53**
7 / 8	1,100.00	1,572.17	744.41	**3,416.58**
8 / 9	1,100.00	1,414.39	744.41	**3,258.80**
9 / 10	1,100.00	1,273.51	744.41	**3,117.92**
10 / 11	1,100.00	1,132.64	744.41	**2,977.05**
11 / 12	1,100.00	997.40	744.41	**2,841.81**
12 / 13	1,100.00	839.62	744.41	**2,684.03**
13 / 14	1,100.00	648.03	744.41	**2,492.44**
14 / 15	1,100.00	478.98	744.41	**2,323.39**
15 / 16	1,100.00	315.56	744.41	**2,159.97**
16 / 17	1,100.00	157.78	744.41	**2,002.19**
17 / 18	1,100.00	67.62	744.41	**1,912.03**
18 / 19	1,100.00	180.32	744.41	**2,024.73**
19 / 20	1,100.00	338.10	744.41	**2,182.51**
20 / 21	1,100.00	507.15	744.41	**2,351.56**
21 / 22	1,100.00	704.38	744.41	**2,548.79**
22 / 23	1,100.00	901.60	744.41	**2,746.01**
23 / 24	1,100.00	1,076.29	744.41	**2,920.70**
24 / TL-5	1,100.00	1,273.51	744.41	**3,117.92**
TL-5 / TL-4	1,100.00	1,374.94	744.41	**3,219.35**
TL-4 / TL-3	1,100.00	1,476.37	744.41	**3,320.78**
TL-3 / TL-2	1,100.00	1,583.44	744.41	**3,427.85**
Crack on Block 1	1,650.00	5,130.67	744.41	**7,525.08**
Total	**35,750.00**	**49,595.33**	**23,821.20**	**109,166.53**

Table D.3 Required cable lengths of joint monitoring option 2

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
TR-1c / TR-2	1.00	314.60	47.34	362.94	14.94
TR-2 / TR-3	1.00	289.40	43.56	333.96	14.94
TR-3 / 0	1.00	279.40	42.06	322.46	14.94
0 / 1	1.00	253.50	38.18	292.68	14.94
1 / 2	1.00	236.50	35.63	273.13	14.94
2 / 3	1.00	220.00	33.15	254.15	14.94
3 / 4	1.00	203.00	30.60	234.60	14.94
4 / 5	1.00	188.00	28.35	217.35	14.94
5 / 6	1.00	171.00	25.80	197.80	14.94
6 / 7	1.00	155.00	23.40	179.40	14.94
7 / 8	1.00	138.50	20.93	160.43	14.94
8 / 9	1.00	124.50	18.83	144.33	14.94
9 / 10	1.00	112.00	16.95	129.95	14.94
10 / 11	1.00	99.50	15.08	115.58	14.94
11 / 12	1.00	87.50	13.28	101.78	14.94
12 / 13	1.00	73.50	11.18	85.68	14.94
13 / 14	1.00	56.50	8.63	66.13	14.94
14 / 15	1.00	41.50	6.38	48.88	14.94
15 / 16	1.00	27.00	4.20	32.20	14.94
16 / 17	1.00	13.00	2.10	16.10	14.94
17 / 18	1.00	5.00	0.90	6.90	14.94
18 / 19	1.00	15.00	2.40	18.40	14.94
19 / 20	1.00	29.00	4.50	34.50	14.94
20 / 21	1.00	44.00	6.75	51.75	14.94

Table D.3 Required cable lengths of joint monitoring option 2 (cont.)

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
21 / 22	1.00	61.50	9.38	71.88	14.94
22 / 23	1.00	79.00	12.00	92.00	14.94
23 / 24	1.00	94.50	14.33	109.83	14.94
24 / TL-5	1.00	112.00	16.95	129.95	14.94
TL-5 / TL-4	1.00	121.00	18.30	140.30	14.94
TL-4 / TL-3	1.00	130.00	19.65	150.65	14.94
TL-3 / TL-2	1.00	139.50	21.08	161.58	14.94
Crack on Block 1	10.00	293.50	45.53	1047.08	14.94
Total	**41.00**	**4207.90**	**637.34**	**5584.29**	**478.10**

