AN EVALUATION STUDY ON INSTRUMENTATION SYSTEM OF CİNDERE DAM

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ABSTRACT

AN EVALUATION STUDY ON INSTRUMENTATION SYSTEM OF CİNDERE DAM

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In this study, the first hardfill dam in Turkey, Cindere Dam, is evaluated regarding current and alternative measurement systems in detail. Cindere Dam is a notable project, especially with its height as an example of hardfill dam, not only in Turkey but also in the world. First, the current instrumentation system of Cindere Dam is evaluated as a whole with reference to the parameters measured, instruments and data acquisition system. Second, an alternative instrumentation system is developed by using additional parameter and instruments to the current instrumentation system. Furthermore, each instrumentation system equipped with manual and automatic data acquisition systems is considered. The comparison between the current and the alternative instrumentation systems are also carried out in terms of technical and economical feasibility aspects. Although, the current instrumentation system is found to be satisfactory, it is observed that the alternative system promotes the current system in various aspects without increasing the cost significantly.

Keywords: Cindere Dam, dam instrumentation, dam safety, hardfill dams.

ÖZ

CİNDERE BARAJI ÖLÇÜMLENDİRME SİSTEMİNİN DEĞERLENDİRME ÇALIŞMASI

Sezgin, Özgün İlke
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Bu çalışmada Türkiye'de ilk kez "Silindirle Sıkıştırılmış Katı Dolgu" yöntemi ile inşa edilen bir baraj olan Cindere Barajı'na ait mevcut ve alternatif ölçümlendirme sistemleri detaylı olarak değerlendirilmiştir. Cindere Barajı hem Türkiye'de hem de dünyada gerek baraj tipi gerekse baraj yüksekliği açısından önemli bir projedir. Öncelikle, barajdaki mevcut ölçümlendirme sistemi ölçüm parametreleri, enstrumanlar ve veri toplama sistemleri açısından bir bütün olarak incelenmiştir. Daha sonra, yeni bir ölçüm parametresi ve farklı sensörler eklenerek, mevcut sisteme ilave bir alternatif sistem geliştirilmiştir. Ayrıca, ölçümlendirme sistemleri hem elle çalışan, hem de otomatik veri toplama sistemleri olarak ele alınmıştır. Mevcut ve alternatif ölçümlendirme sistemleri, teknik ve ekonomik fizibilite kapsamında kıyaslanmıştır. Sonuç olarak, mevcut sistem yeterli bulunmasına rağmen, bu çalışma sonucu elde edilen alternatif sistemin mevcut sisteme maliyet açısından ciddi bir yük getirmeden önemli katkılar sağlayacağı belirlenmiştir.

Anahtar Kelimeler: Cindere Barajı, baraj ölçümlendirme sistemleri, baraj güvenliği, silindirle sıkıştırılmış katı dolgu barajlar.

To My Parents

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LIST OF ABBREVATIONS

ACC	**:**	Accelerometer
ADAS	**:**	Automatic Data Acquisition System
DSİ	**:**	Turkish State Hydraulic Works
EMI	**:**	Electromagnetic Interference
ER	**:**	Electrical Resistance
FSHD	**:**	Face Symmetrical Hardfill Dam
JM	**:**	Joint meter
JNCBX	**:**	Junction Box
MDAS	**:**	Manual Data Acquisition System
MUX	**:**	Multiplexer
NS	**:**	No-Stress Strain Gauge
PC	**:**	Pressure Cell
PF	**:**	Parshall Flume
PM	**:**	Piezometer
RCC	**:**	Roller Compacted Concrete
RLG:	**:**	Reservoir Level Gauge
RTD:	**:**	Resistance Temperature Detector
RUP	**:**	Reference Unit Price
SG	**:**	Strain Gauge
SP	**:**	Survey Point
SWBX	**:**	Switch Terminal Box
VN	**:**	V-Notch Weir
VW	**:**	Vibrating Wire

CHAPTER 1

INTRODUCTION

1.1 General

Water management, practices of planning, developing, distribution and optimum utilizing of water resources is gaining more importance all over the world. In this regard, numerous dams are being constructed in Turkey in order to maximize benefits from water resources. Generally, most of the existing dams were built at relatively suitable locations. The contractors are forced to use poorer sites due to decreasing number of suitable sites. Therefore, planning, designing and constructing of these projects are becoming more complex and the risk of failure is also increased (Kulga and Turan, 1999).

The enormous forces created by the impounded water in a dam may impose potential risk for the downstream life and property. Therefore, dam safety is the main objective that should be carefully considered at design phase. As an integrated part of the design, dam instrumentation plays an important role to monitor structural behavior, which is necessary for verifying design assumptions and complementing surveillance (Bartholomew and Haverland, 1987).

1.2 Scope of the Thesis

This study covers general information about hardfill dams and dam instrumentation. A case study is carried out to observe various possibilities for dam instrumentation in view of feasibility and economy.

To this end, Cindere Dam, which is built in Denizli province, Turkey, is considered. Besides, detailed evaluations of the current instrumentation system of Cindere Dam, alternative scenarios for other possible instrumentation systems were also studied. Three alternatives were designed based on the current instrumentation system. Totally, four different instrumentation systems including the current one were examined and compared in terms of maintainability and the cost issues. Although, the current instrumentation system is a found to be satisfactory, it was observed that each alternative system offers various contributions to the current instrumentation system.

In Chapter 2, detailed information on dam instrumentation is given. General features of Cindere Dam is presented in Chapter 3, evaluation of the current instrumentation system of Cindere Dam is carried out in Chapter 4. An alternative instrumentation system is designed and compared to the current system in Chapter 5. The conclusions derived are presented in Chapter 6.

1.3 Roller Compacted Concrete Dams

Cindere Dam is a face symmetrical hardfill dam. Hardfill is known as lean roller compacted concrete (RCC). Therefore, basic information about RCC dams will be presented in this chapter. Roller Compacted Concrete should be considered as a new construction technique for concrete structures more than a new construction material. The ingredients of concrete mixture are almost identical with conventional concrete. However, RCC is an extra dry, in other words no-slump concrete that facilitates placement technique similar to embankment dams (USACE, 2000).

In the second half of the 20th century, embankment and rock-fill dams were preferred over concrete dams due to their cost advantages derived mainly from the greater efficiency of the equipment and the placement method utilized in construction. The construction of concrete gravity dams at greater valleys was replaced with earth-fill and rock-fill dams while concrete arch dams were kept on building in narrow-valleys. Numerous dams were constructed throughout the world and some of them have failed. The rate of failure was significantly high in embankment dams compared to concrete dams because of their vulnerability to overtopping and internal erosion (Hansen and Reinhard, 1991). Until 1980's, no economically competitive concrete dam type could be designed. Then, roller compacted concrete was presented as a discovery which combines the structural properties of concrete and the placing characteristic of embankment dams (Hansen and Reinhard, 1991). RCC mixture can easily be transported, placed and compacted using ordinary equipment of embankment dams (See Figure 1.1).

Figure 1.1. A photo taken during construction of Cindere Dam (Gürdil, 2003)

RCC dams are considerably economical than conventional concrete dams because of low cementitious content, simple and rapid construction technique which requires less forms, man power, cooling and compaction efforts. The RCC construction process is short compared to that of embankment dams due to reduced material quantities. As a result of reduced construction time, the diversion and cofferdam can be designed according to shorter return-periods. Cofferdam can be operable even when overtopped. Reduced dam volume will also shorten the diversion conduit length. RCC dams offer integrated spillway and appurtenant structures. The smaller quantity of borrow source may make RCC dams environmentally acceptable. All above-mentioned advantages of RCC dams result in economy. Furthermore, concrete dams offer more reliable and maintainable systems regarding safety than embankment dams, statistically (USACE, 2000).

1.4 Face Symmetrical Hardfill Dam Concept

Hardfill is known as lean RCC, which provides unquestionable advantages. Contrary to RCC dams, hardfill dams are constructed using low quantity of cementitious materials (Yanmaz, 2006). Face Symmetrical Hardfill Dam (FSHD) concept, which has side slopes 0.7 H: 1.0 V in both upstream and downstream face was developed in 1990’s. The main advantages of FSHD concept are stated below (Londe and Lino, 1992).

a. The low elasticity modulus of the material results in low stresses in the dam body.
b. Less normal stress occurs on the foundation because of the symmetrical shape and weak foundations are acceptable.
c. Flexibility of the material minimizes the differential settlements. Any unexpected loadings could be tolerated fairly by hardfill dams.
d. The amount of cement is less than conventional concrete, which result in economy.

e. The smaller amount of cement results in low temperature rise. As a result of which, less thermal stress occur in the dam body. Concrete cooling is not a necessity.
f. No tensile stresses have been obtained for many cases at the heel under seismic loading as a result of the symmetrical shape.
g. During design, only compressive stress may be taken into account, tensile stresses may be omitted.
h. Shear stresses are almost half of that of a traditional gravity dam section.
i. Aggregates can be obtained from vicinity. Random alluvium and weak quarry rock are appropriate for the construction if it is well graded.

1.5 Causes of Concrete Dam Failures

There are several factors, which directly and indirectly affect dam performance. Instrumentation enables engineers to monitor the desired parameters but it is not a feasible and reasonable study to measure all the properties. The most important characteristics, which may endanger the dam safety or lead the dam to uncontrolled release of water, must be carefully determined according to dam type and site conditions. Engineering skill and judgment are required for the selection of the critical parameters. Lesson learned by analysis of the past dam failures helps engineers to make better decisions. Although, no dam failure incidents took place in Turkey up to date, recent developments in context of dam safety and instrumentation should be closely followed. In the past nine centuries, over two hundred dams had suffered severe damage and had failed. Less than 20% was concrete dams (Bartholomew and Haverland, 1987). Common evidences of failures were detected as follows. Stress, instability, seepage, foundation and abutment induced problems. Remarkable dam failures are also introduced in this section.

1.5.1 Stress

Due to the monolithic behavior of the concrete dams, structural integrity is a necessity for the safety. When an earthquake, any rapid temperature change or any other detrimental effects occur, it may cause internal cracking, concrete cracking, crushing, and offsets in concrete monoliths (Bartholomew and Haverland, 1987). In the past, either poorly constructed or inadequately designed dams failed as a result of loss of the integrity by stress induced problems.

St. Francis Dam was a 62.5 m high arched gravity dam in California, USA. In 1928, the second year of the dam, due to poor foundation and internal cracking, the dam was collapsed and approximately 450 people died at Santa Paula (A.P., 1985). Austin Dam was 15.2 m high gravity dam in Pennsylvania, USA. In the second year of the dam, on 30 September 1911, the dam suddenly burst. The reason of the failure was concrete cracking and foundation sliding. 78 people were killed by the flood and the owner of the dam paid over $2.000.000 for negligence claims (Downs, 1998).

1.5.2 Instability

Mostly, instability problems occur due to excessive or unevenly distributed uplift pressures. Since a gravity dam relies on its weight for the stability, uplift pressures become a meaningful design consideration. Uplift pressures significantly reduce the compressive stresses on the horizontal plane, which may result in sliding or even overturning failures.

Bouzey Dam was a 21.9 m high gravity dam in France. In 1895, in the 14th year of the dam, the dam collapsed because of excessive uplift and internal hydrostatic pressures (Bartholomew and Haverland, 1987). Khadakwasla Dam was a 39.9 m high gravity and masonry dam in India. In 1961, the dam failed due to extreme uplift and internal cracking and ruined the city of Pune (Karunanidhi et al, 2007).

1.5.3 Seepage

Seepage inevitably occurs through, around and under the dams. The quantity mainly depends on foundation properties, site conditions and the quality of the construction. In concrete dams, vulnerable locations for seepage may be waterstops at monoliths and face slab, embankment sections, and foundation (Bartholomew and Haverland, 1987). Any sudden change in seepage quantity may indicate a severe incident. Seepage directly influences many parameters, such as hydrostatic pressures in the dam body, uplift pressure at the dam basin and so forth.

Vega de Tera Dam was a 34.1 m high, buttress and masonry dam in Rivadelgado, Spain. On 9 January 1959, the second year of the dam, because of the leakage in the construction joints and foundation-sliding dam failed (Bartholomew and Haverland, 1987). More than 150 people were killed in the disaster (A.P., 1985).

1.5.4 Foundation and Abutment Problems

Concrete dams are massive and rigid structures. The foundations must be strong enough to bear the heavy loads and the abutments must be able to resist the arch thrust. Many failures induced by foundation and abutment problems recorded due to piping of material from rock joints or solution channels, clogged drains, consolidation, sliding along bedding planes and movement at faults or shear zones (Bartholomew and Haverland, 1987).

Malpasset was a 66,5m high, double curvature arch dam built on Reyran River, in France. In December 1959, the dam explosively failed, flood wave ran along the river and reached Frejus (Goutal, 1999). As a consequence of flooding, 433 causalities were reported. The primary reason of the failure was weak abutment rock. High water pressures weakened the abutment support and the dam separated from its foundation as a whole and then collapsed (See Figure 1.2).

Figure 1.2. Remaining right abutment of Malpasset Dam

(Structurae, 2008)

As stated above, dam failures are severe and destructive events. Dam instrumentation is vital for monitoring dam performance and may allow us to take required remedial action. More detailed information about dam instrumentation is given in the following chapter.

CHAPTER 2

DAM INSTRUMENTATION

2.1 General

Dams are water barriers or embankments constructed across flowing water to store water. The impounded water is used for several purposes such as irrigation, water supply, hydropower generation, flood control, recharging of the groundwater and many other important economic benefits (Yanmaz, 2006). These structures are subject to enormous forces such as hydrostatic pressure, earthquake, uplift forces, etc. Dams are expected to safely withstand to the potential forces over many years. In case of a dam failure, the massive force of the released water would have a potential for the catastrophic destruction of the population and the environment. Dam safety is essential for the economic benefits and public safety. A proper and safe operating dam could be attained by correct instrumentation, which enables engineers to observe the performance and safety of the dam at the stages of construction, first filling and service.

2.2 Benefits of Dam Instrumentation during Construction

The benefits during construction can be grouped according to safety, economy, legal protection and public relations (Dunnicliff and Green, 1988).

2.2.1 Safety

Safety is the most significant parameter to be considered during design and construction phases. Instruments provide measurements that indicate the behavior of the construction. Because of that, engineers periodically compare the recent readings with respect to the threshold limits to be on the safe side during construction. If any unexpected adverse effects occur, instrumentation permits remedial action to be taken before continuing the construction.

2.2.2 Economy

In the design stage, due to uncertainties in engineering properties and dam behavior, some predictions, which affect the procedures and schedules, must be made. These predictions can be compared to the actual measurements to modify the schedule. For instance, the temperature rise as a result of hydration reaction after the placement of concrete can be observed via instrumentation. Internal temperature tends to decrease as the peak value has reached in several days. Monitoring internal temperature by embedded instruments helps to decide on both duration of artificial cooling of mass concrete and availability of detachment of forms for the next concrete pouring stage. Hence, engineers may easily modify the construction schedule with respect to the observations. Therefore, if correctly planned and properly installed, instrumentation not only ensures safety but also reduces the construction cost.

2.2.3 Legal Protection and Public Relations

The construction of a dam may affect the neighboring structures, environment, and population. Instrumentation allows engineers to collect the measurements by creating a valid data bank. If any adverse effects caused by dam construction occur, collected data can be used in litigation (Dunnicliff and Green, 1988). In other words, dam will gain legal protection against any damage claims. Mostly, dam projects draw attention because of size, submergence, environmental effects and so on. Therefore, the community reactions and political obstacles may appear from the beginning until the end of the construction. These negative events can be minimized by presenting an instrumentation program to emphasize how the dam would be carefully watched at each step. By enhancing public relations, delays are somehow reduced during dam construction and operation.

2.3 Benefits of Dam Instrumentation during Operation

The benefits during construction can be grouped according to evaluation of dam performance compared to design, diagnosis of abnormal dam performance, and design of early warning systems (Dunnicliff and Green, 1988).

2.3.1 Evaluation of Dam Performance Compared to Design

Dam design is much more complex than any kind of civil engineering structure. There are so many parameters, which are uncertain. Hence, in the design phase, material properties are carefully inspected in laboratories and structural analyses are conservatively carried out in computer medium. After that, engineers are supposed to make appropriate predictions and assumptions to be on the safe side. Instrumentation plays a major role in verifying the design assumptions and expectations.

2.3.2 Diagnosis of Abnormal Dam Performance

Abnormal dam performance can be detected via visual observations and/or instrumentation. Dam owners and engineers desire to have an instrumentation system which is able to continuously yield data that indicates whether the dam is performing in a satisfactory manner or not. The factors that can threaten the safety of the dam are carefully watched by the monitoring system. If the readings fall outside the expected range which can lead a failure or a severe distress, remedial measures must immediately be taken. Sometimes it could be too late for the corrective actions but the lessons learned from the disasters are quite valuable to obtain the actual cause of the failure.

2.3.3 Design of Early Warning Systems

Loss of life may reach thousands of lives and damage may cost millions of dollars in case of a dam failure or even a partial failure. Especially large dams have potential danger for the floodplain population and property. A proper instrumentation and measurement system provides quite valuable output to establish an early warning system. The goal of such systems is to reduce loss of lives and damage which caused by dam failure. An instrumentation system for a proposed dam will increase total construction cost about 2-3% according to requirements (Bartholomew and Haverland, 1987). However, the value of added safety makes the instrumentation program cost effective.

2.4 Determination of Parameters

The first step of the instrumentation design is the determination of the parameters to be measured. The determination is based on the project requirements. Clearly understanding of purpose, design assumptions and potential problems of dams are essential to provide an efficient instrumentation system. Mostly, designers who best understand the behavior decide on the monitoring system. Risk assessment and hazard classification analysis of dams are also beneficial to obtain minimum system requirements. Each company offers several systems for any type of dams. Figure 2.1 illustrates possible alternative instruments to be installed on a conventional or RCC gravity dams.

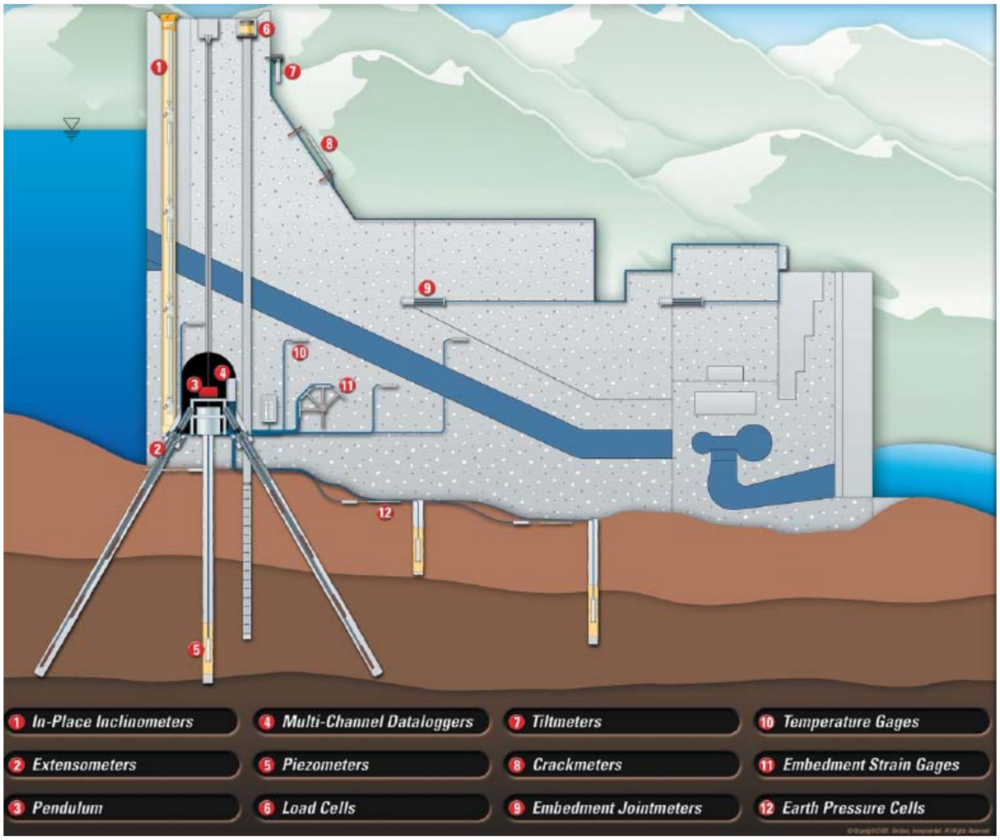

Figure 2.1. Possible alternative instruments to be installed at a gravity dam (GEOKON, 2008)

2.5 Selection of Instruments

Proper selection of correct instruments is vital for a well functioning instrumentation system. In this section, basic concepts regarding instruments and detailed information of sensors will be presented.