Table D.4 Detailed costs of joint monitoring option 2

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
TR-1c / TR-2	550.00	1,778.41	388.46	**2,716.86**
TR-2 / TR-3	550.00	1,636.40	388.46	**2,574.86**
TR-3 / 0	550.00	1,580.05	388.46	**2,518.51**
0 / 1	550.00	1,434.11	388.46	**2,372.56**
1 / 2	550.00	1,338.31	388.46	**2,276.77**
2 / 3	550.00	1,245.34	388.46	**2,183.79**
3 / 4	550.00	1,149.54	388.46	**2,088.00**
4 / 5	550.00	1,065.02	388.46	**2,003.47**
5 / 6	550.00	969.22	388.46	**1,907.68**
6 / 7	550.00	879.06	388.46	**1,817.52**
7 / 8	550.00	786.08	388.46	**1,724.54**
8 / 9	550.00	707.19	388.46	**1,645.65**
9 / 10	550.00	636.76	388.46	**1,575.21**
10 / 11	550.00	566.32	388.46	**1,504.77**
11 / 12	550.00	498.70	388.46	**1,437.15**
12 / 13	550.00	419.81	388.46	**1,358.26**
13 / 14	550.00	324.01	388.46	**1,262.47**
14 / 15	550.00	239.49	388.46	**1,177.94**
15 / 16	550.00	157.78	388.46	**1,096.24**
16 / 17	550.00	78.89	388.46	**1,017.35**
17 / 18	550.00	33.81	388.46	**972.27**
18 / 19	550.00	90.16	388.46	**1,028.62**
19 / 20	550.00	169.05	388.46	**1,107.51**
20 / 21	550.00	253.58	388.46	**1,192.03**
21 / 22	550.00	352.19	388.46	**1,290.64**
22 / 23	550.00	450.80	388.46	**1,389.26**
23 / 24	550.00	538.14	388.46	**1,476.60**
24 / TL-5	550.00	636.76	388.46	**1,575.21**
TL-5 / TL-4	550.00	687.47	388.46	**1,625.93**
TL-4 / TL-3	550.00	738.19	388.46	**1,676.64**
TL-3 / TL-2	550.00	791.72	388.46	**1,730.17**
Crack on Block 1	1,650.00	5,130.67	388.46	**7,169.12**
Total	**18,700.00**	**27,363.00**	**12,430.60**	**58,493.60**

APPENDIX E

Required cable lengths of Rotation (Tilt) Monitoring Option 1 and Option 2 are presented in Table E.1 and Table E.3, respectively. Also, detailed cost calculations of Rotation (Tilt) Monitoring Option 1 and Option 2 are given in Table E.2 and Table E.4, respectively.

Table E.1 Required cable lengths of rotation (tilt) monitoring option 1

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
3	1.00	220.00	33.15	254.15	94.83
11	1.00	99.50	15.08	115.58	94.83
21	1.00	61.50	9.38	71.88	94.83
Total	**3.00**	**381.00**	**57.60**	**441.60**	**284.50**

Table E.2 Detailed costs of rotation (tilt) monitoring option 1

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
3	895.00	1,245.34	2,465.67	**4,606.00**
11	895.00	566.32	2,465.67	**3,926.98**
21	895.00	352.19	2,465.67	**3,712.85**
Total	**2,685.00**	**2,163.84**	**7,397.00**	**12,245.84**

Table E.3 Required cable lengths of rotation (tilt) monitoring option 2

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
11	1.00	99.50	15.08	115.58	99.50
Total	**1.00**	**99.50**	**15.08**	**115.58**	**99.50**

Table E.4 Detailed Costs of rotation (tilt) monitoring option 2

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
11	895.00	566.32	2,587.00	**4,048.32**
Total	**895.00**	**566.32**	**2,587.00**	**4,048.32**

APPENDIX F

Required cable lengths of Earthquake Acceleration Monitoring Option 1 and Option 2 are presented in Table F.1 and Table F.3, respectively. Also, detailed cost calculations of Earthquake Acceleration Monitoring Option 1 and Option 2 are given in Table F.2 and Table F.4, respectively.

Table F.1 Required cable lengths of earthquake monitoring option 1

Place	**Cable Length (m)**				**Cable Carrier Length (m)**
	Vertical	**Horizontal**	**Sag**	**Total**	
Block 10 Crest	50.00	136.80	28.02	214.82	147.43
Block 10 Gallery	1.00	112.00	16.95	129.95	147.43
Left Abutment	1.00	443.50	66.68	511.18	147.43
Total	**52.00**	**692.30**	**111.65**	**855.95**	**442.30**

Table F.2 Detailed costs of earthquake monitoring option 1

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
Block 10 Crest	19,182.33	1,052.62	3,833.27	**24,068.22**
Block 10 Gallery	19,182.33	636.76	3,833.27	**23,652.36**
Left Abutment	19,182.33	2,504.76	3,833.27	**25,520.36**
Total	**57,547.00**	**4,194.13**	**11,499.80**	**73,240.93**

Table F.3 Required cable lengths of earthquake monitoring option 2

Place	Cable Length (m)				Cable Carrier Length (m)
	Vertical	Horizontal	Sag	Total	
Block 10 Crest	50.00	136.80	28.02	214.82	**186.80**
Total	**50.00**	**136.80**	**28.02**	**214.82**	**186.80**

Table F.4 Detailed costs of earthquake monitoring option 2

Place	Cost of Instrument (€)	Cost of Cable (€)	Cost of Cable Carrier (€)	Total Cost (€)
Block 10 Crest	35,041.00	1,052.62	4,856.80	**40,950.42**
Total	**35,041.00**	**1,052.62**	**4,856.80**	**40,950.42**

APPENDIX G

Option codes of alternatives are introduced in Table G.1. Number of instruments and required number of multiplexers of instrumentation alternatives are presented in Table G.2. In Table G.3, the total costs of alternatives are given. Table G.4 also presents the total costs of alternatives, but sorted according to total cost. It should be noted that an additional cost of 15,000 € has been added to total costs in order to compensate the costs of required handheld datalogger and possible repair kits.

Table G.1 Option codes

	Option Code		**Option Code**
Alternative 1	P1 A1 W1 J1 T1	Alternative 13	P1 A2 W2 J1 T1
Alternative 2	P1 A1 W1 J1 T2	Alternative 14	P1 A2 W2 J1 T2
Alternative 3	P1 A1 W1 J2 T1	Alternative 15	P1 A2 W2 J2 T1
Alternative 4	P1 A1 W1 J2 T2	Alternative 16	P1 A2 W2 J2 T2
Alternative 5	P1 A1 W2 J1 T1	Alternative 17	P2 A1 W1 J1 T1
Alternative 6	P1 A1 W2 J1 T2	Alternative 18	P2 A1 W1 J1 T2
Alternative 7	P1 A1 W2 J2 T1	Alternative 19	P2 A1 W1 J2 T1
Alternative 8	P1 A1 W2 J2 T2	Alternative 20	P2 A1 W1 J2 T2
Alternative 9	P1 A2 W1 J1 T1	Alternative 21	P2 A1 W2 J1 T1
Alternative 10	P1 A2 W1 J1 T2	Alternative 22	P2 A1 W2 J1 T2
Alternative 11	P1 A2 W1 J2 T1	Alternative 23	P2 A1 W2 J2 T1
Alternative 12	P1 A2 W1 J2 T2	Alternative 24	P2 A1 W2 J2 T2

Table G.1 Option codes (cont.)

	Option Code
Alternative 25	P2 A2 W1 J1 T1
Alternative 26	P2 A2 W1 J1 T2
Alternative 27	P2 A2 W1 J2 T1
Alternative 28	P2 A2 W1 J2 T2
Alternative 29	P2 A2 W2 J1 T1
Alternative 30	P2 A2 W2 J1 T2
Alternative 31	P2 A2 W2 J2 T1
Alternative 32	P2 A2 W2 J2 T2
Alternative 33	P3 A1 W1 J1 T1
Alternative 34	P3 A1 W1 J1 T2
Alternative 35	P3 A1 W1 J2 T1
Alternative 36	P3 A1 W1 J2 T2

	Option Code
Alternative 37	P3 A1 W2 J1 T1
Alternative 38	P3 A1 W2 J1 T2
Alternative 39	P3 A1 W2 J2 T1
Alternative 40	P3 A1 W2 J2 T2
Alternative 41	P3 A2 W1 J1 T1
Alternative 42	P3 A2 W1 J1 T2
Alternative 43	P3 A2 W1 J2 T1
Alternative 44	P3 A2 W1 J2 T2
Alternative 45	P3 A2 W2 J1 T1
Alternative 46	P3 A2 W2 J1 T2
Alternative 47	P3 A2 W2 J2 T1
Alternative 48	P3 A2 W2 J2 T2

Table G.2 Number of instruments and multiplexers

	Number of Instruments (excluding Accelerometers)	Number of Multiplexers
Alternative 1	114	15
Alternative 2	112	14
Alternative 3	83	11
Alternative 4	81	11
Alternative 5	114	15
Alternative 6	112	14
Alternative 7	83	11
Alternative 8	81	11
Alternative 9	114	15
Alternative 10	112	14
Alternative 11	83	11
Alternative 12	81	11

Table G.2 Number of instruments and multiplexers (cont.)