2.5.1 Basic Concepts

Any engineer dealing with instrumentation should be familiar with basic concepts to provide maximum efficiency from the instrumentation system. These concepts are summarized below;

Range: The minimum and maximum possible values that an instrument is able to measure.

Accuracy: The degree of conformity of a measured or calculated value to its standard or absolute value. It is an indication for an instrument which performs within the limits of the specified range.

Precision: The degree of mutual agreement among a number of similar measurements. It has a greater importance for instrumentation systems since the change is the primary interest.

Repeatability: The variation in consecutive measurements, which have been performed by the same instrument under the same operating conditions.

Resolution: The smallest increment in a measurement that can be discerned (ASCE, 2000).

An ideal reliable measurement device must be accurate, precise and repeatable within the specified range. There are some other significant factors, which may be taken into account to minimize the potential problems during installation, operation and maintenance (ASCE, 2000).

a. Magnitude and range of the parameters should be estimated correctly to enable proper selection of instruments.
b. Instruments must be properly located and installed to provide representative data for the chosen parameter.
c. Each instrument is susceptible to damage especially during installation and right after the installation. Proper protection measures must be taken at every stage of the dam construction.
d. Redundancy must be taken into account to improve the reliability of the system and to extend the functioning life of the monitoring system.
e. Accessibility of the instruments must be considered since this parameter directly affects the installation, maintenance and data acquisition methods.

f. Some instruments should not be preferred just because of their suitability for the automatic data acquisition. Most of the instruments may be easily adapted to automatic data acquisition.
g. The extreme weather conditions must be considered as well to verify the operating temperature range of each instrument.
h. Power requirement of the system must be evaluated carefully, particularly for automated data acquisition systems.
i. Durability, simplicity and satisfactory long-term performance are the other significant criteria that must be considered for instrument selection.

2.5.2 Hydrostatic Pressure Measurement Devices

Measurement of hydrostatic pressure is the most common application for all type of dams. Reservoir, tailwater and the ground water levels are the main variables, which influence the magnitude and the distribution of the water pressure. Before decision-making on the critical locations, the main characteristics of the site such as permeability, strata variations and regional features must be properly investigated. Site modifications e.g. drains, grout curtains, etc., must also be taken into account.

Various types of instruments are installed to several dams at significant locations to observe water pressures. Instruments embedded within concrete or soil serve for the detection of pore water pressures. The uplift, which is defined as upward pressure of water at the base of the dam, is commonly monitored at both dam foundation contact and foundation level. Piezometers are extensively preferred for measuring water pressures. Piezometers are mostly installed at abutments and foundations of concrete dams. The goal of such applications is to observe seepage and uplift pressures. Concrete dams located in broader valleys require embankments either single or each end. Monitoring pore water pressure within the soil of embankment portion is essential to ensure satisfactory performance

(Bartholomew and Haverland, 1987). Piezometers are mainly divided into two groups; open and closed systems. In the following section both type of instruments will be described, in detail. Reservoir and tailwater level measurements are commonly carried out via staff gauges, float-type water level gauges and pressure transducers.

2.5.2.1 Open System Piezometer

These are relatively simple instruments, which are installed into boreholes at foundations and abutments. Generally, observation wells and open standpipe (Casagrande piezometers) are preferred to measure water pressure for concrete dams.

2.5.2.1.1 Observation Wells

In current practice, observation wells are used to detect the initial groundwater pressures and seasonal fluctuations. These wells consist of PVC or steel pipe, a filter at its lower end, which allows water to pass and rise in the pipe, a protective seal at the upper end to prevent surface runoff from entering the borehole and a vented cap to eliminate the barometric effects in the pipe.

The measurement of water level inside an observation well is generally taken manually by a water level indicator, which is commonly called as dipmeter. A dipmeter consists of an electrical cable graduated for the desired sensitivity. The cable is lowered into the piezometer tube, when the tip touches the water surface; it activates a buzzer or signal lamp located on the tape reel. This activation position indicates the water level depth and can easily be read from the graduated table (SIS, 2008). Although automatic data acquisition is available, it is rarely used because of the complication of the application.

The backfill material for observation wells is permeable, commonly coarse-grained sand or gravel. As a consequence of this, observation wells create an undesirable vertical connection between all aquifers through which they are installed. Thus, the measurements commonly indicate the head in the most permeable zone and not reliable. The data obtained via observation wells only represent an average piezometric pressure.

The water quantity needed to obtain measurement is one of the main features for the selection of the piezometers. The term, *hydrodynamic time lag* is defined as the response time for the pore water pressure change with respect to the corresponding change of water level inside the well (Dunnicliff and Green, 1988). It is primarily dependent on the permeability of the medium and the diameter of the pipe. Observation wells should not be preferred in clay or other low permeability soils due to their slow response time or long hydrodynamic time lag. The main reasons for frequent use of observation wells are their simplicity and longevity. They are also inexpensive and no skilled personnel are essential since installation is typically implemented into the drilled holes. Installation for the embankment portions of concrete dams requires additional measures. Under the construction of an earth embankment, sufficient compaction around the well must be provided to prevent formation of a sinkhole (ASCE, 2000).

2.5.2.1.2 Casagrande Standpipe Piezometers

A Casagrande standpipe piezometer system is almost identical to observation wells within operating principle and the components. These types of piezometers are used to detect the hydrostatic pressures at the desired strata. It is achieved by providing bentonite seals either above or both ends of the inlet pipe. They are commonly preferred for the intermediate permeability soils. The advantages and disadvantages are summarized in a Table 2.1.

Table 2.1. Advantageous and limitations of various hydrostatic pressure measurement devices (Adapted from Bartholomew and Haverland, 1987)

<table>
<tr><th>Instrument</th><th>Typical Specifications</th><th>Advantageous</th><th>Limitations</th></tr>
<tr><td>Observation Well</td><td rowspan="2">PVC or Steel
3.0 m length each pipe
50 mm outside diameter (OD)
40 mm internal diameter (ID)</td><td>Simple
Easy to install
No skilled workmanship required</td><td>Long time lag
Creates undesirable vertical connection between strata</td></tr>
<tr><td>Standpipe Piezometers</td><td>Simple, Reliable
Long Experience Record
Self de-airing if ID is sufficient
Can be used for measuring permeability, sampling groundwater
Can be converted to other type open system piezometers</td><td>Long time lag
Freezing problems
Pipe and tubing must be placed almost vertical
Subject to damage by construction equipment
Costly drilling</td></tr>
<tr><td>VW Piezometers</td><td>Stainless Steel
25 mm diameter
215-275 mm length
0.5-1.0 kg</td><td>Short time lag
Simple to read, maintain
Readout unit location independent of tip level
No freezing problems
Suitable for manual or automatic readings
Can be used for reading negative pressures
Data can be transmitted over long cables</td><td>Sensitive to temperature and barometric changes
Susceptible to lightning damage
Extensive attention required during manufacturing</td></tr>
</table>

2.5.2.2 Closed System Piezometers

Closed system piezometers are commonly used to observe pore water pressures. These types of piezometers have a rapid response time, in other words short hydrodynamic time lag, so they are capable of detecting the small pressure changes even in low permeable soils. There are several instruments available commercially, but vibrating-wire (VW) devices are widely preferred especially for concrete dams. VW piezometers have been extensively used on dams throughout the world. They have served for several applications including measurement of ground water levels, measurement and control of permeability and monitoring uplift pressures at the base of the dams. There are various models of VW piezometers such as various diameter versions, filters with different porosity in order to suit any kind of applications. A typical VW unit is composed of a stainless steel piezometer tip, backfill materials, electrical cable and readout unit. Typical VW Piezometers are shown in Figure 2.2.

Figure 2.2. Various types of VW piezometers (SIS, 2008)

The sequential steps of the operation may be summarized as follows (See Figure 2.3):

a. Porous filter enables water to enter and press against the pressure sensitive diaphragm.
b. A tensioned high-strength wire is coupled to the diaphragm at one end and to a wire grip at the other end in a hermetically sealed medium.
c. The diaphragm deflects due to applied water pressure, as a result of this, wire tension changes.
d. An electromagnetic coil plucks the wire and enables vibration and reads the frequency of the vibration.
e. Pore water pressure is calculated based on the vibration frequency of the wire.

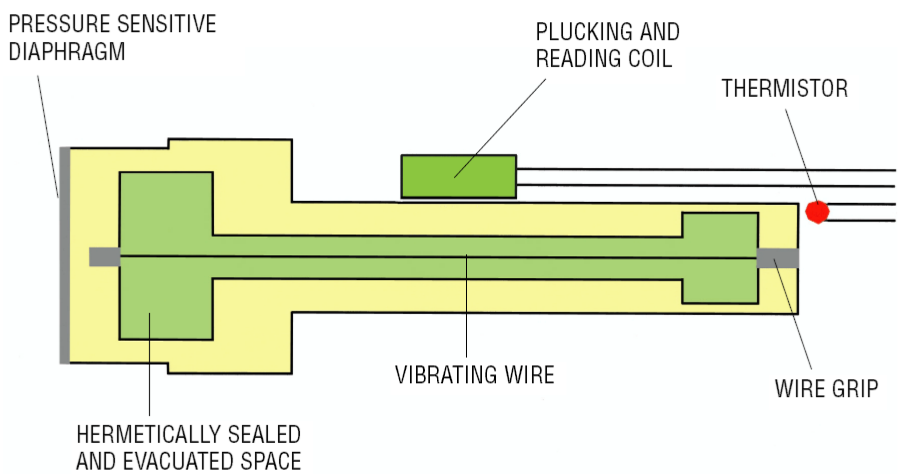

Figure 2.3. A typical VW piezometer tube (SIS, 2008)

The equation for the frequency of vibrating wires (Hawkes and Bailey, 1973) is;

$$f = \frac{1}{2L}\sqrt{\frac{\sigma \times g}{\rho}} \tag{2.1}$$

where,

f :natural frequency (sec^{-1})

L : length of the vibrating wire (in.)

σ : stress in the wire (lb / in.^{2})

ρ : density of the wire material (lb/in.^{3})

g : acceleration due to gravity (in. /sec^2)

To include wire strain; ($\sigma = E \times \varepsilon$)

$$f = \frac{1}{2L}\sqrt{\frac{E \times \varepsilon \times g}{\rho}} \tag{2.2}$$

where,

E : modulus of elasticity of the wire (lb/in.^{2})

ε : strain in the wire

Thus,

$$\varepsilon = Kf^2 \tag{2.3}$$

in which $K = \frac{4L^2\rho}{Eg}$

As can be seen from Equation (2.3), K is a constant, which depends on the length, diameter and characteristic of the wire material. The initial tension under zero pressure of the wire, f_i and the reading under actual pressure, f are known. The strain can be calculated via the following equation;

$$\varepsilon = K(f^2 - fi^2) \tag{2.4}$$

The pressure obtained from the readout unit is an approximate value. Corrections should be taken into consideration for barometric pressure and the temperature.

The main features of VW piezometers are as follows (Bartholomew and Haverland, 1987).

a. Simple maintenance and long-term stability.
b. Since the undesirable effects, such as cable resistance, contact resistance have been minimized, the frequency signal can be transmitted accurately over long cables.
c. The location of the readout unit is independent of the location of the piezometer tip and relatively easy to read.
d. Short time lag enables the practice even in impermeable soils.
e. Ability to monitor negative pore water pressures.
f. No freezing problems.
g. Wire corrosion problem has been solved via hermetically sealed tip.
h. Vulnerability for lightning damage must be considered.
i. Sensitive to temperature and barometric changes.

Right after installation, calibration has been performed by technicians under “no load” condition. It is desirable for the calibration to match “zero” reference point without any drift or shift. Any subsequent change from this no-load "zero" reference point is defined as zero-drift. Generally, creep of vibrating wire or slippage wire grip limits precise measurements. Intensive attention has been paid by the manufacturers to minimize zero-drift. Stresses relieving operations e.g. high temperature or loads cycling (for wire, clamps and transducer cap) have been carried out to provide long-term stability (Dunnicliff and Green, 1988). VW piezometers have been preferred among dam engineers to monitor pore water pressures in both foundations and abutments. These instruments achieved an outstanding reputation over years (ASCE, 2000). Research and development departments of instrument companies have developed new devices such as fiber optic piezometers. Due to relatively higher prices, they are not commonly used.

2.5.3 Stress Measurement Devices

Stress measurements between dam and its abutments or between components of dam are extremely important in some cases, especially for verifying the design assumptions. Generally, stress is monitored indirectly since it is impossible to measure directly for the most cases. Strain has been monitored via available instruments and then stress has been calculated according to the measurements.

Mostly, strain gauges also named strain meters have been used for stress measurements. Strain gauges can be divided into two types; surface and embedded (internal) gauges. Surface gauges have been served to monitor long-time static strains on the structures. Mechanical and electrical instruments are more suitable for monitoring surface strains. Measurements have been implemented with respect to reference points located on the structures.

Internal stress distribution and orientation have been observed by means of embedded strain gauges. There are mainly two types of embedded strain gauges, which have been extensively used; electrical resistance (ER) and VW types. ER strain gauges operate on the principal of changing resistance in the internal wire as the length changes. Engineers have experienced some difficulties in the past especially for temperature compensation coefficient of the ER instruments. ER type strain gauges are not organized to a specific resistance at a reference temperature. Each instrument requires separate calibration data for temperature compensation hence data acquisition becomes more complex. ER type instruments have been abandoned by developed countries. VW strain gauges are the most requested instruments for strain measurements (ASCE, 2000).

Total pressure cells, also called, earth pressure cells are able to measure stress directly. Generally, these instruments have been used for monitoring total pressure in between structural components, dam and foundation contact and in soil for embankment portions of concrete dams. Earth pressure cells can be used either fixed to the contact surface between soil and structure or embedded in concrete mass. ER and VW types are commercially available but VW type will only be introduced herein, the disadvantageous of ER type has already been explained above.

2.5.3.1 Relationship between Stress and Strain

The strength of the materials is usually expressed in terms of stress. In order to determine stress, a non-dimensional term strain is commonly monitored. A structural member subjected to either tensile or compressive stress undergoes a dimensional change. Strain is defined as the ratio between the change in length due to applied load and the original length. In a typical stress-strain curve, there are two regions separated by a yield point. Deformations up to yield point are called elastic and material is able to return its original shape after loading. Above this point, in the plastic region, applied load causes a permanent deformation. Hooke's Law is only valid in elastic region where stress is directly proportional to the strain. Related equations (2.5) and (2.6) are given below.

$$\varepsilon = \frac{\Delta L}{L} \tag{2.5}$$

where,

ε : Strain

ΔL : Change in length due to applied load

L : Original length before loading

Hooke's Law stated that

$$\sigma = E\varepsilon \tag{2.6}$$

in which,

σ : Stress (Pa)

E : modulus of elasticity of the material (Pa)

ε : Strain (μm / m)

The applied load can also be calculated by multiplying σ and the cross sectional area subject to stresses.

2.5.3.2 Embedded VW Strain Gauges

Strain gauges are embedded within the mass concrete to monitor stresses at the locations of interest. Strain gauges have been manufactured in different dimensions and types to be able to answer several questions. The instrument size is dependent on the maximum size of the aggregate. In order to attain reliable measurements, the length of the strain gauge must approximately be 3-4 times of maximum aggregate size (ASCE, 2000).

VW strain gauges use the same operating principle as all other vibrating wire sensors. In a protective stainless steel tube, a tensioned wire is firmly fixed between two end flanges. Either tensile or compressive stresses within concrete body affect the embedded strain gauge and cause relative movements of the flanges. Thus, tension of the wire is altering with respect to the magnitude of the applied load. An electromagnetic coil plucks the wire, enables vibration and reads the frequency of the vibration. Then, strain can be calculated by multiplying gauge factor and the square of frequency difference as given in Equation (2.3). Various types of strain gauges are shown in Figure 2.4.

Figure 2.4. Various types of strain gauges (SIS, 2008)

Embedded strain gauges can be installed into mass concrete with different configurations. Single gauge installation only measures strain in one direction, multiple gauge frame or strain gauge rosettes are two or more closely positioned gauge grids, which have been formed to measure the normal strains along different directions. The goal of the applications is to obtain principal strains and directions.

Generally, biaxial stress state which typically has vertical and horizontal principal directions occur in structural members. Since the state has known in advance, two independent strain measurements are essential to determine the principal strains and stresses. In case the principal directions are unknown beforehand, at least three strain gauges in different directions are required to obtain principle strains and stresses.

The initial step of the design is the selection of strain gauge rosette configuration that is proper for the task. There are several rosette configurations such as simple, rectangular, delta, pyramid, and spider as introduced below (Bartholomew and Haverland, 1987).

a. Simple rosette configuration consists of two strain gauges which are located mutually perpendicular in two principle directions.
b. Rectangular rosette configuration has been formed of three strain gauges installed in three different axes (See Figure 2.5).
c. Delta shaped rosette configurations are functionally equivalent with rectangular configuration. These rosettes are presumed to illustrate the optimum sampling of strain distribution because of maximum possible uniform angular separation (effectively 120^o).
d. Spider rosette configuration is a specific combination of pressure cells and strain gauges. They are commonly used in arch dams to monitor horizontal, vertical and angular orientation of stresses.

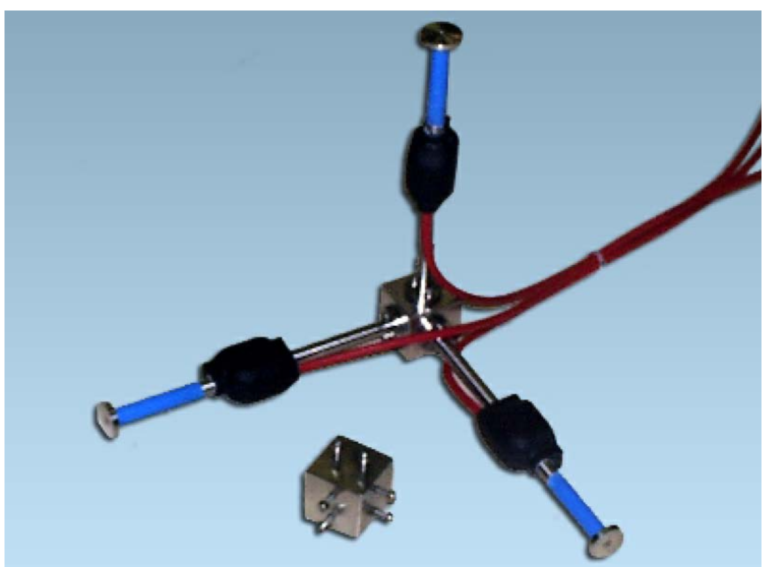

Figure 2.5. A typical 3-D rosette assembly and rosette mount (SIS, 2008)

Additional strain gauge(s) may be inserted to rosettes or installed adjacent point(s) to provide some redundancy. In case of malfunction, either one or more strain gauges of the rosette, redundant gauges still allow to monitor principal stresses. Moreover, attention must be given to geometrical irregularities such as galleries, gutters, holes, etc., while locating both rosettes and redundant strain gauges. Prior to concrete pouring, strain gauges must be tied to rods to fix them in proper place. Any surrounding member must not interfere with the gauges and concrete vibration must be implemented away from the gauges.

2.5.3.3 Total Pressure Cells

Total pressure cells have been commonly installed in between the interfaces of concrete dam components and between dam body foundation contacts. These instruments not only enable engineers to verify design assumptions but also to determine the orientations and distribution of the stresses. Pressure cells (PC) are designed to directly measure only compressive stresses. Mostly, vertical stresses have been monitored. In some instances, especially for concrete arch dams horizontal stresses normal to the thrust direction have been also monitored via PC (Bartholomew and Haverland, 1987).

A typical PC consists of pressure pad, steel tube, and pressure transducer, as shown in Figure 2.6. Pressure pads are composed of two stainless steel circular plates with hydraulic fluid filled inside. Hydraulic tube is also filled with either oil or glycol and act as a bridge between pressure pad and the transducer. External pressure causes deflection on the sensitive plates; steel tube transmits the altering pressure to the transducer, transducers using VW principle convert the pressure to the electrical signal and allow readings via readout units.

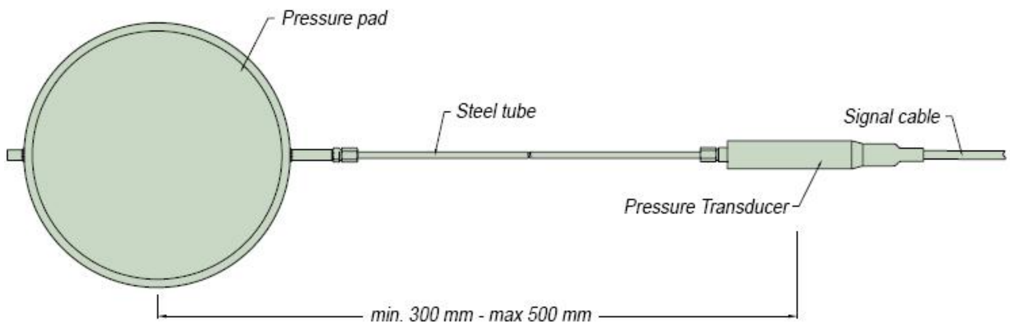

Figure 2.6. Top view of a typical hydraulic pressure cell (SIS, 2008)

Two major factors must be taken into account to obtain maximum efficiency from PCs; selection of the type of instrument and arching effect. Firstly, attention must be paid for the selection of the type of instruments. Pressure cells are commercially available as diaphragm or hydraulic types. In a diaphragm type PC, a circular plate is supported via a steel ring and deflections are sensed via a central internal strain gauge. As to hydraulic type PC, the pad is filled with special fluid and alteration in pressure is read via a pressure transducer. Diaphragm type PCs are negatively influenced by point or unevenly distributed loads and may report questionable measurements. This negative effect is minimized for a hydraulic pressure cell because of its operating principle. Both types are manufactured with VW and ER transducers.

Hydraulic type PCs are also used for concrete dams in rectangular and circular pad shapes. Typical pad dimensions are 20 cm in the diameter and 12 mm in thickness. The recommended ratio between the thickness and the diameter is almost 1/20 (ASCE, 2000). Point loads are the primary reason of non-uniform loadings. Some instrumentation companies have manufactured thicker plated pressure cells to minimize any point load effects. In addition, sufficient compaction of fill material around the pressure pad is essential to obtain reliable measurements.

Secondly, installation must be carried out with required equipment by qualified personnel. In order to achieve adequate performance from PCs, the complete contact between the pressure pad and the adjacent concrete must be provided (Bartholomew and Haverland, 1987). After installation of an embedded pressure cell, the most common problem encountered is the loss of contact between the concrete and the pad interface which is also called arching effect. Due to arching effect, the stress around the cell is monitored instead of the stress acting on the pressure cell. In order to overcome this problem, a special process, re-pressurization, is carried out. Right after the concrete curing completion, a special pump is connected to the re-pressurization valve and hydraulic fluid is gradually filled until the sufficient pad-concrete contact is achieved (See Figure 2.7). Moreover, the stiffness of pressure pads must be almost identical to the medium. Pressure pads with low stiffness may report misleading measurements due to arching effect. It is a common problem for embedded instruments in high modulus media, such as rock and concrete.

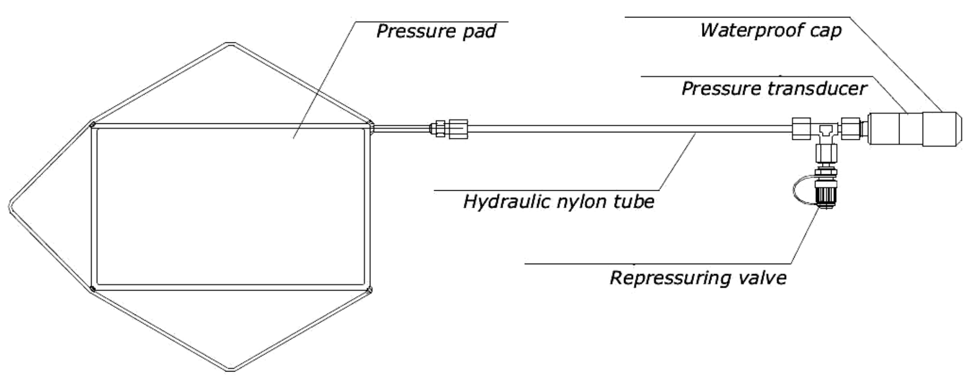

Figure 2.7. A typical concrete stress cell with re-pressurization valve

(SIS, 2008)

2.5.4 Seepage Measurement Devices

The head difference between upstream and downstream side of the dam is the primary cause of seepage. Seepage measurement is vital to detect the quantity of water leaking through, beneath or around a dam. Instruments serving for seepage measurement are commonly located at foundation, joint and face drain outlets. Seepage rate is directly depends on permeability and the pressure.

The amount of seepage is expected to be proportional to reservoir water level, which is the main variable for pressure changes. Right after a heavy rainfall, seepage rate normally increases because of the rise in the reservoir level. If this is not the case, drains might have been clogged or seepage is occurring in other locations instead of where being measured. Occasionally, seepage quantity may be greater than the estimated value. For instance, during cold weather much water leaks through additional cracks as a result of thermal contraction. Thus, seepage rate increases rapidly not parallel to the reservoir level. Hence, prior to decision-making the remedial actions, the actual cause must be clearly investigated (Hansen and Reinhard, 1991).

Water quality and clarity is also important because e.g. cloudy, discolored water may be an indicator of progressive erosion in dam, foundation or abutments. Therefore, not only amount of leakage but also the composition of water is a signal for satisfactory performance of dams. Generally, weirs, flumes, calibrated containers and flowmeters are widely used devices throughout the world in dam instrumentation. Detailed information will be presented in succeeding sections.

2.5.4.1 Weirs

Weirs are often used for open channel flow measurement. Typical applications include measurement of irrigation water, wastewater treatment plant discharge and measurement of seepage flow for dams. For concrete dams, weirs are generally installed in gallery drainage gutters to measure total cumulative flow above each weir. These instruments are classified in accordance with shape of the notch and commercially available in V-notch, square, rectangular and trapezoidal shapes.

The triangular sharp crested or V-notch weirs are the most precise instruments particularly for small flows. A typical 90^{o} V-notch weir consists of a stainless steel thin plate with sides of the notch being beveled 45 degree immediately downstream of the crest (See Figure 2.8). V-notch weirs operate as contracted weir. The gutter upstream must have a reasonable slope (1-3%) and minimum upstream distance from the instrument must be long enough to obtain accurate measurements (GW, 2007).

The lowest point of the edge over which water flows is almost zero for V-notch weirs. Since they have no crest length, nappe will spring free of the crest if the pocket beneath the nappe is adequately ventilated. Generally, either ventilated air pockets or vent pipes are arranged under the nappe to provide air circulation. Weirs are highly susceptible to sediment accumulation and algae formation, therefore, regular maintenance and flushing is required.

Figure 2.8. A typical 90^o V-notch weir (SIS, 2008)

A typical 90^o V-notch weir is able to accurately measure a discharge 10 lt per second. For greater discharges rectangular, for smaller discharges either 22.5^o or 45^o V-notch weirs are more accurate. The flow rate is directly measured with respect to the head on the bottom of the V-notch. Manual measurements are commonly carried out via graduated staff gauges. Staff gauges mostly graduated in millimeters and zero point is placed at the same elevation as the weir crest. Water flowing over the weir forms a downward curve on the top surface extending upstream which is called drawdown effect. A suitable location beyond the drawdown effect region should be chosen to obtain reliable measurements. There are various equations to compute the discharge over weirs.

Automatic measurements can be performed via pressure transducers or level transducers (See Figures 2.9 and 2.10). A pressure transducer unit is designed to measure water level at the stilling basin of a weir. Flow rate can be calculated corresponding to the water level measurement. A level transducer is mainly consisting of a load cell and floating object. Floating object placed at stilling basin of the weir is housed inside a rectangular shaped steel frame. The floating object, influenced by water level fluctuations, physically transmits the level variation to the load cell. The pressure sensed via load cell can easily be transformed to discharge by using simple equations. It is always recommended to provide automatic measurement instruments to better apprehend the seepage rate in a gutter (SIS, 2008).

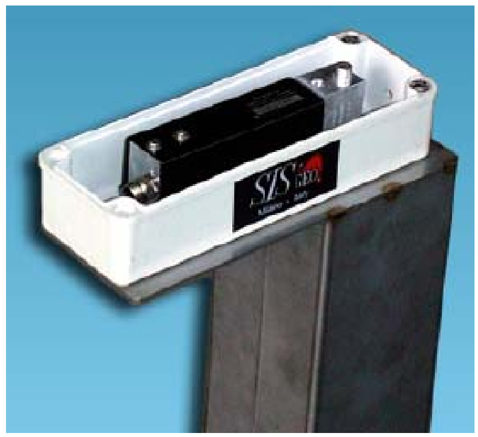

Figure 2.9. Level transducer unit (SIS, 2008)

Figure 2.10. Pressure transducer unit (SIS, 2008)

2.5.4.2 Parshall Flumes

Parshall flumes are specially shaped sections located in a channel to measure the flow rate. They are best suited to rectangular channels. These instruments are made of either reinforced fiberglass or stainless steel and permanently installed in concrete or earthen channels. Parshall flumes are composed of a converging upstream section, a downward sloping throat and diverging downstream section. Firstly, the incoming flow is accelerated at the converging upstream section; secondly, flow follows a lowering gradient at the downward sloping throat, which enables the measurement of discharge. A typical Parshall flume is shown in Figure 2.11.

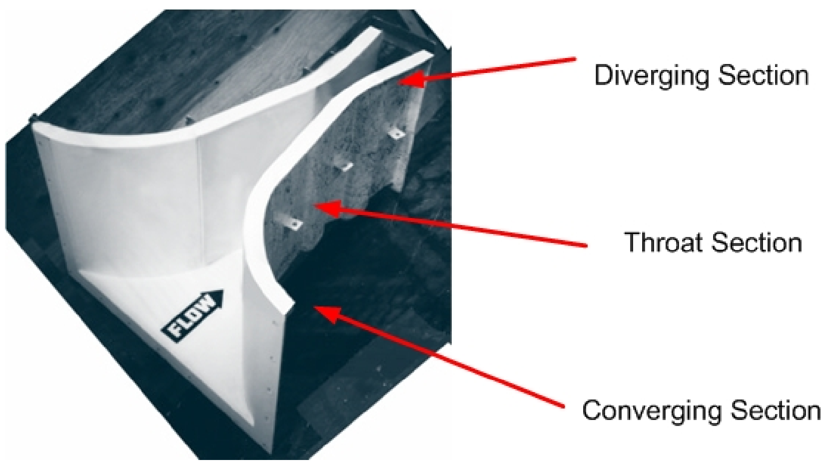

Figure 2.11. Parshall flume with curved-inlet wing walls (PF, 2001)

Parshall flumes are capable of operating both free-flow and considerable submerged-flow conditions because of inclined throat and the diverging section. Discharge through a Parshall flume can be determined either by an equation or by specified table. The head at the converging section is sufficient to compute the flow rate for free-flow condition. These flumes are reliably operable up to 50 or 80% submergence (depending on the size of the flume), but two variables; head at the converging and throat sections are required to determine the discharge. Flumes are preferable to weirs because of relatively low head loss (Bartholomew et al, 1987). Self-scouring design eliminates the sedimentation problems. Other advantageous of Parshall flumes are their low maintenance and long service life.

2.5.4.3 Calibrated Containers

The simplest instruments for measuring seepage discharge are calibrated containers. Flow rate is computed according to the time required to fill a known container volume. Calibrated containers are effective in low flow conditions. Leaking water should be diverted to container by a pipe or other possible application. Calibrated containers have a limited practice since they are appropriate for flows about three liters per second. Typical specifications are given in the Table 2.2.

2.5.4.4 Velocity Meters

Several velocity meters are available in the market to measure flow rate in pipes and open channels. These instruments are categorized by their operating principles; mechanical, vortex, electromagnetic, ultrasonic etc. While in contact with water flow, each velocity meter produces special signals, which indicate the velocity of the flow. The dimensions of the channel and the flow depth are essential to compute the discharge. Permanent and portable types exist but portable ones are commonly preferred because they require less maintenance. Precise measurement may not be accomplished via velocity meters since the velocity of the flow varies across the stream profile.

Table 2.2. Advantageous and limitations of various seepage measurement devices (Adapted from ASCE, 2000).

Instrument	Typical Specifications	Advantageous	Limitations
V-Notch Weir	Stainless Steel 150-300 mm width 200-300 mm height Discharge range for 90^{o} V-notch	Simple, Reliable No nappe clinging problems Inexpensive Suitable for manual and automatic data acquisition	Relatively high head loss Sedimentation problem Algae formation problem
Parshall Flumes	Stainless Steel or Fiberglass Throat width 25 mm-10 m	Simple, Reliable Little maintenance Relatively low head loss No sedimentation problem Operable for submerged flow condition	Relatively expensive Installation
Calibrated Containers	1.9, 3.8, 7.6 liters capacity Discharge range around 3 lt/s	Simple Accurate for low flows	Limited use Require diversion
Velocity Meters	Mechanical, Vortex, Ultrasonic, Electromagnetic types available	Easy, quick, accurate for average flow measurement	High maintenance for permanent models

2.5.5 Movement and Deformation Measurement Devices

Dams have experienced movement as a result of applied enormous forces. Significant movements typically occur in vertical, transverse horizontal and longitudinal horizontal directions. It is desired to observe the magnitude and rate of these movements to be sure that they are within the acceptable range. Movements can be assessed in two types; surface movement and internal movement.

Surface movement is defined as horizontal or vertical movement of a point located on the surface of a dam with respect to a reference point that is installed on a stable location. Various surveying methods are commonly used for the measurements. Internal movement refers to either horizontal or vertical movement, which occurs within the dam. There exist particular instruments to observe any kind of change within the structure. Pendulums, inverted pendulums, inclinometers, tiltmeters, extensometers, joint and crackmeters are the mostly used instruments for dam monitoring.

Concrete dams have experienced vertical deformations commonly due to self-weight of the structure, reservoir load and consolidation of foundation materials. Transverse and longitudinal movements take place as a result of temperature or moisture fluctuations of the concrete, alkali aggregate reaction, uplift forces, reservoir load, earthquakes, etc. Rotational movements have been witnessed because of the low shearing strength of the foundation rock (Bartholomew and Haverland, 1987).

2.5.5.1 Joint Meters

Joint meters have been used for monitoring the altering distance across joints and cracks in concrete structures. These instruments are commercially available as embedment and surface types with VW and ER transducers. Embedment joint meters are installed between concrete lifts to observe relative movement. Surface types have been utilized for monitoring joint movement at either single or multiple directions. Surface joint meters, also called crackmeters are mostly used for concrete dam instrumentation. A typical joint meter is mainly composed of a stainless steel body, a steel rod and a signal cable. The waterproof stainless steel body covers the transducer. Steel rod is the sliding part of the instrument, which allows sensing both closing and opening of the joint. Signal cable is the connection unit for the suitable readout unit. Operating principle of these devices is quite similar to strain gauges.

Prior to installation of instrument, at least two points are drilled at the opposite side of the joint. Then, two anchors are grouted or bolted into drilled holes to fix the joint meter to the structure. Finally, instrument is attached to the anchors with ball joints. Ball joints enable compensation of eventual cross displacements. Right after installation, initial reading must be implemented (SI, 2008). A typical jointmeter is shown in Figure 2.12.

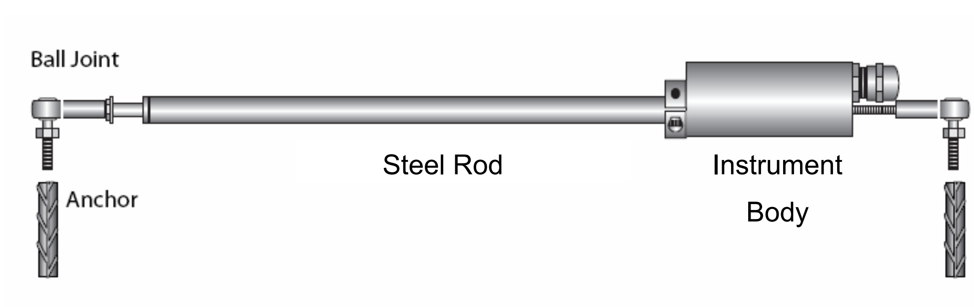

Figure 2.12. A typical joint meter (SI, 2008)

2.5.5.2 Survey Monuments

Monuments are movable points located on the surface of the dam. A stable reference point located off the structure must be established to perform measurements. The magnitude and the rate of horizontal and vertical surface deformations can be measured either by conventional or specific surveying techniques.

There are various factors affecting the performance of measurements that must be taken into account (Bartholomew and Haverland, 1987).

a. Piers of reference points must be set on a solid, stable and easily accessible location.
b. Durability of both monuments and reference points must be carefully considered since they are subjected to several negative effects, such as traffic, rain runoff, erosion, freeze-thaw, etc., which may distort accuracy of measurements.
c. Monuments must be periodically cleaned and kept discernible from any reference points.
d. It should be considered that dams constructed in the regions where environmental and climatic conditions are tough, the practicability and accuracy of the measurements are questionable.
e. The measurements are commonly evaluated less accurate than any developed instruments, such as inclinometers, plumblines, etc. Frequently, surveying methods are considered adequate if the measurements are carried out by well-trained surveyors.
f. Monuments are capable of monitoring vertical, horizontal, axial and surface deformations. It should be noted that rotational movements cannot be detected via monuments.

2.5.5.3 No-Stress Strain Gauges

No-stress strain gauges, also called dummy gages, are useful devices to obtain reliable data from conventional strain gages. They have been commonly installed close to embedded strain gages to observe the volumetric changes. Conventional strain gauges directly monitor the strains caused by true loading of the concrete while dummy gages are concerned with the strains in case of no-load related situations.

A typical dummy gage consists of a customary strain gage housed in a rectangular steel box. Each side of the box is covered with soft material like polystyrene foam to absorb external loads applied to the internal strain gage. The box has only one open end, which permits concrete to be filled inside during pouring. The internal strain gage only serves for monitoring strains related to absence of loading cases, such as volumetric changes due to temperature, chemical and moisture fluctuations (ASCE, 2000).

Concrete dams exhibit expansion due to hydration heat especially in the first 28 days right after concrete pouring. Generally, this hydration effects last few weeks and become stable in a long period. No-stress strain meters experience loadings because of hydration reaction from pouring of concrete to the end of their service life. Installation of dummy gages adjacent to the conventional strain gauges is a common practice. The location of additional gages must be properly determined not to allow modifying stress distribution of any gage.

The combination of these instruments in a layout enables correlation between measurements. The readings monitored via no-stress strain gauges can be either subtracted or added to conventional gages' readings to obtain actual stress within the concrete. This application somehow cancels the necessity for temperature compensation coefficient of any VW sensors (ASCE, 2000).

2.5.5.4 Vibration Measurement Devices

The vibrations initiated either natural (earthquake or tectonic movements) or artificial incidents (e.g. blasting in the vicinity of the dam) may endanger the safety of the dam and have serious consequences. Seismic vibrations are mostly considered as severe for the structural integrity. Today, all large dams have been instrumented with vibration measuring devices. Data obtained from instrumentation systems during earthquakes might differ in a great deal compared to design assumptions. Technological innovations allow engineers to accurately investigate the behavior of the dams during strong ground motions. The validity of the design predictions for the seismic behavior of dams can only be verified via comparison with field measurements.

Earthquake and earthquake-induced forces have been seriously taken into account at design phase, especially for the dams located in active seismic zones. However, existing theories are still considered insufficient and somewhat questionable since they are based on experimental studies. It is desired to observe behavior of dams during earthquakes from the initiation to the end. The interpretation of recorded data is quite valuable to upgrade analysis, devise adequate layout of instrument and to improve the dams' resistance against earthquakes. Data obtained can also guide the investigation for possible damages and establishment of remedial measures.

Many instruments have been developed since 1930's, but currently strong motion accelerometers have been widely used throughout the world (Bartholomew and Haverland, 1987). Accelerometers have been placed at various points to be able to answer different questions. Instruments have been commonly installed at crest of the dam, abutments, and foundations near the dam site. The earthquake behavior of dams can only be checked by way of comparison with regard to the free-field measurement. The free-field instruments are commonly located in vicinity of the dam site.

The free-field instrument must be installed far enough from the dam and from the appurtenant structures to prevent it from vibrations and their presence. On the other hand, it must be close enough to obtain measurements of motions, which characterize of those at the site. A typical distance, which equals to twice the dam height, is adequate for concrete dams. The distance can be reduced to once the dam height in case the modulus of elasticity of the foundation is equal to or higher than the modulus of the dam concrete (Darbre, 1995).

Abutment instruments are placed if any non-uniform motion is expected along the dam-abutment or dam-foundation interface due to topographical, inertial and energy dissipation property differences. Foundation instruments permit engineers to apprehend wave propagation at the located level. Instruments located at the crest of the dams allow assessing the response of the dam during strong motions. The crest instruments must be located at mid-span where the maximum modal deflections are anticipated.