	Number of Instruments (excluding Accelerometers)	**Number of Multiplexers**
Alternative 13	114	15
Alternative 14	112	14
Alternative 15	83	11
Alternative 16	81	11
Alternative 17	110	14
Alternative 18	108	14
Alternative 19	79	10
Alternative 20	77	10
Alternative 21	110	14
Alternative 22	108	14
Alternative 23	79	10
Alternative 24	77	10
Alternative 25	110	14
Alternative 26	108	14
Alternative 27	79	10
Alternative 28	77	10
Alternative 29	110	14
Alternative 30	108	14
Alternative 31	79	10
Alternative 32	77	10
Alternative 33	95	12
Alternative 34	93	12
Alternative 35	64	8
Alternative 36	62	8
Alternative 37	95	12
Alternative 38	93	12
Alternative 39	64	8
Alternative 40	62	8
Alternative 41	95	12

Table G.2 Number of instruments and multiplexers (cont.)

	Number of Instruments (excluding Accelerometers)	Number of Multiplexers
Alternative 42	93	12
Alternative 43	64	8
Alternative 44	62	8
Alternative 45	95	12
Alternative 46	93	12
Alternative 47	64	8
Alternative 48	62	8

Table G.3 Total costs of alternatives

Alternative	Instrument and Cable Cost (€)	Automation Cost (€)	TOTAL COST (€)
1	306,281.92	97,779.00	**419,060.92**
2	298,084.40	93,354.00	**406,438.40**
3	255,608.99	80,079.00	**350,687.99**
4	247,411.47	80,079.00	**342,490.47**
5	301,496.92	97,779.00	**414,275.92**
6	293,299.40	93,354.00	**401,653.40**
7	250,823.99	80,079.00	**345,902.99**
8	242,626.47	80,079.00	**337,705.47**
9	273,991.41	97,779.00	**386,770.41**
10	265,793.89	93,354.00	**374,147.89**
11	223,318.48	80,079.00	**318,397.48**
12	215,120.96	80,079.00	**310,199.96**
13	269,206.41	97,779.00	**381,985.41**
14	261,008.89	93,354.00	**369,362.89**
15	218,533.48	80,079.00	**313,612.48**

Table G.3 Total costs of alternatives (cont.)

Alternative	Instrument and Cable Cost (€)	Automation Cost (€)	TOTAL COST (€)
16	210,335.96	80,079.00	**305,414.96**
17	294,120.43	93,354.00	**402,474.43**
18	285,922.91	93,354.00	**394,276.91**
19	243,447.50	75,654.00	**334,101.50**
20	235,249.98	75,654.00	**325,903.98**
21	289,335.43	93,354.00	**397,689.43**
22	281,137.91	93,354.00	**389,491.91**
23	238,662.50	75,654.00	**329,316.50**
24	230,464.98	75,654.00	**321,118.98**
25	261,829.92	93,354.00	**370,183.92**
26	253,632.39	93,354.00	**361,986.39**
27	211,156.99	75,654.00	**301,810.99**
28	202,959.47	75,654.00	**293,613.47**
29	257,044.92	93,354.00	**365,398.92**
30	248,847.39	93,354.00	**357,201.39**
31	206,371.99	75,654.00	**297,025.99**
32	198,174.47	75,654.00	**288,828.47**
33	276,380.47	84,504.00	**375,884.47**
34	268,182.95	84,504.00	**367,686.95**
35	225,707.54	66,804.00	**307,511.54**
36	217,510.02	66,804.00	**299,314.02**
37	271,595.47	84,504.00	**371,099.47**
38	263,397.95	84,504.00	**362,901.95**
39	220,922.54	66,804.00	**302,726.54**
40	212,725.02	66,804.00	**294,529.02**
41	244,089.96	84,504.00	**343,593.96**
42	235,892.44	84,504.00	**335,396.44**
43	193,417.03	66,804.00	**275,221.03**
44	185,219.51	66,804.00	**267,023.51**

Table G.3 Total costs of alternatives (cont.)

	Instrument and Cable Cost	**Automation Cost**	**TOTAL COST**
45	239,304.96	84,504.00	338,808.96
46	231,107.44	84,504.00	330,611.44
47	188,632.03	66,804.00	270,436.03
48	180,434.51	66,804.00	262,238.51