The minimum instrumentation systems are composed of three accelerometers located in a proper scheme. A free-field instrument, abutment instruments either located at the base or at an abutment and a crest instrument are required to efficiently evaluate the seismic behavior of a dam. Correlation between measurements must be implemented to gain a better understanding of seismic behavior of the dam. The array configurations must be organized in such a way that the correlation between accelerometer measurements can be implemented easily. Moreover, accelerometers in a seismic array are required to trigger simultaneously whenever an event occurs. During an event, two or more instruments receiving an earthquake trigger, the common triggering feature activates all the accelerometers in an array to start recording. This can be achieved with a single external trigger source, which allows connection to several digitizers by wiring up together. This feature enables complete data coverage and almost eliminates all the problems related to artificial incidents.

2.5.5.5 Temperature Measurement Devices

Measurement of temperature is more crucial for concrete dams. The practicability has been seriously limited for embankment dams since temperature instruments have been only served for monitoring migration of ground water or seepage paths. Monitoring temperature is more common for concrete dams because they exhibit volumetric changes within due to chemical reactions and heat differentiation between dam sections.

Concrete is mainly composed of cement, aggregates and water. The mixture solidifies and hardens as a consequence of chemical exothermic reaction called hydration which occurs as water and cement meet. Deformations and cracking may take place due to excessive internal temperature rise or fall whether temperate fluctuations are not observed or proper curing are not performed. Cracks may form as a result of expansion if concrete cooling has been poorly maintained. Conversely, shrinkage cracks may result if the curing has been rapidly performed. Installation of sensors capable of measuring temperature enables engineers to modify the rate of curing to avoid thermal cracking. It is also a good application for long-term structural behavior monitoring since hydration reaction may last over years.

During operation, heat distribution varies within dams because of transfers from reservoir water, ambient air and radiative sunlight. The temperature difference between submerged upstream face and downstream face is the main reason of differential stresses. Commonly, embedded instruments suitable for long-term measurements have been installed within concrete dams to observe the propagation of thermal cracking. Moreover, correlation between other instruments' readings can be performed to better evaluate structural behavior.

Today, most of the manufactured instruments serving for different purposes are integrated with temperature sensors. This feature allows compensation for temperature induced errors. Temperature sensor can mainly be of three types; Thermocouples, Resistance Temperature Detectors (RTD) and VW temperature sensors are commercially available just to monitor temperature. Thermocouples are appropriate for short-term monitoring. These instruments have been utilized for modifying curing rate during construction. RTDs are common since they are capable of accurately monitoring temperature for both short and long-term measurements. VW Temperature Sensors are less common because they are relatively expensive. Typical RTDs are shown in Figure 2.13.

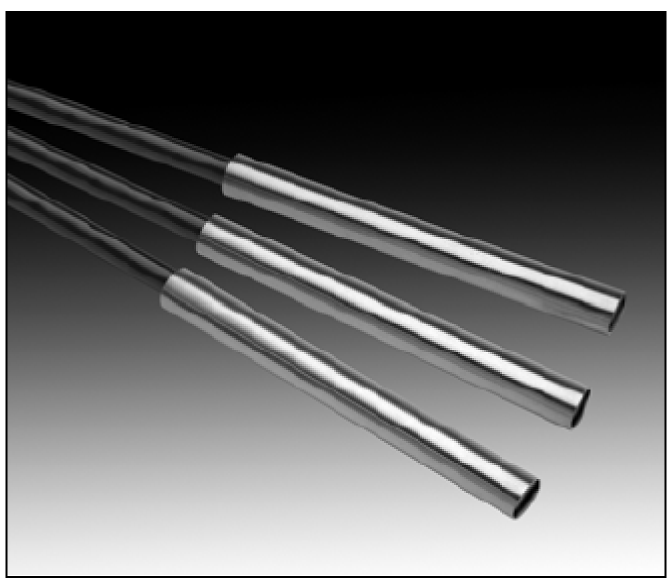

Figure 2.13.Typical resistance temperature detectors (SI, 2008)

2.5.6 Data Acquisition Systems

Every step of the design is mutually connected to each other but the weakest link is the data acquisition system. Properly selected parameters and correctly installed instruments are useless if the data acquisition system has been poorly planned. The system also cannot improve the dam safety. Therefore, collection of instrument data must be implemented in a logical manner. A typical data acquisition program includes five steps to be followed; collection, reduction, presentation, interpretation and reporting of instrumentation data. Each step must be organized before construction commences in the field. The same steps should be followed irrespective of the type of the data acquisition system (Bartholomew and Haverland, 1987).

2.5.6.1 Data Collection

Data acquisition must be conscientiously implemented to provide proper data for assessing the dam performance. Data acquisition systems are varied from simple readout units to complex automated systems. Instruments report physical changes via transducers, which convert the changes into output signals. These outputs are transmitted by shielded signal cables to specified locations. Generally, signal cables consist of two or more conductors. During construction, each instrument cable is connected to proper portable readout unit in different conductor combinations to collect parameters of interest. An example instrument reading is shown in Figure 2.14.

Technicians who will perform manual data acquisition should have some qualifications, such as mechanical and electrical ability, computational skill and background in fundamentals of civil engineering. Prior to beginning of field data collection, technicians must be trained in instruments, installation, calibration, maintenance, construction procedure, monitoring schedule, recording procedure, etc. They must also be aware of the necessity, basic operating principle and maximum allowable limits of each instrument to assess both instrument and dam performance. A typical record form for manual measurements is given in Appendix-A.

Figure 2.14. Strain gage measurement via portable reader unit during initial filling of Cindere Dam

At the end of construction, depending on the type of data acquisition system, system components must be set up at the field. Manual data acquisition systems are relatively simple and composed of switch terminal boxes, portable readout units and other accessories. Switch terminal boxes are utilized to connect up to twenty-four instruments at a single point. They are commonly placed at an easily accessible location and allow data acquisition via both portable readout units and portable data loggers. Located either inside or outside of the dam, open ends of each signal cable are vulnerable to deterioration and corrosion, which may significantly reduce the accuracy of the measurements. Switch terminal boxes promote the manual data acquisition systems not only by reducing the operating efforts but also by sheltering signal cables against adverse effects.

An automatic data acquisition system includes all the required components for the manual systems and additional units such as multiplexers, data loggers and multi-logger software. The primary function of multiplexers is to increase the number of sensors that can be measured by data loggers. A typical multiplexer is capable of multiplexing up to thirty-two sensors. Data loggers interconnected to multiplexers are managed via multi-logger software. Multi-logger software enables real-time monitoring at desired intervals. Measurements can be automatically converted into engineering units and illustrated in graphical form. Threshold limits can be specified for activating alarm function (SIS, 2008). All the above-mentioned units have electromagnetic interference (EMI) protection.

Visual observations are vital, neglecting the type of data acquisition systems. They should be implemented according to specified monitoring schedule to identify several indications, such as cracks, misalignment, offsets or any other adverse effects regarding dam performance. The greatest disadvantage of automatic data acquisition systems is that visual observations will not be accomplished while it is a routine application for manual data collection (FERC, 2008). Other advantages and limitations of automatic data acquisition systems over manual systems are summarized in Table 2.3.

Table 2.3. Advantages and limitations of automatic data acquisition systems (Dunnicliff and Green, 1988)

Advantages	**Limitations**
Reduced personnel cost for reading and analyzing data	Replacement of a knowledgeable observer by an item of hardware (there is a real possibility that visual observations will not be made, that other factors influencing measured data will not be recorded, and that causal information will therefore not be available)
More frequent readings	Possibility of generating an excess of data, encouraging a "file and forget" attitude, and therefore failing to take timely action in response to data
Retrieval data from remote or inaccessible locations	Possibility of blind acceptance of data, which may or may not be correct (garbage in, garbage out)
Instantaneous transmittal of data over long distances, using telemetry	High initial cost and often high maintenance cost
Increased reading sensitivity and accuracy	Currently, often requires some custom designed components that are un-proven
Increased flexibility in selecting required data	Complexity, requiring an initial "debugging" period
Measurement of rapid fluctuations, pulsations and vibrations	Need for regular field checks and maintenance by specialized personnel
Recording errors are fewer and immediately recognizable	Need for backup manual recording arrangements
Electronic data storage is in a format suitable for direct computer analysis and printout	Need for a reliable and continuous source of power Susceptibility to damage caused by weather conditions and construction activity

2.5.6.2 Data Reduction

Most instruments produce frequency and 4-20 mA signals, which are considered as raw data. The gathered raw data recorded on data sheets are converted into usable engineering units by simple numerical calculations. Technicians may carry out required calculations under the supervision of engineers. The software of automatic data acquisition system is capable of automatically converting raw data to engineering units according to the predefined formula.

2.5.6.3 Data Presentation

Reduced data must be processed and should be summarized in comparative graphical form. Commonly, the comparison is executed between the new measurement and the previous ones. The simplest graph is plotted data versus time manner but location versus movement can be utilized depending on the type of the instruments. Different variables can be plotted in the same graph to correlate the reason and the effect for instance seepage quantity and reservoir level. The plots are automatically generated via the software of automatic data acquisition systems in well-known formats.

2.5.6.4 Data Interpretation

Data interpretation is an intensive human activity to review all gathered data in many aspects. The suitability of measurements is checked in terms of magnitude of change, range of instruments, the number of measurements, site conditions and unusual specific conditions. The primary purpose is to present clearer understanding of real behavior of dams. Cause and effect relationship should be clearly apprehended to illustrate proper plots of instrumentation data.

2.5.6.5 Reporting of Instrumentation Data

Either collected manually or automatically, experienced engineers who are familiar with instruments and specific site conditions must check the interpreted documents to assess the dam performance. If any abnormalities were inspected, engineers would decide on remedial actions to be taken to ensure safety.

CHAPTER 3

GENERAL FEATURES OF CİNDERE DAM

3.1 Introduction

Cindere Dam is the first hardfill dam in Turkey. The construction was started in January 2002 and accomplished at the end of 2007. Cindere Dam is located in Denizli province, in the western Anatolian region in Turkey (See Figure 3.1). The dam is located 2 km away from the Cindere village, between Boztepe and Aktarla hills, on the Büyük Menderes River. The dam has been designed as a Face Symmetrical Hardfill Dam (FSHD), which has side slopes 0.7 H: 1.0 V in both upstream and downstream faces. The purpose of the dam is irrigation and power generation. The power plant to be constructed has a capacity of 28.5 MW and the average annual power generation is 88 GWh (Batmaz, 2003). Cindere Dam will also provide irrigation water to Gölemezli, Pamukkale and Buldan plains, which are covering an area of approximately 4,600 ha (EGEV, 2007). Plan view and typical cross-section of the dam are given in Figures 3.2 and 3.3.

Cindere Dam, with a total reservoir capacity of 84.3 hm^3, a height of 78 m from thalweg, 107 m from foundation and a crest length of 281 m, is one of the highest dams on the world of its type. The dam volume is 1,680,000 m^3, which contains 1,500,000-m^3 hardfill and 180,000-m^3 conventional concrete. There are three internal drainage galleries at the upstream of the dam at 245.50, 220.50, and 195.50 m elevations. There exists a grouting and drainage gallery at the elevation of 170.50 m. The imperviousness is provided by a watertight upstream facing, which is made of precast concrete panels with PVC covered inside and 1.0 m thick conventional concrete.

The project is owned by Turkish State Hydraulic Works (DSİ). The construction consultant is a Brazilian firm, Andriolo. The structural design was carried out by Temelsu International Engineering Services Incorporated. The construction of the dam has been performed by Özaltın Construction Co. Incorporated (Batmaz, 2003).

Figure 3.1. A satellite view of Cindere Dam (Google Earth, 2008)

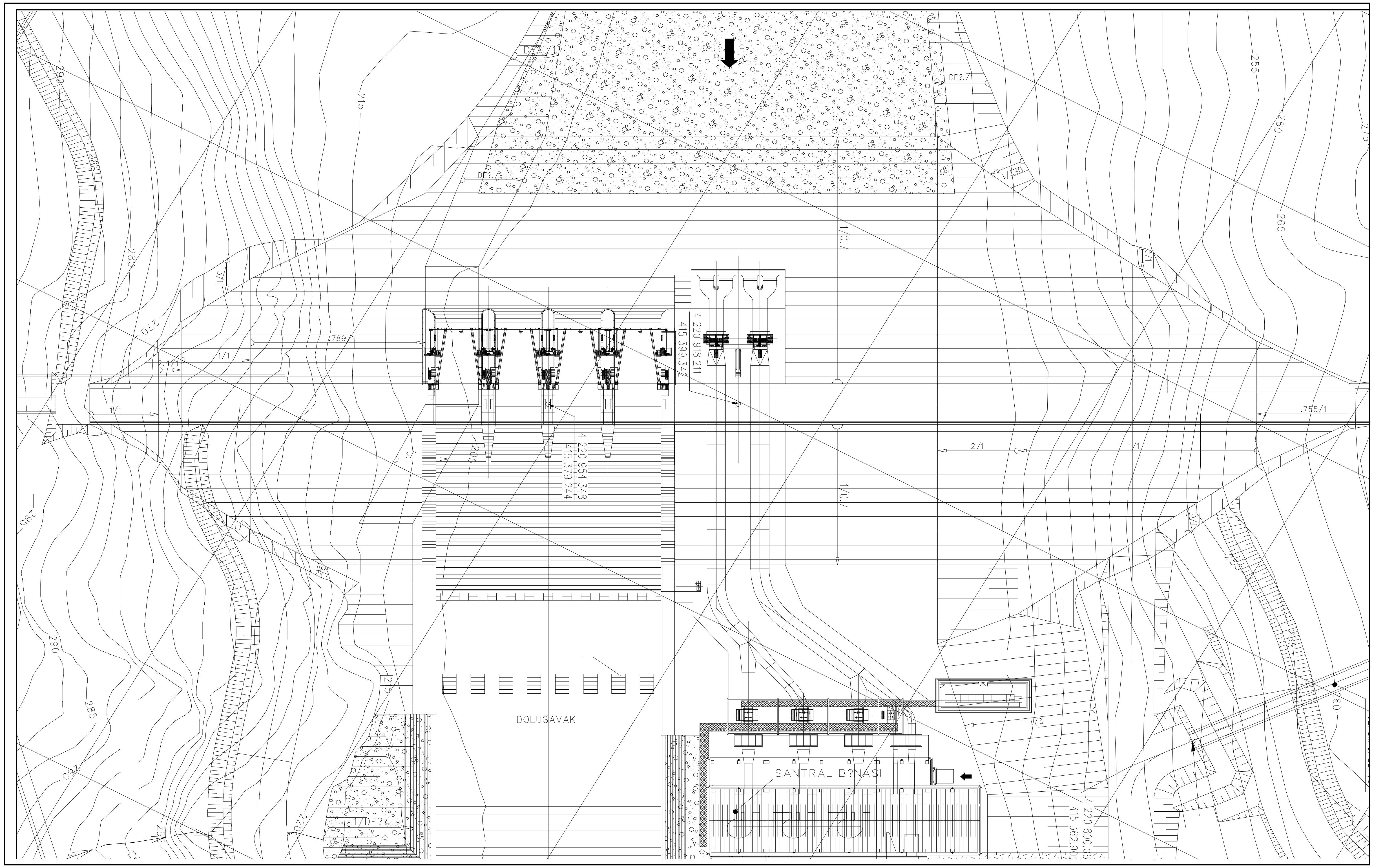

Figure 3.2. Plan View of the Cindere Dam (Gürdil, 2003)

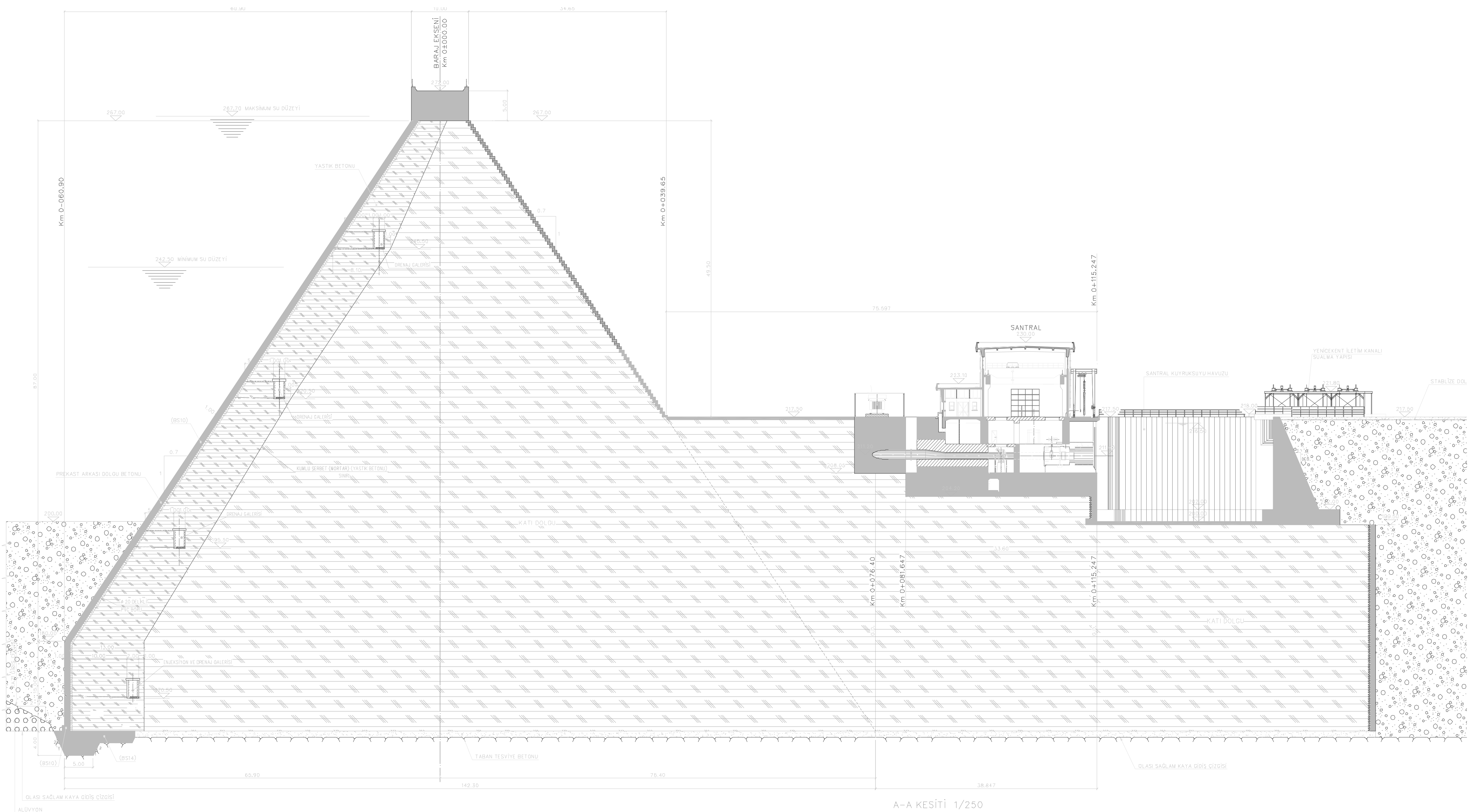

Figure 3.3. Typical Cross Section of the Project (Gürdil, 2003)

3.1.1 Site Conditions

In this section, site conditions will be presented briefly.

3.1.2 Climate

The climate at the dam site is Mediterranean-type, which has mild, moist winters and hot dry summers. The average temperature is 26.6 ^{o}C during summer. Maximum temperature is estimated as 41.2 ^{0}C in August. No detrimental weather and climate problems are expected at the dam site. Some measures should be taken against cracking in very hot summer days and during rare frosty winters.

3.1.3 Geology and Foundation

The dam site is composed of metamorphic schists including mica, chalk, sericite and graphite schists. The rock quality has been determined as “fair rock” in saturated conditions. Typical characteristics of fair rock are given below (Gürdil and Batmaz, 2003).

a. Angle of Repose : 25-35^{0}
b. Cohesion : 200-300 kPa
c. Modulus of Elasticity : 8-9 GPa

Mainly well-developed schist planes are observed at the dam site. There is no significant shear or fault zone detected which may require special treatment. A deep grout curtain will be constructed to ensure the impermeability of the foundation. The inclined galleries and primarily the gallery at 170.5 m elevation will serve as grouting galleries. One line of holes will be drilled with an angle of 15^{0} varied from 80 m at the riverbed to 30 m close to the crest. These holes are located three meter away from the upstream face of the dam. In addition, two lines of blanket grouting (2.5 m long) will be constructed away from the upstream face (Batmaz, 2003).

3.1.4 Seismicity

Cindere Dam is located at the eastern part of the Western Anatolian Graben, which is an active tectonic zone. Severe earthquakes occurred in 1955, 1963, 1966, and 1995 at Söke, Denizli, Aydın and Dinar, respectively (Gürdil, 2003). In addition, the location of the dam is in Seismic Zone-1, which is the worst condition in the Seismic Hazard Map of Turkey. By considering, the facts mentioned above, the dam was designed according to the ICOLD criteria. Two types of earthquakes are considered; Operating Basis Earthquake, OBE, and Maximum Credible Earthquake, MCE, which have been widely accepted scenarios in dam engineering. No structural damage is allowed and dam must remain operable after the OBE. In MCE, structural damage is acceptable but uncontrolled release of water is not permitted (Gürdil and Batmaz, 2003). Dynamic analysis has been carried out with respect to OBE and MCE. The peak ground acceleration of the dam site was estimated as 0.4 g for OBE hazard level with a return period of 2000 years and maximum probability of exceedence of 5% in 100 years. For MCE hazard level, the peak acceleration was assessed as 0.60 g with a return period of 10.000 years and maximum probability of exceedence of 1% in 100 years. The "horizontal earthquake coefficients" were selected as 0.2g and 0.3g for OBE and MCE, respectively. These coefficients are the highest ones used in dam design in Turkey. Moreover, these are the peak horizontal earthquake coefficients for concrete dam design in Japan (Gürdil, 2003).

3.2 Design of the Dam

In this section, design of the dam will be introduced briefly.

3.2.1 Selection of the Dam Type

Firstly, the dam was planned to be built as a clay core earthfill dam. There were, however, several reasons to abandon earthfill type such as high excavation cost, lack of clay material, stability problems, etc., (Batmaz, 2003). Dam type was chosen as FSHD by considering the benefits as explained in Section 1.4.

3.2.2 Dam Body Design

Basic steps of the design are explained in the following sections.

3.2.2.1 Design of Embankment

Several downstream and upstream slopes have been analyzed in order to determine the optimum shape and volume of the dam. The upstream slope varied from 0.4 H – 1.0 V to 0.7H – 1.0 V while downstream slope varied from 0.7H – 1.0 V to 1.0 H- 1.0 V. The minimum dam volume was achieved by 0.4 H – 1.0 V upstream and 0.8 V – 1.0 H downstream slopes with a 123.0 m base width. Although the shape gives the minimum quantity of hardfill, the safety factor against sliding was not sufficient at both OBE and MCE loading cases. The symmetrical shape with 0.7H - 1.0 V slopes was preferred but the amount of hardfill was increased by 11% with respect to previous shape. The base width of the symmetrical cross section was 20 m wider, 143 m. As a result of which, base pressure has been reduced around 21% compared to the previous one (Gürdil, 2003).

Thermal analyses have been performed in order to determine the spacing between contraction joints to avoid cracking. Gürdil (2003) stated that a “Level 1 Thermal Analysis (USACE, 1997)” was followed because of its simplicity, instead of complex finite element analysis. As a consequence of the thermal analysis, minimum crack spacing was obtained as 35.2 m. Contraction joints were conservatively provided by 20 m intervals to avoid any additional cracking (Gürdil, 2003). Two rows of PVC waterstops were placed between successive joints to minimize the amount of water leaking through (See Figure 3.4).

Figure 3.4. Upstream facing system details (Gürdil, 2003)

The quality of the materials and proportions of the mixture are too important to achieve durability, workability and strength. A dry mixture (VeBe Time: 25-30 s) composed of well-graded aggregate, (D_{max}= 63mm), 50 kg/m^3 cement and 20 kg/m^3 fly ash was spread in 300 mm layers (Batmaz, 2003). The compaction has been carried out by 15-ton vibrating rollers.

VeBe Time is a parameter of the concrete mixture, which indicates the workability of the mixture. Reference values are given in Table 3.1. The desired material properties for the Cindere Dam project are presented in Table 3.2.

Table 3.1. VeBe times (Erdoğan, 2003)

Description	VeBe Time (seconds)
Extra Dry	32-18
Too Dry	18-10
Dry	10-5
Plastic Dry	5-3
Plastic	3-0
Fluid	---

Table 3.2. Desired material properties at Cindere Dam (Batmaz, 2003)

Material	Compressive Strength (MPa)	Tensile Strength (MPa)	Cohesion (MPa)	Internal Friction (Degree)	Total Cementitious Content
Hardfill	6.0	0.6	0.8	45	70(50+20)
Leveling Concrete	14.0	1.4	2.0	37	250
Facing Concrete	10.0	1.0	1.5	37	160
Bedding Mortar	7.0	0.7	1.0	40-45	300
Precast Elements	30.0	3.0	4.5	37	350

Hardfill is a zero slump concrete, which must be dry enough to support the weight of large vibratory compaction rollers and wet enough for sufficient distribution of the paste. The placement of hardfill has been carried out in 30 cm lifts. Because of the dry nature of the material, bonding between subsequent lifts is weak which decreases shear strength. In order to bond successive layers, either bedding mortar or conventional concrete have been commonly applied as thin layers. In the construction of the Cindere Dam, 25 mm thick bedding mortar layers have been utilized at vulnerable zones. The adjustment has been implemented at both the upstream and downstream face. This application not only increases structural integrity but also limits the seepage through the body (Batmaz, 2003).

3.2.2.2 Upstream and Downstream Facing

Hardfill is a pervious material and an upstream facing is necessary in order to control the seepage through the dam body. The watertightness was provided by precast PVC covered concrete panels and 1.0 m thick conventional concrete (Batmaz, 2003). Precast concrete panels act as forms during the placement of the hardfill and will constitute a suitable layer for the construction of conventional upstream concrete facing (Batmaz et al, 2003). Durability and the appearance are the important factors for the selection of the downstream facing. High quality cast in place concrete was preferred for the facing of the downstream.

3.2.2.3 Diversion Facility and Spillway

In order to provide a dry zone for the construction, a central clay core cofferdam was built at the upstream. Diversion tunnel, 768 m long, 5.0 m in diameter, was located at the left bank (Batmaz, 2003). The capacity of the diversion tunnel was 254 m^3/s, which is the discharge for the return period of 25 years (Batmaz, 2003). Radial-gated, chute spillway was located central right portion of the dam. The discharge capacity of the spillway is 3600 m^3/s. Energy dissipation basin has a length of 66 m (Batmaz, 2003).

3.2.3 Stability Analysis

Stress analyses have been carried out by running the finite element analysis SAP2000 software. During design, both dam and foundation have been added to the model in order to obtain behavior of both under several loading conditions. Intensive attention has been paid for meshing the regions where stress concentrations were expected (Gürdil and Batmaz, 2003). Loading conditions, minor and major principal stresses, maximum shear stress and peak lateral deflections were shown in the Table 3.3. These parameters are accepted within tolerable limits. So, one can state that the dam is safe for the loading conditions considered.

Table 3.3. Results of finite element analysis (Gürdil and Batmaz, 2003)

Load case	**Major Principal Stress (kPa)**	**Minor Principal Stress (kPa)**	**Horizontal Shear Stress (kPa)**	**Lateral Deformation (mm)**
End of Construction	623	-6030	-1540	11.5
End of Construction + EQ (OBE)	**913 (around gallery)**	-6590	-1810	-27.22
Service Condition	220	-5820	1320	42.45
Service Condition + EQ (OBE)	**747 (at the upstream heel)**	**-7620**	**2090**	**75.64**
Flood Case	211	-5610	1400	46.94

3.3 Instrumentation of Cindere Dam

Instrumentation of Cindere Dam will be basically introduced in this section. The layout of many instruments is shown in Figure 3.5.

3.3.1 Measured Parameters

Cindere Dam is a face symmetrical hardfill dam with an impervious upstream facing. By considering the location, type, geometry and other significant parameters, several instruments have been installed for monitoring the dam performance. Parameters to be measured are given below.

a. Stress between dam body and dam components
b. Stress within upstream facing concrete
c. Stress within dam body
d. Relative movements between the upstream facing joints
e. Pore water pressures within foundation and abutments
f. Earthquake magnitude within dam body
g. Earthquake magnitude within foundation
h. Reservoir level
i. Seepage quantity
j. Horizontal and vertical movements

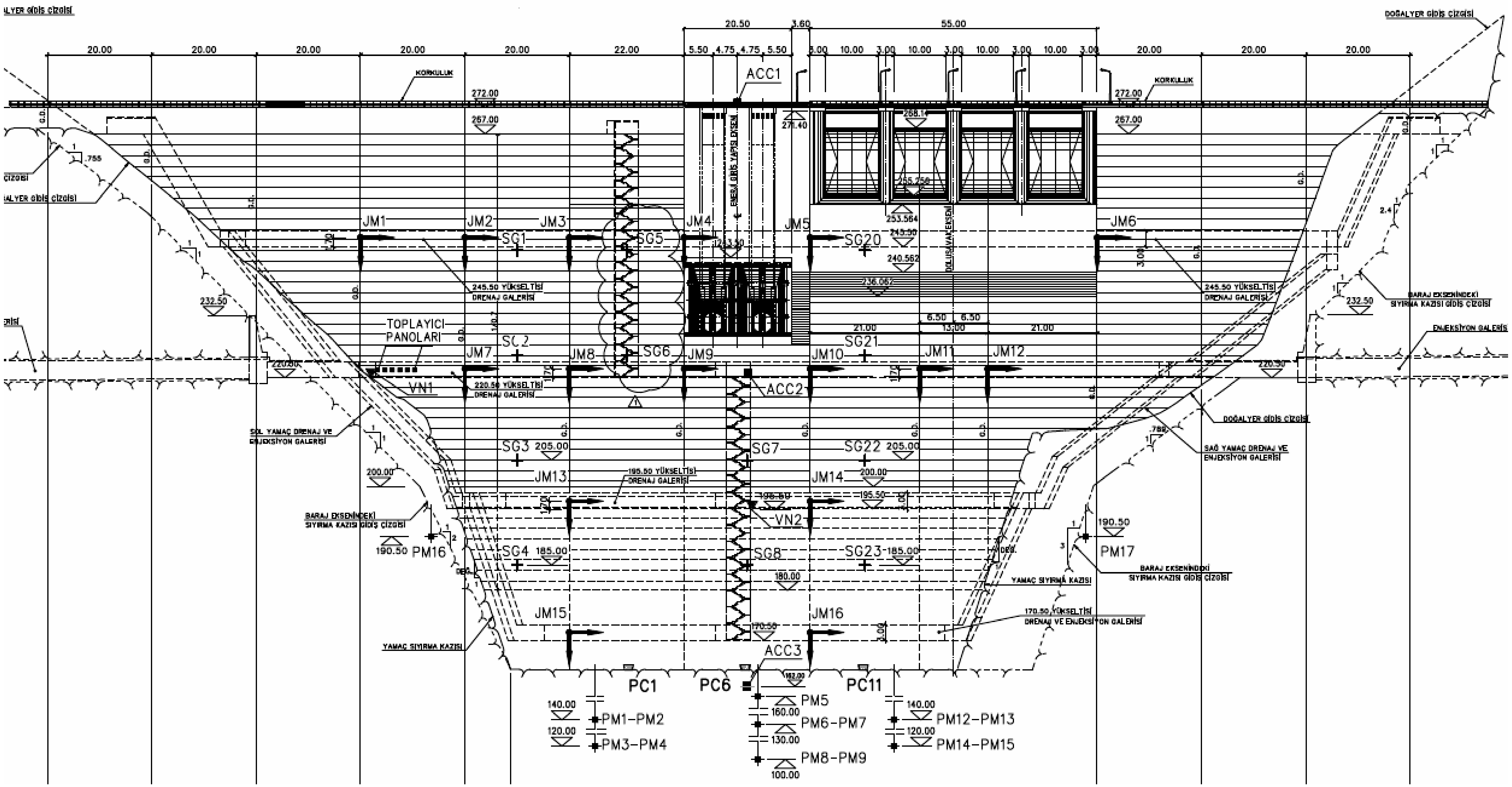

Figure 3.5. Layout of Instruments at Cindere Dam (Upstream View), (Temelsu, 2003)

3.3.2 Current Instruments

In this section, current instruments of the dam will be presented briefly.

3.3.2.1 Piezometers

PMs (Piezometers) were placed within the foundation rock and abutments to observe pore water pressures and the effectiveness of the grout curtain. Vibrating wire (VW) PMs have been installed at seventeen points. Thermistors are also integrated within VW PMs to measure temperature. During construction, all PMs were installed. Measurements have been carried out for PM10 and PM11 via portable readout units. No unexpected behavior has been observed. The layout of VW PMs is shown in Table 3.4.

Table 3.4. Piezometer layout at Cindere Dam

Piezometer Label	Location	Elevations (m)
PM1	Upstream of the grout curtain	140
PM2	Downstream of the grout curtain	140
PM3	Upstream of the grout curtain	120
PM4	Downstream of the grout curtain	120
PM5	Upstream of the grout curtain	160
PM6	Upstream of the grout curtain	130
PM7	Downstream of the grout curtain	130
PM8	Upstream of the grout curtain	100
PM9	Downstream of the grout curtain	100
PM10	At the dam axis	162
PM11	Close to downstream toe	162
PM12	Upstream of the grout curtain	140
PM13	Downstream of the grout curtain	140
PM14	Upstream of the grout curtain	120
PM15	Downstream of the grout curtain	120
PM16	Within the left abutment	190.50
PM17	Within the right abutment	190.50

3.3.2.2 Earth Pressure Cells

Earth pressure cells have been installed between the dam body and foundation contact. Fifteen circular VW earth pressure cells are placed at 165 m elevation. Thermistors are also integrated within VW earth pressure cells to measure temperature. The layout of instruments is simple. Three rows and five columns of earth pressure cells are located in between 30-40 m intervals.

3.3.2.3 Strain Gauges

Strain gauges have been installed within the upstream facing concrete and dam body to determine the stress and deformations. There exist 58 strain gauges, 11 2-D rosette VW strain gauges within the dam body and 12 3-D rosette VW strain gauges within the upstream facing. Thermistors are also integrated within VW strain gauges to measure temperature.

3.3.2.4 Joint Meters

2-D Joint meters have been installed within the galleries to monitor relative movements between joints. Totally 16 joint meters will be placed, two at 170 m elevation gallery, two at 200 m elevations gallery, six at 220 m elevations gallery and six at 243 m elevation gallery.

3.3.2.5 Strong Motion Accelerometers

Observing the behavior of the dam during any vibrations is quite important for assessing the validity of the design and having future information. By considering the location of the dam, three strong motion accelerographs have been installed at foundation level, dam body and crest at 162, 220, and 272 m elevations, respectively.

3.3.2.6 Reservoir Level Gauges

Two reservoir level gauges are placed on the upstream face of the dam. The first, one is located at the 243.50 m elevation, which is one meter higher from the minimum operating level. The second one is installed at 255.00 m elevation, which is 17 m below the crest level.

3.3.2.7 V-Notch Weirs

Two each V-Notch weirs equipped with automatic measurement system are placed at 245.50 and 220.50 m elevation drainage galleries to monitor the seepage quantity.

3.3.2.8 Survey Monuments

Survey monuments are installed at 14 points at the downstream face of the dam. These instruments indicate the horizontal, vertical and axial deformations.

3.3.3 Data Acquisition System

The block diagram of the automatic data acquisition system to be used in the Cindere project is given in Figure 3.6. The system mainly includes sensors, multiplexers, data acquisition unit, computers and monitoring systems. As it can be seen from the block diagram, data produced by each sensor are sent to data acquisition unit via multiplexers. Preliminary data processing (filtering, digitizing) in the data acquisition unit feed the computer and monitoring system. These data are transmitted to remote locations (regional and headquarters computer).

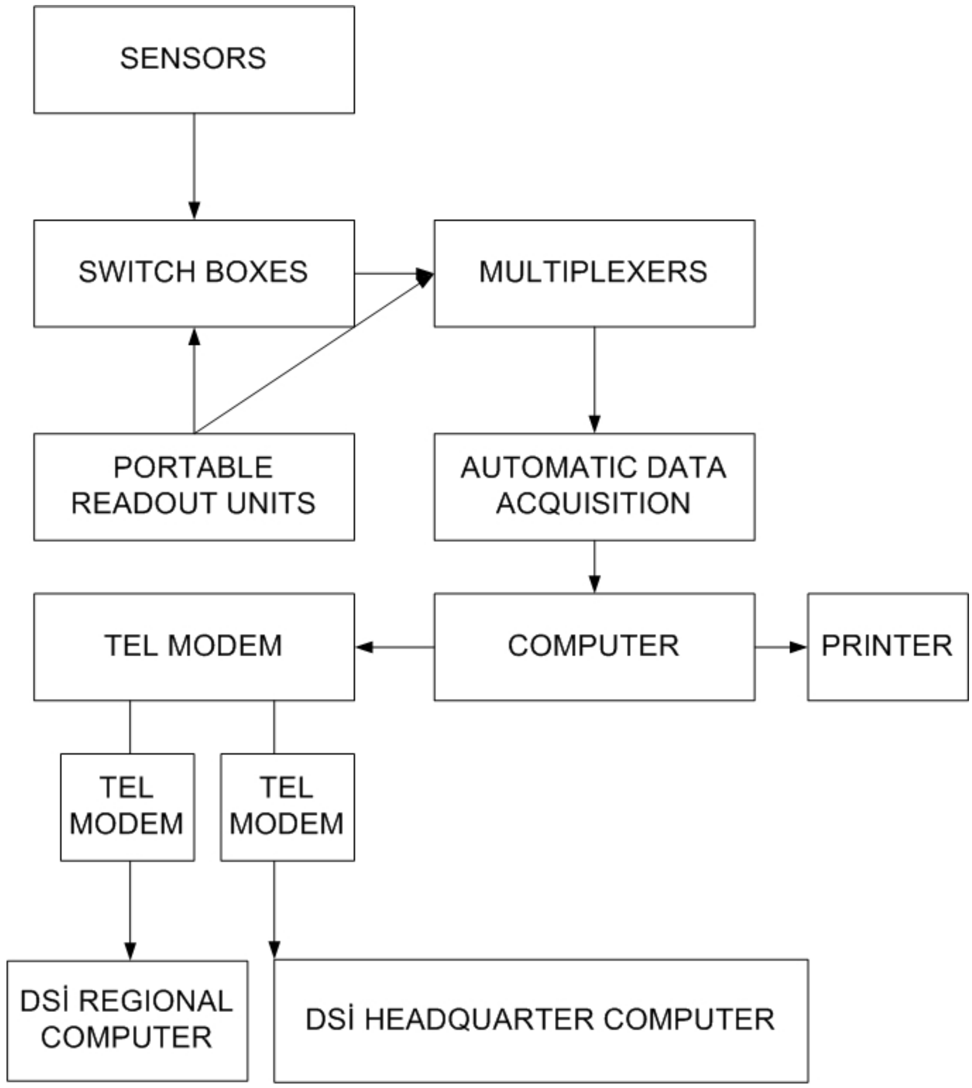

Figure 3.6. Block diagram of data acquisition system at Cindere Dam

(Tosya, 2005)

CHAPTER 4

EVALUATION OF THE CURRENT INSTRUMENTATION SYSTEM

4.1 General

Cindere Dam is a notable project, especially with its height as an example of hardfill dam, not only in Turkey but also in the world. In this chapter, a detailed evaluation of the current instrumentation system of Cindere Dam is presented. Additional analysis and several examinations are performed to emphasize the necessity of an instrumentation system.

4.2 Cindere Dam Risk Assessment and Hazard Classification

The first step of the instrumentation design is the determination of the parameters to be measured. This can only be correctly addressed by fully understanding of dam as whole. Fundamental features; type, location, materials, geometry and many other significant properties of the dam have been introduced in Chapter 3. In this section, specific analysis and comparisons are presented to underline the need of an instrumentation system.

Dam risk assessment studies are sound tools for prioritizing structural and non-structural risk reduction measures. The outcomes of such studies are also useful for further investigations and analyses, such as monitoring and surveillance, inspections, and emergency planning.

There exist particular risk analyses from simple to complicated, respectively; Failure Modes Identification (FMI), Index Prioritization (IP), Portfolio Risk Assessment (PRA) and Quantitative Risk Assessment (QRA), (USSD, 2003).

A risk assessment study according to DSİ criteria have been followed by 21st Regional Directorate of the State Hydraulic Works (Aydın, 2007). The aim of this study is to provide a comprehensive safety evaluation of the dam. The primary pronounced parameters have been assessed and dams have been separated into different groups. Especially; type, height and seismicity of the dam site has led the designers to carry out specific efforts for minimizing the possible constructional and operational problems. The risk assessment tables are given in Tables 4.1 and 4.2.

Table 4.1. Potential risk analysis of Cindere Dam (Aydın, 2007)

	Extreme	High	Medium	Low
Reservoir Capacity (m^3)	>1.2 10^9 (10)	1.2 10^9 -1.5 10^6 **(6)**	1.49 10^6-25 10^4 (4)	<25 10^4 (0)
Dam Height (m)	>60 **(5)**	60-30 (3)	29-15 (2)	<15 (0)
Downstream Population	>1000 (**40**)	1000-100 (20)	99-1 (10)	0 (0)
Potential d/s Hazard (*)	(**25**)	(15)	(10)	(0)
Seismic Risk Coefficient (**)	>0.25g (10)	>0.25g (**6**)	0.10g-0.25g (4)	<0.10g (0)
Hydrological Data Currency (***)	T>30 (10)	30<T<15 (**6**)	15<T<5 (4)	T<5 (0)
Spillway Type	Number of Controlled Gates >6 (5)	Number of Controlled Gates 4-6 (**3**)	Number of Controlled Gates <4 (2)	Uncontrolled (0)
Total Points	**70**	**21**	**0**	**0**
Cindere Dam Total Points				**91**

* In case of a dam failure, possible industrial, agricultural, natural resources, infrastructural losses.

** There is an active fault zone in 10 km distance => extreme, otherwise high.

*** The term (T) is the period (years) elapsed since hydrological studies carried out.

Note: Dam heights are considered from the foundation level to the crest.

Table 4.2. Classification of the risk groups for potential risk analysis (Aydın, 2007)

Total Points	Risk Group
(0 - 16)	Low
(17 – 36)	Medium
(37 – 59)	High
(60 – 105)	Extreme

The total points of each parameter indicate the potential risk group. Extreme risk group ranges from 60 to 105 total points. With 91 total points, Cindere Dam was classified in the extreme risk group. Thus, the dam safety is a major factor, which must be taken into account.

In addition, the dam hazard classification systems have been widely utilized to establish priority of investments regarding emergency action, dam safety and instrumentation programs. Dam hazard potential is defined as the possible adverse incremental outcomes resulted from either the release of water or stored contents as a result of failure of the dam or miss-operation of the dam or appurtenances (FEMA, 2004). There are numerous systems to classify dams but each study deals with potential impacts in case of a dam failure or unscheduled release of water. FEMA (Federal Emergency Management Agency) has declared a system, which is simple, concise and adaptable to any agency's current system. The system has been categorized in three levels in the order of increasing adverse effects on life and property. The summary is shown in Table 4.3.

Table 4.3. Dam hazard classification according to FEMA (FEMA, 2004)

Hazard Potential Classification	Loss of Human Life	Economic, Environmental, Lifeline Losses
Low	None Expected	Low and generally limited to owner
Significant	None Expected	Yes
High	Probable. One or more expected	Yes (but not necessary for this classification)

FEMA hazard classification is primarily based on the loss of life and secondarily economic, environmental, lifeline losses. Cindere Dam was constructed in the vicinity of centre of population. Either partial failure or unscheduled release of stored water threats approximately 15,000 people and various agricultural, industrial lands located downstream. Cindere Dam could be classified in high hazard potential level according to FEMA criteria. Both assessments indicate that a proper instrumentation system must have a priority in both design stage and cost issues for Cindere Dam. Emergency action plans and an early warning system might also be considered.

4.3 Parameters Measured at Current System

Instrumentation systems are varied from simple to complicated ones. The complexity of the system is entirely dependent on the number of concerned parameters. These parameters are determined based on risk assessment, hazard classification and numerical analysis.

For ordinary or less significant dams, especially for the low-budget dams, minimum instrumentation systems are commonly preferred. The primary purpose of instruments provided in such systems is ensuring safety. Verifying the design assumptions is considered as a secondary purpose. Although the instruments serving for secondary function are additional cost to the project, the collected data are valuable for better future designs. In Turkey, almost all the dams are funded by DSİ and it interests both primary and secondary functions of the instrumentation systems.

A logical process has been followed for the design of the Cindere Dam. Risk assessment and hazard classification studies have been seriously performed. Afterwards, various analytical surveys and structural behavior examinations have been carried out as indicated below.

a. Classical gravity analysis for the slope optimization
b. Finite Element analysis for the strength requirements
c. Thermal analysis to obtain the required spacing between joints to avoid cracking

Based on the results of the processes described above, since safety and verification of the design assumptions are very important, the following are identified as important parameters to be monitored; total displacements of the structure, and relative displacements within the structure itself, temperature, stress and strain (cracking) status, water pressure and seepage and earthquake related ground motions.

4.4 Instruments of the Current System

In the current system, Cindere Dam was equipped with various kinds of sensors to monitor the key parameters. Type, location, combination and many other aspects of instruments are too important to have a proper functioning system. In this section, a detailed evaluation is carried out to illustrate all the possible alternatives.

4.4.1 VW Piezometers

At Cindere Dam, totally seventeen VW piezometers were provided to perform hydrostatic pressure measurements. The characteristics of the foundation material are important for the selection of the type of the sensors for hydrostatic pressure measurements. Dam site is formed of well-developed stratified metamorphic schist planes. The permeability of the foundation rock is determined as moderate with respect to the measurements carried out after grouting. VW piezometers are reliable instruments for any kind of soil or rock medium because of their short hydrodynamic time lag, in other words rapid response time.

At Cindere Dam, thirteen piezometers have been located at both sides of the grout curtain at different elevations at three sections (See Figure 4.1). The aim of this application is to determine the effectiveness of the grout curtain. Installation of piezometers at different elevations enables monitoring pore water pressure at each stratum. Piezometers installed at the upstream site of the grout curtain may seem unnecessary because hydrostatic pressure at these points may be considered as equal to the reservoir level for the most cases. In fact, at these points hydrostatic pressure is less than reservoir level since the blanket grouting extends the seepage path. In order to obtain reliable data, it is significant to provide piezometers at the upstream side of the grout curtain. Uplift pressure distribution is observed via 13 VW piezometers located upstream, one at the mid-section and one close to the downstream toe. In addition, pore water pressures within each abutment are observed via two piezometers.

Figure 4.1. Installation of PM6 labeled VW piezometer at Cindere Dam

Since foundation rock has moderate permeability, open type piezometers can be considered as an alternative for Cindere Dam. Observation wells are not a proper choice, because they produce misleading measurements where the foundation is formed of variable strata. Casagrande standpipe piezometers allow hydrostatic pressure measurements at desired strata. Installation of standpipe piezometers can only be implemented adjacent to the upstream face since the other sections are not accessible via galleries. At Cindere Dam, a conventional standpipe piezometer monitored via a water level indicator cannot be practiced due to permeability of foundation rock. The boreholes located at the upstream work as drains in such a permeable medium due to high water pressure. Several manometers at the end of the boreholes can be provided inside the grouting gallery to carry out the measurements. The complexity of the system results in questionable measurements and difficulty in operation. In addition, each strata desired to be monitored requires individual boreholes while it is possible to install several VW piezometers to a single borehole.

Open type system piezometers must be disregarded for Cindere Dam as the disadvantages are stated above. Besides, both 170.50 m and 195.50 m galleries will be submerged during operation due to operational strategies. Measurements cannot be performed in galleries due to the fact stated above. Selection of VW piezometers as hydrostatic pressure measurement sensors at Cindere Dam is adequate, reliable and maintainable due to factors explained below.

a. The layout of instruments answers several questions concerning safety. The simplicity of the layout facilitates the installation of cables.
b. Several piezometers can be installed at different strata inside a single borehole. Hence, drilling cost is significantly reduced.
c. Each piezometer has a thermistor unit, which allows compensation for the temperature.
d. The output signal can be transmitted over long cables. Hence, location of measurement is independent of the location of sensors.

4.4.2 Reservoir Level Gauges

Reservoir level gauges (RLG) are commonly vented pressure transducer units. These sensors have been utilized to measure the alteration of reservoir level. At Cindere Dam, two reservoir level gauges were installed at upstream face. The minimum operating level of the dam is 242.50 m elevation. First instrument marked as RLG2 was placed at 243.50 m elevation. One-meter length space has been left between RLG2 and minimum operating level to avoid either any adverse sedimentation effects or any possible damage to sensor caused by a floating object. The second gauge, RLG1, was placed at 255.00 m elevation. The reason of two RLG installations is to provide redundancy. Each gauge has different ranges and both are capable of measuring water level up to maximum operating elevation 267.70 m. Cindere Dam has built for the purpose of hydroelectric power generation so even small water level changes are significant. The arrangement of instruments permits correlation between sensors, which may significantly promotes the accuracy of the system. The higher the accuracy of measurement that is performed, the greater the efficiency that power plant will generate. The output of reservoir level gauges is 4-20 mA signals that can be transmitted over long distances. The system is evaluated as sufficient and maintainable for Cindere Dam.

4.4.3 VW Strain Gauges

Stress measurements have been performed via embedded VW strain gauges at Cindere Dam. Instruments were placed at both dam body and impermeable upstream face at 23 points to monitor internal stress distributions. A single strain gauge is capable of measuring strain in one direction. In order to calculate principal stresses at least two independent strain measurements are required. Various rosette configurations; in other words, several combinations of strain gauges exist to obtain principal stresses at the desired components of dams. The reader is suggested to refer to Section 2.5.3.2.

At Cindere Dam, body was equipped with strain gauges at eleven points. Dam body is primarily subjected to reservoir load in the horizontal direction and its own weight in the vertical direction. Stresses, that may take place at transverse horizontal direction due to volumetric or other changes, are considered significantly less effective than the mentioned loads. It is assumed that biaxial stress state will occur within the dam body. A simple 2-D rosette configuration was assembled at horizontal and vertical directions to determine principal stress within the dam body.

The imperviousness of the dam is provided by precast concrete panels and 1.0-meter thick conventional concrete constructed at the upstream face. It is a good practice to install proper strain gauges at desired locations within the upstream layer since its integrity is crucial for impermeability. In case the principal directions cannot be clearly determined in advance, rosettes formed of at least three strain gauges, which are capable of accurately obtaining principal strains, and stresses must be utilized. In order to be on the safe side, by considering the relatively thin thickness of the upstream layer, a rectangular tri-axial rosette configuration has been assembled to be installed at 12 points (See Figure 4.2).

Figure 4.2. 3-D rectangular strain gauge rosette installation at the upstream face of Cindere Dam

Determination of the layout and location of instruments can be executed by considering finite elements analysis results and engineering judgment. Cindere Dam was analyzed under several loading conditions by running SAP2000 software. The analysis results achieved are beneficial to determine local stress concentrations, distributions and peak values. The sketch given in Figure 4.3 shows the major stress distribution in kPa for the loading condition of End of Construction + Operating Basis Earthquake. It is one of the most critical conditions for Cindere Dam. The dam body can be equipped with strain gauges to the locations indicated as critical in the finite element analysis. Each case can be individually examined to determine a proper arrangement for the strain gauges.

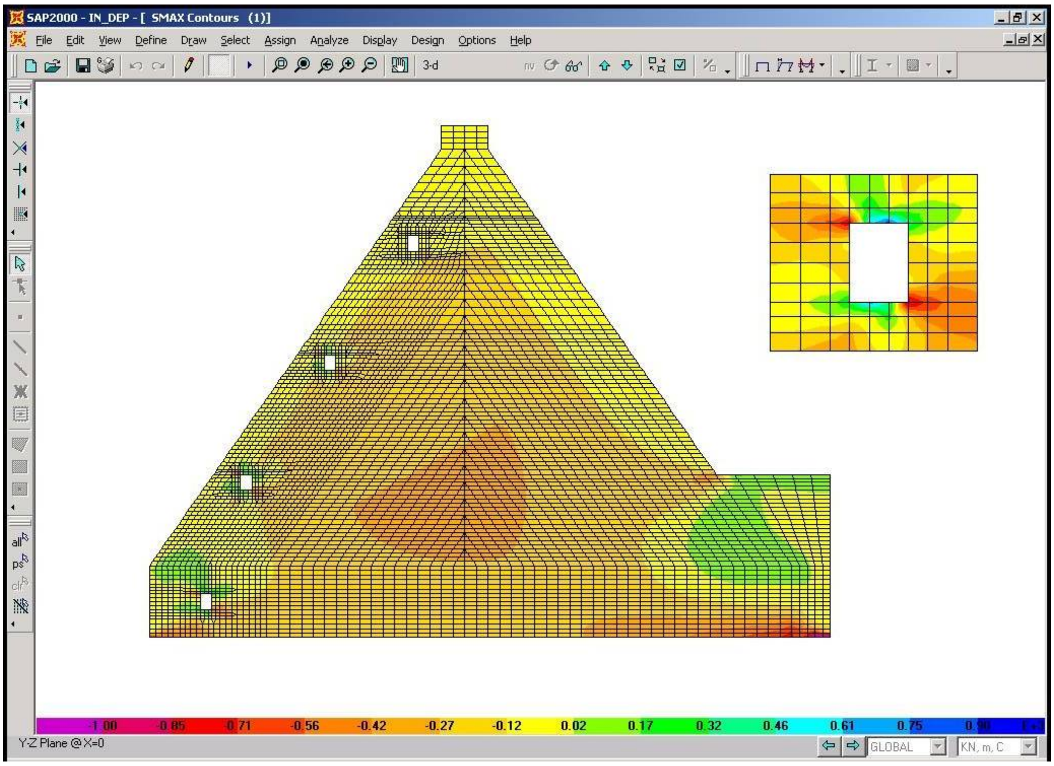

Figure 4.3. Major principal stress distribution, in kPa, for the loading case of End of Construction + Operating Basis Earthquake (Gürdil, 2003)

As a consequence of complex analysis, there might be so many strain gauges to be installed at several points. This application is not feasible for most cases. In addition, the basic assumption for any computer-based calculations is the homogeneity of the materials. Unfortunately, it cannot be attained due to particular problems at construction sites. According to laboratory tests performed on the cores, which were taken from numerous sections of Cindere Dam, it is discovered that the elasticity modulus of cores was varying from 4000 to 11586 MPa (CB-HES, 2007).

Results of the tests reveal that the compaction of concrete could not be perfectly implemented. As a result, local stress concentrations and stress distribution may occur in a different manner. Therefore, verification of data may not be performed efficiently according to the results of the finite element analysis.

At Cindere Dam, a simple strain gauge layout was formed at both dam body and upstream layer. For the dam body, a triangular arrangement, which is almost identical to the cross section of the dam, was designed. As to upstream layer, placement was performed at 12 points as three sections in a rectangular manner. Totally, at 23 points, 58 embedded strain gauges were installed. The aim of providing embedded strain gauges is to observe the dam behavior under various loading conditions during construction and service life. Simple and uniform layouts enable engineers to acquire representative data from the gauges. It also promotes the system because it provides easiness for correlation between measurements, installation of cables and reduces the risk of accidental damage to cables. It should also be noted that, computer based finite element analysis have been always reliable sources which engineers must resort to advance instrument arrangements.

Another important matter is the selection of the strain gauges. The length of the strain gauge should be 3-4 times of the maximum aggregate size in order to acquire accurate measurements (ASCE, 2000). At Cindere Dam well-graded aggregate with a maximum size of 63 mm have been used. The length of each strain gauge is 215 mm, which can be received, in acceptable range. VW transducers have been preferred to be capable of transmitting data over long distances. Integrated thermistor unit allows compensation for temperature changes. Installation was accomplished by qualified technicians. Each strain gauge was carefully tied by using special rosette mounts before concrete pouring. Junction boxes were utilized to transmit data over a single multi-core cable instead of two or three regular cables as shown in Figure 4.2. Providing multi-core cables reduces cabling complexity and cost.

The configuration of rosettes, the layout of instruments, type and transducers of strain gauges are adequate to obtain reliable and maintainable measurements at Cindere Dam.

4.4.4 VW Total Pressure Cells

Pressure cells (PC) were installed at dam body-foundation contact to directly measure stress and orientation. Fifteen PCs were placed in a rectangular manner in three sections. There are five PCs in a section separated approximately 35-40 m from each other. All PCs have been located at 164.00 m elevation except PC1, PC6 and PC11 (161.00 m). The design of the layout provides easiness for correlation between measurements, installation of cables and reduces the risk of accidental damage to cables. Since hydraulic type PCs have been less adversely influenced of both unevenly stress distribution and point loads compared to diaphragm type, hydraulic type pressure cells with circular pad have been preferred in order to obtain accurate measurements.

Each pressure cell was installed to specified pits by well-trained technicians (See Figure 4.4). As the completion of concrete curing, re-pressurization was carried out under the supervision of qualified engineers. The aim of this application is to provide whole contact between pressure pad and concrete. Hence, satisfactory performance of PCs can be somehow assured. The measurements, which have been carried out at different elevations during construction, indicate that pressure values increase proportionally to rise of the dam height. Each PC exhibits similar construction height-pressure diagrams and no abnormal readings have been witnessed up to now. Measurements of PC13 are shown in Figure 4.5 in a graphical form for illustration.

Figure 4.4. Installation of a VW pressure cell at Cindere Dam

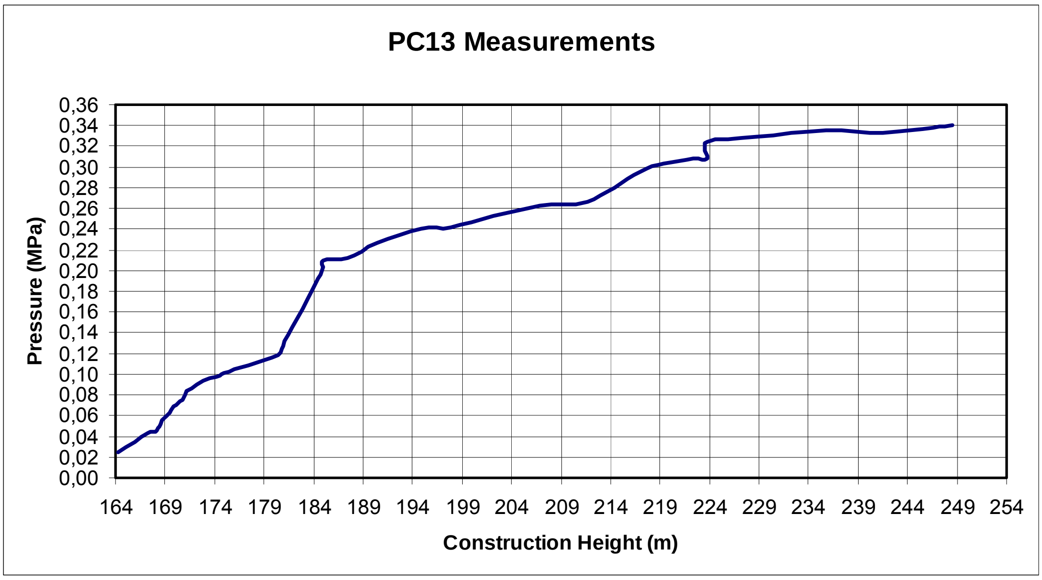

Figure 4.5. Pressure cell 13 measurements, (Adapted from Tosya, 2005)

VW transducer unit allows transmitting signals over long distances. Hence, data acquisition can be performed at desired location. All pressure cells have an integrated thermistor for modifying temperature coefficient. The layout, type and transducers of PCs are convenient to obtain reliable and maintainable measurement at Cindere Dam. Moreover, extensive attention paid during installation to achieve maximum cell response to changing concrete stress.

4.4.5 V-Notch Weirs

At Cindere Dam, there exist four drainage and grouting galleries, which are located at 170.50, 195.50, 220.50 and 245.50 m elevations. Although it is desired to monitor seepage discharge at upper galleries and total cumulative discharge at the bottom gallery, this layout cannot be arranged due to operational strategies. The two lower galleries will be submerged during operation. Two 90o V-notch weirs will be installed to the galleries at 220.50 and 245.50 m elevations.

V-notch weirs to be utilized are made of five-millimeter thick stainless steel. The capacity of each 90o V-notch weir is 10 liter per second. Weirs can accurately measure seepage discharges if they are correctly installed. During design, upstream conditions were carefully examined and modified for representative accurate measurements. Channel bends, elevation drops, or other blockages were taken into account for the selection of weir locations along gutters. In order not to allow excessive velocities, a fair slope (1-3%) will be constructed before installation at the upstream of the weir (GW, 2007). It should be noted that weirs prone to sediment accumulation and algae formation so regular maintenance is required to clean stilling basin of weirs.

Parshall flumes, calibrated containers and velocity meter may be considered as alternatives. Parshall flumes can be preferred over weirs due to several reasons. The details will be introduced on the following sections. Range of calibrated containers is not adequate for such discharges and practicability is questionable due to diversion requirement. In addition, calibrated containers are not suitable for automatic data acquisition. Permanent models of velocity meters can be utilized for dam monitoring. These instruments are not proper choice because of their high maintenance cost.

The flow rate is directly proportional to the water level at stilling basin. Each weir will be monitored automatically via pressure transducer unit. A similar water pressure transducer unit installation is shown in Figure 4.6. These units enable simultaneously data transmitting to ADAS House via cables. Staff gauges will also be provided to enable manual measurement. Visual observations must also be performed especially for water content.

Figure 4.6. Typical installation of V-Notch weir with pressure transducer unit (SIS, 2008)

90^{o} V-notch weirs with pressure transducers are satisfactory for seepage measurements at Cindere Dam. Automatic data acquisition promotes the system since it enables real-time monitoring for flow variations.

4.4.6 Joint Meters

Joint meters (JM) also called crack meters have been used to monitor relative movements across joints and cracks. Contraction joints are a necessity to avoid thermal cracking for concrete dams. At Cindere Dam, the spacing between contraction joints was obtained by numerical analysis as 35.2 m but conservatively designed with 20.0 m intervals to prevent formation of additional cracks. Special combination of JMs can be assembled to monitor bi-axial or tri-axial movements. At Cindere Dam, it is desired to observe relative motions between contraction joints in both transverse horizontal and vertical axes. Electrical 2-D JMs were installed at sixteen points at the upstream face. Installations have been performed inside the specified locations of galleries. Right after installation, initial readings have been implemented to enable comparison between consecutive measurements. A triangular layout was designed similar to the valley shape. The simple arrangement of JMs provides easiness for correlation between measurements, installation of cables and reduces the risk of accidental damage to cables. The output signal (4-20 mA) can be transmitted over long cables. The type and layout of 2-D JMs are convenient for relative movement measurements at Cindere Dam.

4.4.7 Survey Monuments

Survey monuments are certain targets located on several points of the structure to enable monitoring vertical, horizontal, axial movements and surface deformations. Measurements have been carried out via various surveying techniques from a stable location. Monuments will be placed at 12 points on the dam body and at two points on the crest. Reference points will be established on a solid, stable and easily accessible location. Measurement will be implemented via qualified survey crew in accordance with a predetermined schedule. The layout of monuments is shown in Figure 4.7.

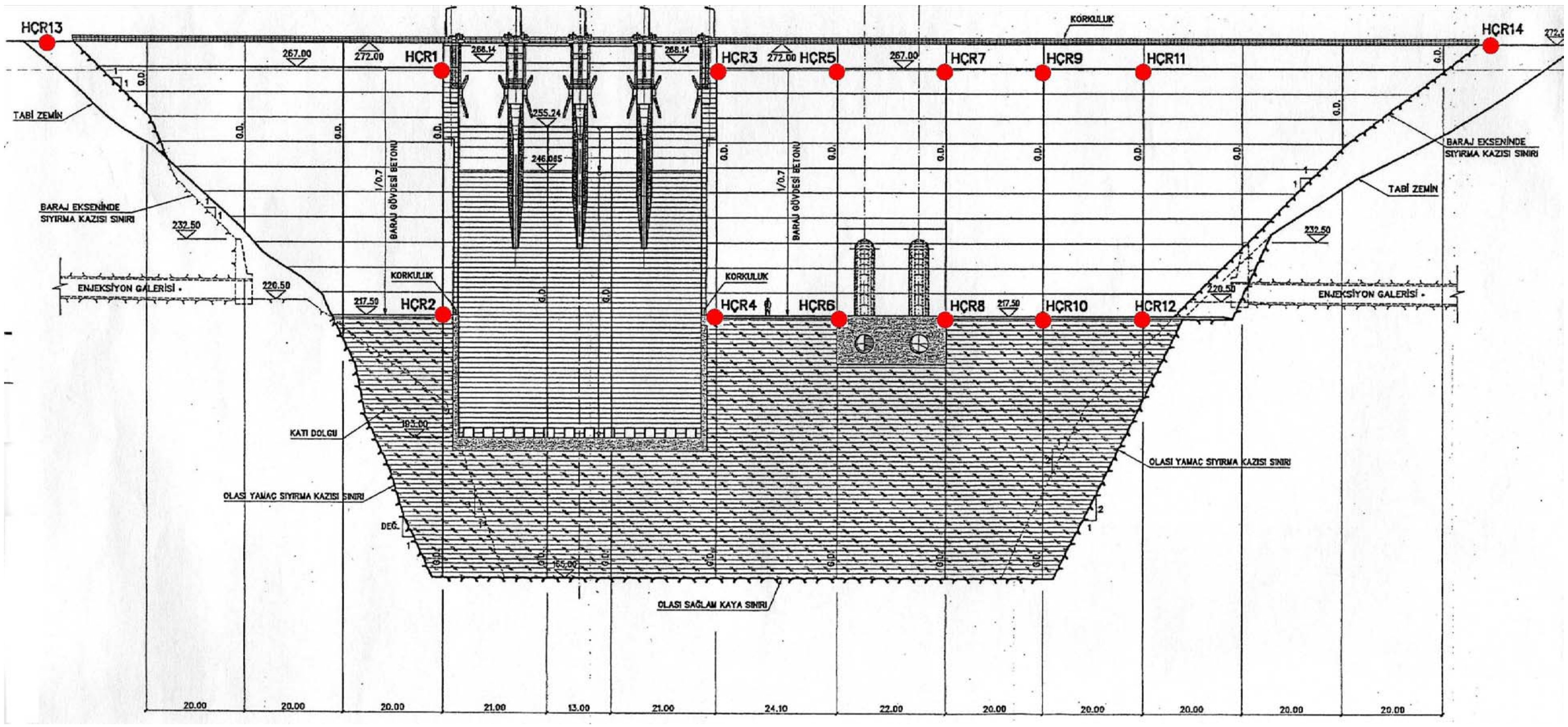

Figure 4.7. The layout of Survey Monuments at Cindere Dam (Downstream View), (Temelsu, 2003)

4.4.8 Other Instruments

Parameters to be monitored have been determined on the basis of several analyses. Instruments to be utilized were carefully chosen to monitor the key parameters. Installation of numerous instruments is neglected because an absolute necessity could not be proven or the system could not be promoted. Therefore, most instruments have been considered as just additional cost to the project. In this section, the reasons for negligence of instruments will be explained in detail. The instruments, which are considered to be installed, are inclinometers, pendulums and tiltmeters.

Inclinometers are capable of monitoring vertical, horizontal, axial, rotational and subsurface deformations. Vertical installations are common for concrete dams. A typical inclinometer consists of grooved casing, measurement probe and readout unit. Grooved casings are installed into drilled boreholes. The probe has wheels, which travel on the grooves of the casing. Deflections are sensed via probe wheels and transmitted to the readout unit. Inclinometers provide high accuracy, precision and resolution.

Pendulums are capable of monitoring vertical, horizontal, axial and subsurface deformations. They have been commonly installed inside boreholes or shafts of concrete dams to monitor verticality. Pendulums are simple instruments consisting of stainless steel wire, tensioning weight and bucket filled with damping fluid. The wire is anchored to the top of the structure and tensioning weight suspended inside the bucket at the lower end. Measurement can be performed via numerous units, e.g. mechanical, optical readout units. Pendulums provide high accuracy, precision and resolution. Tiltmeters have been utilized to measure changes in the inclination of dams. Permanent and portable models are commercially available. Rotational movements of concrete dams can be accurately detected via tiltmeters.

The necessity of each instrument will be discussed based on the finite element analysis results of Cindere Dam. The most critical loading condition is Service Condition + Operating Basis Earthquake. In this case, tensile stress occurs at the upstream toe, which may cause either formation of a crack or crest movement. Horizontal shear stresses and lateral deformation have been determined over tolerable limits at various points. The stress distribution and local stress concentrations around the lowest gallery are shown in Figures 4.8 and 4.9. However, analysis results indicate severe incidents, some measures were taken to enhance dam resistance and no detrimental effects are expected by considering the facts below (Temelsu, 2003).

a. Earthquakes are cyclic events and acceleration pulses have a short duration.
b. Foundation is formed of well-developed metamorphic schists without any significant shear or fault zone.
c. Site modifications have been carried out to enhance the shear strength at both upstream and downstream face.
d. In order to improve strength, enriched RCC mixture has been poured around the galleries where tensile stresses are maximum.

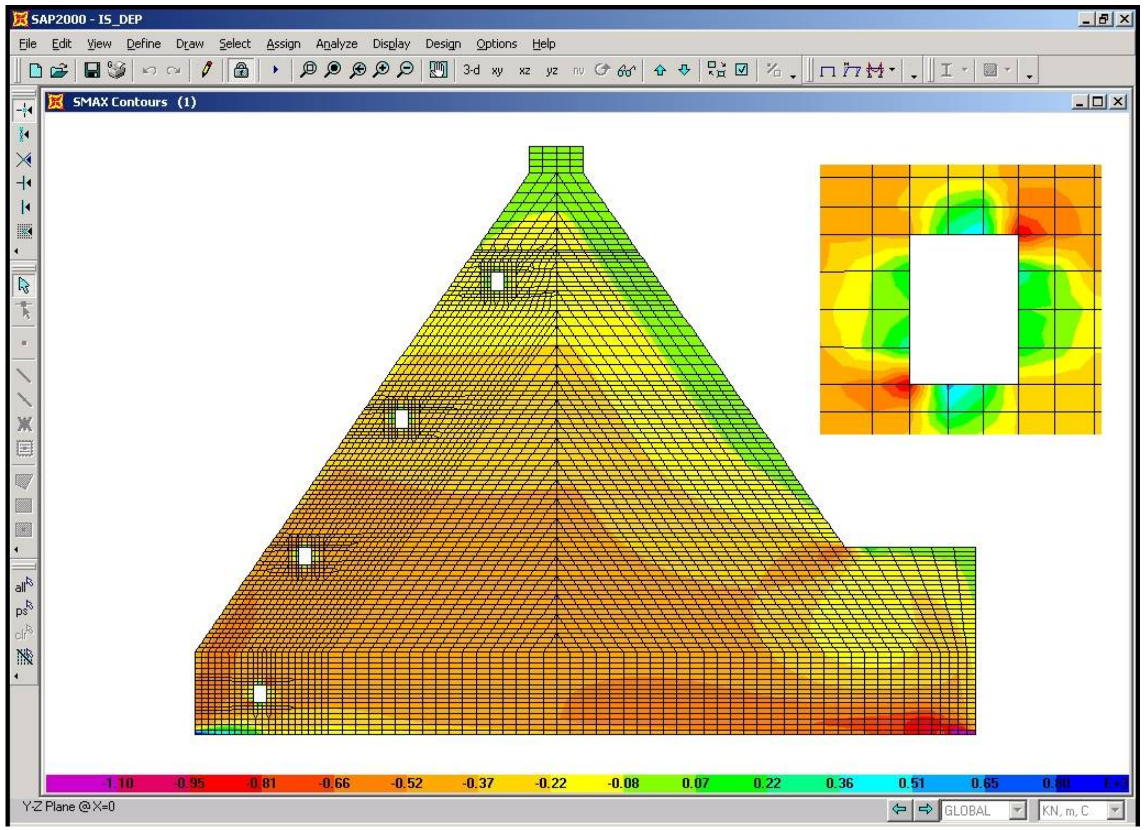

Figure 4.8 Major principal stress distribution in kPa, for the loading case of Service Condition + OBE (Gürdil, 2003)

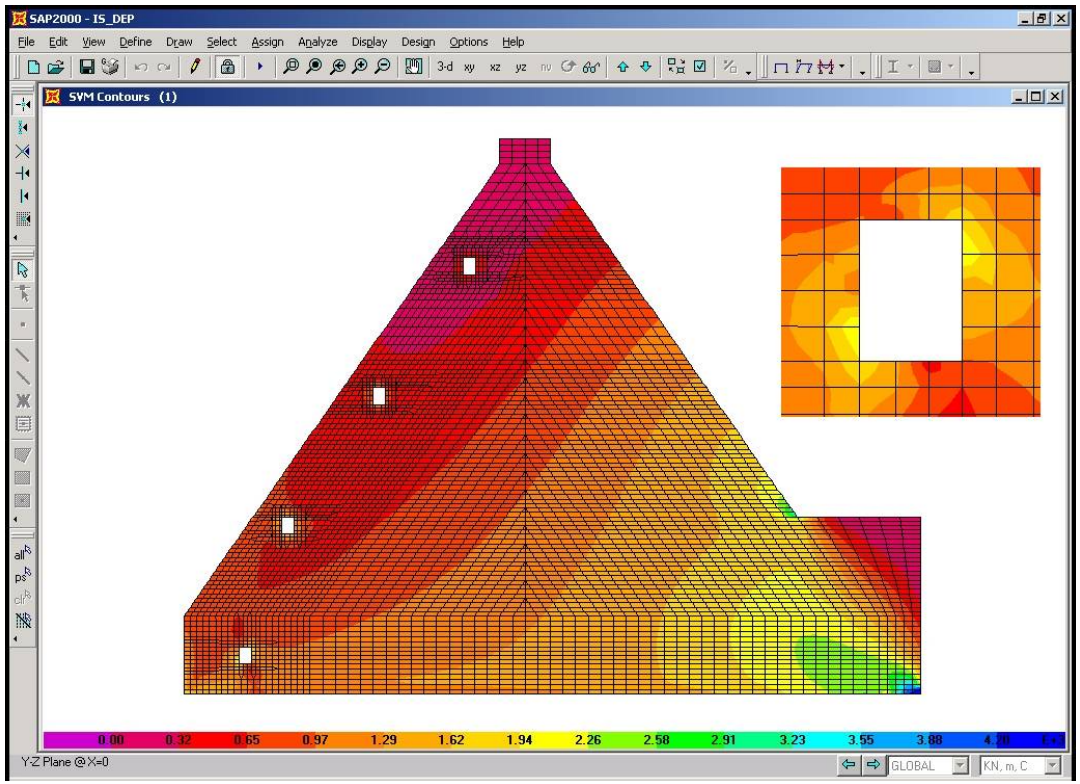

Figure 4.9. Horizontal shear stress in distribution in kPa , for the loading case of Service Condition + OBE (Gürdil, 2003)

Inclinometers or pendulums could be utilized vertically to measure vertical, horizontal and axial deflections. It has been found out that after the modification of upstream and downstream sections the safety factors against sliding have been significantly increased. Hence, surface monuments have been provided to observe vertical and horizontal changes. Monuments are inexpensive and less accurate instruments compared to inclinometers and pendulums. However, they are convenient because any possible deflections can be fairly tolerated by the dam. The inclined arrangement of the galleries is not suitable for placement of both inclinometers and pendulums. Additional shafts or boreholes are essential for installation. Drilling operations significantly increase the cost of instrumentation system. Tiltmeters are abandoned since rotational movements are unlikely because of symmetrical shape of dam body. Installation of mentioned instruments is neglected due to stated reasons above.

4.4.9 Strong Motion Accelerometers

Cindere Dam is located in an active seismic region. It is desired to instrument the dam with strong motion accelerographs to monitor behavior in case of earthquakes. Three tri-axial accelerometers (ACC) were installed at foundation level, dam body and the crest. The utilized accelerometers at the crest and the dam body are conventional type and installation procedure is relatively easier compared to the one at foundation level. A submersible waterproof strong motion accelerometer was provided at foundation level (See Figures 4.10 and 4.11). Prior to installation, a borehole was drilled up to specified elevation. A specially grooved stainless steel tube has been placed inside the borehole. The servo-accelerometer sensor equipped with guiding wheels facilitates the installation by traveling on the grooves of the tube. Each ACC in the system require no calibration and maintenance. In addition, an external triggering unit was wired up to each ACC to enable simultaneously recording as any ACC receives an earthquake trigger. The arrangement of instruments will be discussed in the following sections.

Figure 4.10. Installation of accelerometer tube at Cindere Dam

Figure 4.11. Servo-accelerometer sensor equipped with guiding wheels, which was located at foundation level at Cindere Dam

4.5 The Current Data Acquisition System

Cindere Dam was equipped with a modern automatic data acquisition system (ADAS). The current ADAS is capable of transmitting the collected data to the Headquarters of DSİ in a real-time manner. A functional block diagram of the current ADAS is given in Figure 3.6. During construction, data required for engineering assessment to ensure the construction safety is acquired via portable readout units. Two personnel were assigned to manual data acquisition duty. Technicians who have electrical and mechanical ability were chosen and trained for one month. In this intensive training program, basic information about instruments, taking measurements, record sheets, etc., were introduced. During the construction, technicians have just taken instrument measurements. Data reduction, processing, interpretation and reporting have been accomplished by the qualified engineers of authorized Instrument Provider Company. No abnormal measurements, which may indicate a critical situation, have been witnessed until the end of the construction.

Instruments, which are suitable for automatic data acquisition, were connected to ADAS House built at 220.50 m elevation downstream. In order to minimize data transmitting losses, proper type shielded cables were selected for each instrument. Cables were routed from the sensors to ADAS House passing through galleries and stairs shafts (M1, M2, M3 and M4). For concrete embedded instruments, cables were housed in PVC pipes to the galleries. Cables were conveyed on heavy-duty PVC cable trays from the galleries to the ADAS House. Due to the enhanced durability in humid medium, PVC cable trays were preferred over conventional galvanized cable trays. Occasionally, installation of cables cannot be implemented properly. Cables installed inside the cable trays may follow a zigzag or “S” trend, which will considerably extend the cabling path. In order to be on the safe side, a safety factor of 1.20 was taken for cable lengths.

A typical cabling path is indicated via red arrows in Figure 4.12 for the sensors gathered in 170.50 m gallery to the ADAS House.

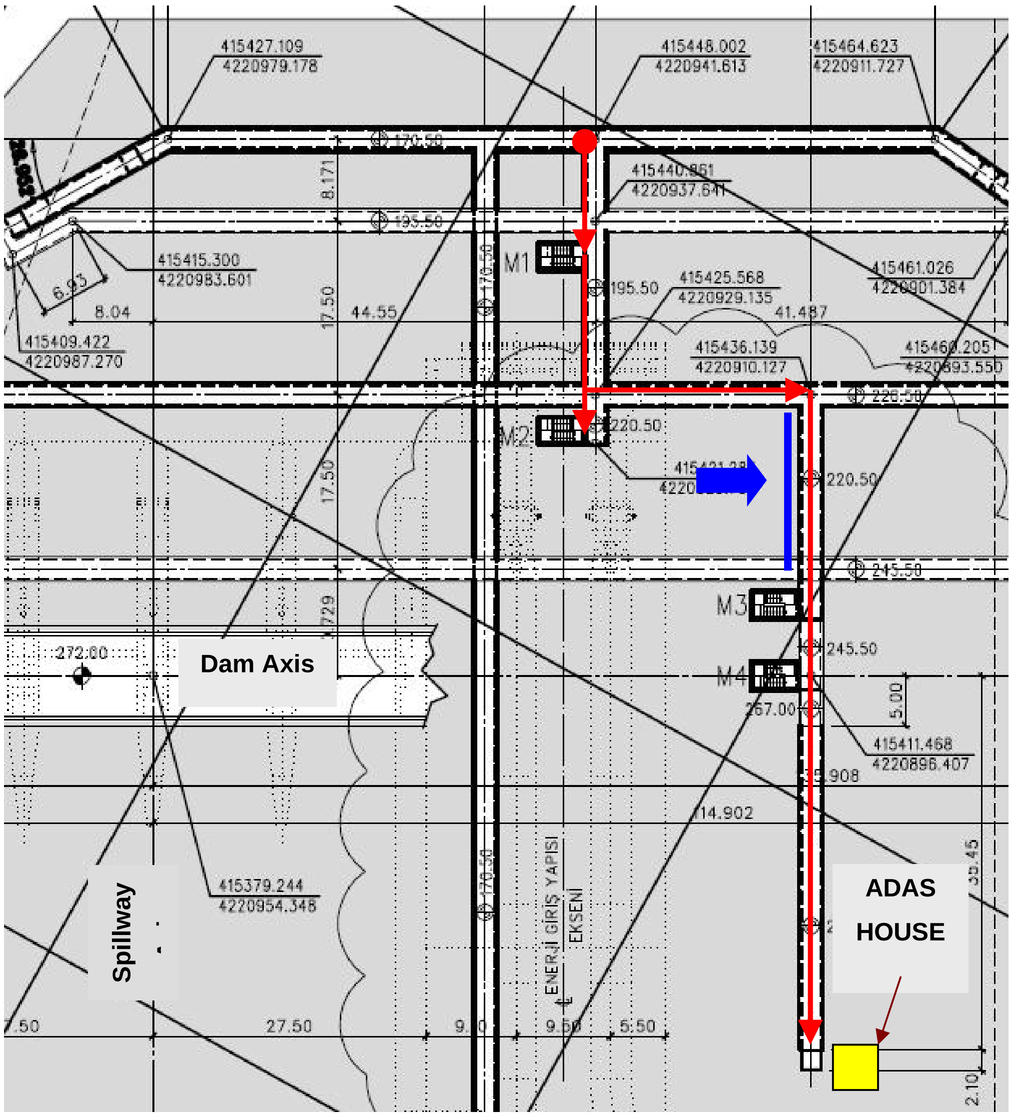

Figure 4.12. A section from plan view of Cindere Dam (Adapted from Temelsu, 2003)

The current ADAS comprise all the components of conventional manual data acquisition systems (MDAS) such as switch terminal boxes, portable readout units, etc. Switch terminal boxes (SWBX) allow collecting instrumentation data manually at an easily accessible location via portable readout units or data loggers. Totally, 126 sensors were utilized in the current system except accelerometers. ACC were not taken into account for both MDAS and ADAS since they have their own strong motion recorder units. These sensors were connected to SWBX, which are located inside the 220.50 m gallery close to the ADAS House (See Figure 4.13). Seven SWBX with 18 ports and one SWBX with 24 ports were installed. A maximum capacity of 150 sensors was provided by the system for redundancy and to permit installation of additional instruments. Portable readout units for both vibrating wire and electrical resistance devices have been readily procured and staff gauges for each V-notch have been provided.

Figure 4.13. Automatic Data Acquisition System House located at the 220.50 m elevation downstream

As explained in Chapter 2, automatic data acquisition systems are complex structures, which include multiplexers (MUX), advanced data logger systems controlled via multi-logger software. MUX were used to increase the number of sensors that can be monitored via data loggers. The capacity of MUX is directly dependent on the type of the sensors. A typical MUX is capable of multiplexing 32 sensors requiring two conductors and 16 sensors requiring four conductors. Each VW instrument installed at Cindere Dam requires four conductors for data transmission since they have an integrated temperature sensor. Totally, eight MUX were installed for 126 instruments. A single data logger utilized at the project is capable of controlling six MUX. Therefore, two data loggers interconnected to MUX. All the components of the system have Electromagnetic Interference (EMI) protection and were placed inside the ADAS House. The multi-logger software enables simultaneously carrying out data collection, reduction and processing according to a predefined schedule. Interpretation and reporting of instrumentation data will be performed by qualified engineers at Headquarters of DSİ.

The installation of ADAS was accomplished by a group of qualified technicians and engineers. Power requirement of system was critically discussed. An uninterrupted power source was provided via backup solar panels. The protection against adverse environmental and weather effects was provided by installing all the components inside the ADAS House.

4.5.1 Design of an Alternative Data Acquisition System

As an alternative, a manual data acquisition system (MDAS) equipped with SWBX and portable readout units, is considered. The components of MDAS are designed to operate in harsh conditions. In order to reduce cabling cost, a suitable location is chosen for SWBX at the mid-section of 220.50 m gallery (indicated with blue arrow in Figure 4.12). For long-term operation, a monitoring schedule is defined by considering project conditions and requirements. Two technicians who have basic qualifications in civil engineering will take an extensive training program. Then, they will carry out manual measurements and visual observations according to the predetermined schedule. Data collection, reduction and processing will be executed by technicians under the supervision of a qualified engineer. The data interpretation and reporting stages will be performed by experienced engineers at the Headquarters of DSİ.

The initial and life cycle cost of each system will be discussed in the following section. Several comparisons between systems in many aspects will also be introduced.

CHAPTER 5

ALTERNATIVE INSTRUMENTATION SYSTEM

5.1 General

The current instrumentation system is carefully examined and found that it is a convenient and reliable system. Alternative instrumentation system can be considered as a contribution to the current system. The primary purpose of designing such an alternative system is to check whether a more reliable and maintainable system can be achieved or not. A comparison regarding features and cost is carried out between the current and alternative instrumentation system including both manual and automatic data acquisition configurations. During the examination of the current system, it is determined that four modifications stated below are required in order to increase the reliability and maintainability of the system.

a. Volumetric Change Measurements (New parameter)
b. Additional Temperature Sensors
c. Replacement of seepage measuring instruments
d. Modifying the arrangement of strong motion accelerometers

5.2 Volumetric Change Measurements

Concrete dams exhibit thermal volumetric changes either expansion or shrinkage which is mainly caused by hydration reaction of concrete and heat differentiation between dam sections. These volumetric changes can be precisely measured in terms of strains via no-stress strain gauges. These gauges are almost identical to conventional concrete embedded strain gauges. The only difference is the installation method. As also explained in Section 2.5.5.3, no-stress strain gauges are installed inside a special casing, which absorbs external loads from any direction. A typical casing has single open end that allows concrete pouring around the internal strain gauge. Hence, the internal strain gauge surrounded with the concrete only experiences the same volumetric changes of the mass concrete.

In the absence of no-stress strain gauges, it is a necessity to measure temperature along with each measurement of strain. Temperature measurements can be carried out via either additional instruments or specific built-in sensors of instruments. Generally, conventional strain gauges are integrated with temperature sensors for the compensation of temperature-induced errors. In order to determine the magnitude of real strains in concrete, a theoretical correction process must be implemented. A typical numerical equation is shown in Equation (5.1) to determine the strain in the concrete (SIS, 2008).

$$\mu\varepsilon_{\text{Load}} = (L_e - L_0) + (T_e - T_0).\ (CF_1 - CF_2) \tag{5.1}$$

where,

$\mu\varepsilon_{Load}$	: micro strains due to load change
L_e	: initial strain reading
L_0	: subsequent strain reading
T_e	: initial temperature reading (^{0}C)
T_0	: subsequent temperature reading (^{0}C)
CF_1	: thermal expansion coefficient of instrument material
CF_2	: thermal expansion coefficient of concrete

Note: The thermal expansion coefficients in Equation (5.1) are in micro-strain / ^{0}C.

The strains calculated by above equation are questionable due to assumptions made for the thermal coefficient of the concrete. Installing no-stress strain gauges adjacent to the conventional strain gauges eliminates the necessity for thermal expansion coefficient of concrete. Strains caused by thermal changes can directly be monitored via no-stress strain gauges. The combination of these gauges in a line enables correlation between measurements. The readings acquired from no-stress strain gauges can be either subtracted or added to conventional gages' readings to obtain actual stress within concrete. Thus, more accurate strain measurements can be acquired to evaluate the dam performance.

The primary purpose of utilizing no-stress strain gauges at Cindere Dam is to promote the accuracy of readings obtained via conventional strain gauges. In addition, no-stress is capable of directly measuring thermal changes in terms of strains, which enables us to evaluate thermal behavior of the relatively new construction material "hardfill" from pouring stage to service life of the dam.

At Cindere Dam, totally six no-stress strain gauge are placed in the dam body as additional instruments adjacent to the current conventional strain gauges as shown in Figure 5.1. No-stress strain gauges are installed at the same elevation of the current instruments to reduce the cost regarding the installation schedule. In order to obtain representative data of overall behavior of the dam, a simple uniform layout is preferred. This instrument arrangement also facilitates correlation between measurements, installation of cables and reduces the risk of accidental damage to cables.

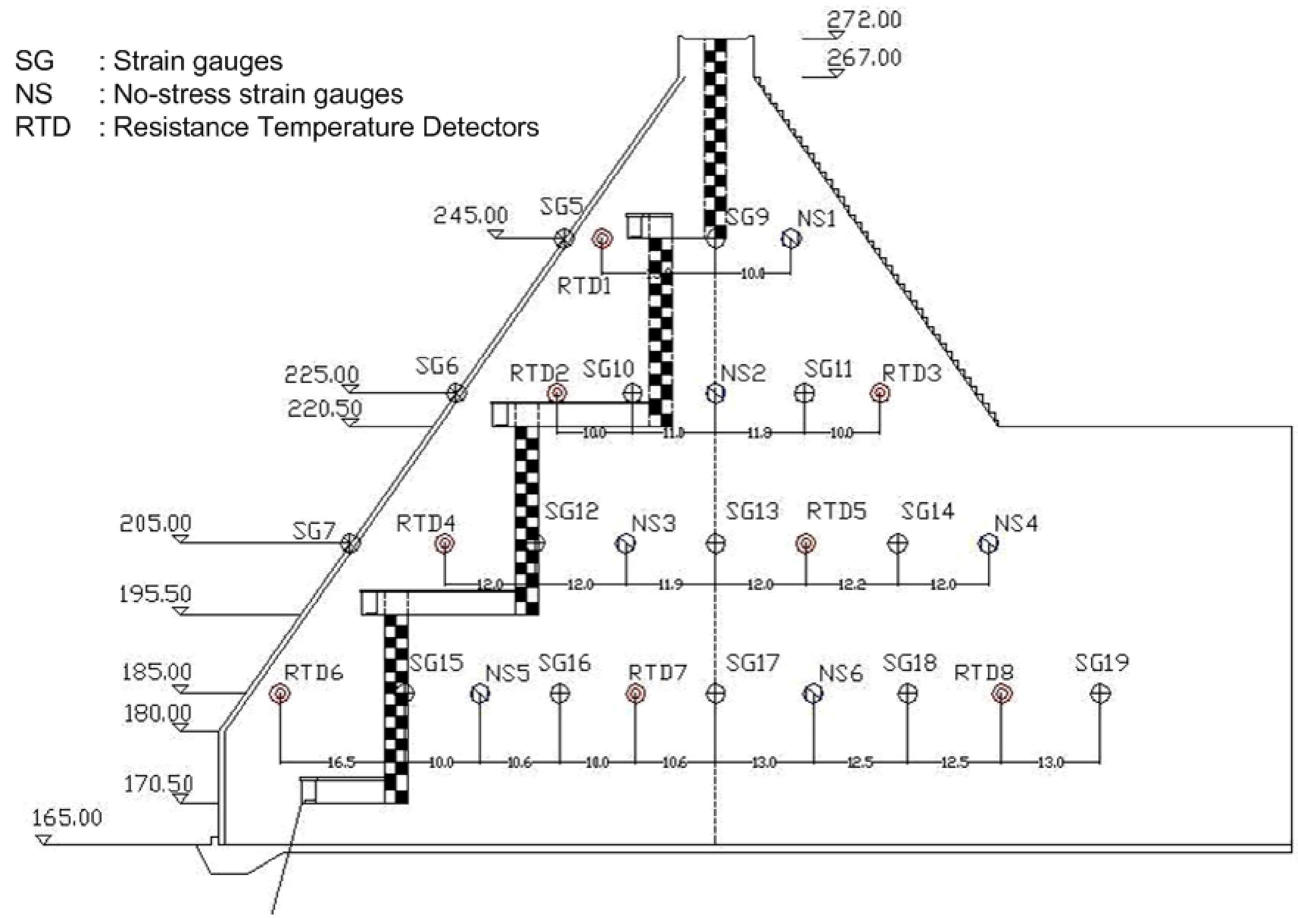

Figure 5.1. Typical cross section of the alternative instrumentation system

This application cancels the requirement for built-in temperature sensors of conventional strain gauges. Providing conventional strain gauges without temperature sensors may reduce the cost of the system. However, it is always advisable to measure temperature at numerous points to detect the temperature distribution and orientation within dam body.

Readings obtained via no-stress strain gauges may also be correlated with the measurements of joint meters and any other instruments equipped with temperature sensors. The comparison between actual thermal concrete expansions obtained via no-stress strain gauges and theoretically, calculated thermal concrete expansions provide valuable data to develop a specific concrete expansion coefficient for the relatively new construction material “hardfill”.

5.3 Additional Temperature Sensors

Measurement of temperature is critical for concrete dams since concrete mixture solidifies and hardens as a consequence of chemical exothermic reaction which is called hydration. Hydration effects are severe especially for mass concrete structures due to the large magnitude of thermal variations. Volumetric changes may result in concrete cracking which takes place within dam body because of thermal variations. It is not tolerable for concrete dams since the structural integrity is vital. Therefore, it is essential to monitor precisely temperature level and distributions through the dam.

Various kinds of instruments are commercially available for temperature measurement. Temperature measuring instruments installed at concrete dams serve for two main purposes; to modify the rate of concrete curing and to compensate the measurement obtained via an instrument sensitive to the temperature fluctuations.

Hydration effects are more remarkable for the 28-days day period after concrete pouring. During construction, the excessive temperature rise is controlled by concrete curing. Concrete curing process consists of cooling and other measures. The rate of cooling is so important that rapid or inadequate cooling may result in shrinkage and expansion cracks, respectively. Installation of sensors capable of measuring temperature enables engineers to modify the rate of curing to avoid thermal cracking. Installation of temperature measuring devices adjacent to the instrument, which are sensitive to temperature variations, enables compensation for thermal changes.

At Cindere Dam, no independent temperature sensors were used except built-in ones. In this study, both above-mentioned functions of temperature measuring devices are desired to be utilized. Resistance Temperature Detectors (RTD) are chosen over thermocouples and VW temperature sensors. Thermocouples are disregarded because of short-term service life. Both VW temperature sensors and RTD are suitable for long-term operations. VW temperature sensors are considerably expensive although they provide higher resolution. RTD are considered convenient to obtain reliable data.

Totally eight RTD are placed in the dam body adjacent to the current conventional strain gauges as shown in Figure 5.1. RTDs are installed at the same elevation of the current instruments to reduce the cost regarding the installation schedule. In order to obtain representative data of overall behavior of the dam, a simple uniform layout is preferred. This instrument arrangement also facilitates correlation between measurements, installation of cables and reduces the risk of accidental damage to cables.

An arrangement of three different instruments is achieved in a line. Correlations between these instruments will significantly increase the accuracy of the measurements. It is assumed that is sufficient to constitute conventional strain gauges and no-stress gauge in a line to obtain accurate measurement. However, RTDs advance the system in terms of redundancy by using more closely spaced different instruments in a line to measure the same parameter. Moreover, RTDs placed close to the upstream face also allow correlation between the conventional strain gauges installed within the 1.0 m thick concrete face. Each RTD plays also an important role to obtain a better picture for the temperature distribution and orientation within the dam body. Hence, curing rate during construction can be determined more precisely. Furthermore, the enhanced system with RTDs and no-stress strain gauges will provide representative data during long-term structural behavior monitoring for Cindere Dam. These data are quite valuable for a relatively recently developed dam type.

5.4 Replacement of Seepage Measuring Instruments

Seepage measurements at concrete dams have been performed via various instruments. The current seepage measurement instruments are V-Notch weirs at Cindere Dam. Two V-notch weirs were placed at the drainage gutters of 220.50 m and 245.50 m elevation galleries. These instruments were equipped with water pressure transducer units to enable automatic data acquisition. Extensive attention paid for the selection of features and installation of V-Notch weirs to be capable of acquiring accurate data.

The current system is evaluated as convenient, however, another alternative instrument for seepage measurement called Parshall flume is considered in this study. The primary purpose of utilizing Parshall flume instead of V-notch weirs is to promote the system in terms of maintainability. The greatest disadvantage of V-notch weirs is the vulnerability of sediment deposition and algae formation. Therefore, weirs need regular cleaning.

Especially for the full-automated systems where no stationary personnel are essential, performing such a regular maintenance might be at risk. In case maintenance fail, the accuracy of seepage measurement may adversely be affected in a great deal.

The water sources in the vicinity of Cindere Dam are very rich in mineral content. As it can be seen in Figure 5.2, during initial filling of the dam, accumulation of the minerals was observed at numerous points of the galleries. Therefore, V-notch weirs are highly susceptible to sediment deposition. Because of self-scouring ability, which eliminates sedimentation problems, utilizing Parshall flumes is a reasonable approach. More detailed information about Parshall flumes are given in Section 2.5.4.2.

Figure 5.2. Accumulation of minerals at the 195.50 m gallery of Cindere Dam during initial filling

Two Parshall flumes are installed at 220.50 m and 245.50 m elevation galleries in place of the current V-Notch weirs. Each flume is made of durable fiberglass and equipped with water pressure transducer unit for automatic data acquisition. The capacity of flumes is directly dependent on the throat width. The 3-inch throat sized flumes are chosen since they provide approximately 10.0 liter per second capacity, which is almost identical to weirs. The necessary site modifications and inspections are also identical with V-notch weirs as explained in Section 4.4.5.

5.5 Modifying Arrangement of Strong Motion Accelerometer

Cindere Dam is located in a high seismic region. It is observed that, various conservative assumptions were made and numerous computer-based finite element analyses were carried out to assess the dam performance in case of earthquakes. In the current system, three accelerometers were installed at the locations (foundation, dam body and crest) to monitor seismic behavior of the dam. Generally, utilizing three accelerometer with proper arrangement is considered as adequate for concrete dams. Installation points for a typical arrangement are given below.

a. The first accelerometer must be located at either abutments or foundation if any non-uniform motion is expected either along the dam-abutment or dam-foundation interface due to topographical, inertial and energy dissipation property differences.
b. The second one called free-field instrument must be located near the dam, which allows correlation between the other accelerometer in the arrangement.
c. The third one must be located at mid-span of the crest where the maximum modal deflections are anticipated.

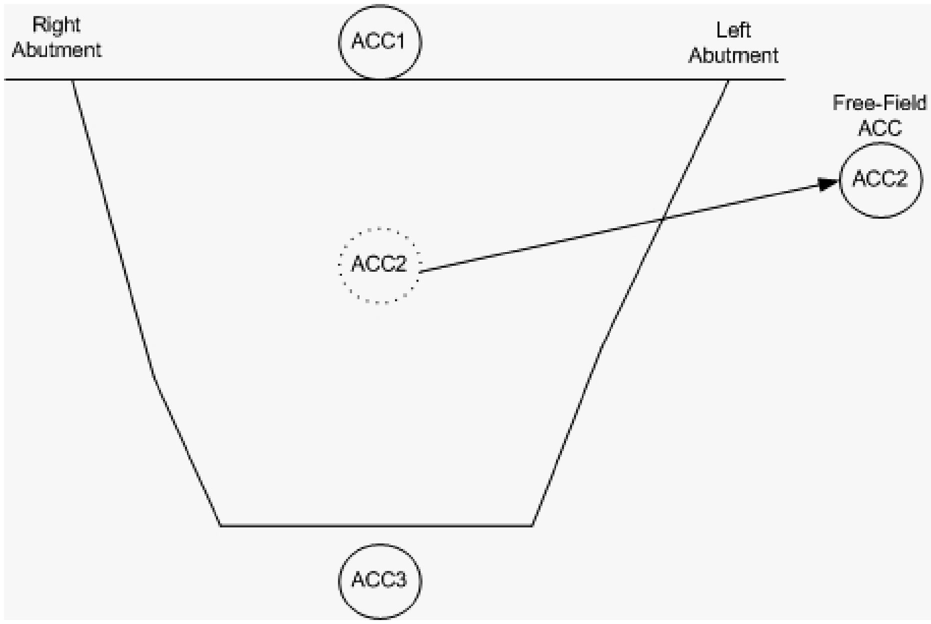

Figure 5.3. Replacing the place of the current accelerometer with free-field accelerometer at Cindere Dam (Downstream View)

(Not to Scale)

At Cindere Dam, all the accelerometers were installed properly at the expected locations except the second one, which was installed within the dam body. In this study, a free-field instrument is provided instead of the one placed within the body (See Figure 5.3). The assessment of the earthquake behavior of dams can only be checked by way of comparison with regard to field observations. Therefore, a free-field instrument is essential. The free-field accelerometer should be located at a certain distance in order not to be influenced by vibrations caused by dam. The distance should also be determined correctly to obtain representative data from the site. A typical distance is 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 the dam concrete (Darbre, 1995).

The height of Cindere Dam is 107 m from foundation level and the modulus of elasticity of foundation is 5.0×10^6 kPa while the modulus of elasticity of dam concrete is 1.0×10^7 kPa. Therefore, the distance of free-field accelerometer from the dam is to be twice the dam height, 214 m and can be rounded up to 215 m.

Several appurtenant structures are located on the right side of the dam. The replaced accelerometer, ACC2 should be installed on the left side of the dam away from these structures in order to prevent from the artificially induced vibrations. An external common triggering unit will be wired up to each accelerometer to enable simultaneous recording of accelerations generated by nearby earthquakes. The main purpose of common triggering unit is to activate all accelerometers at a time, in case it senses any earthquake trigger. This application prevents accumulation of undesired data and provides complete data coverage.

A comparative information on the type and the quantities of instruments utilized in the current and alternative instrumentation systems is presented in Table 5.1.

Table 5.1. Instruments utilized at current and alternative instrumentation systems

Instrument	Parameter	Current System	Alternative System
Pressure Cells	Stress	15	15
Piezometers	Pore Water Pressure	17	17
Strain Gauges 2-D	Stress	11	11
Strain Gauges 3-D	Stress	12	12
Joint meters 2-D	Relative Movement	16	16
Strong Motion Accelerometers	Vibration	3	3
Reservoir Level Gauges	Water Level	2	2
V-Notch Weirs	Seepage Rate	2	-
Parshall Flumes	Seepage Rate	-	2
No-stress Strain Gauges	Volumetric Change	-	6
RTDs	Temperature	-	8
Survey Points	Horizontal and Vertical Movements	14	14

5.5.1 Cost of Instrumentation Systems

In this study, an alternative system is considered based on current system. The alternative system is designed with additional instruments and different instrument arrangements. The two systems are equipped with both manual and automatic data acquisition systems. Hence, totally four systems are established and the cost calculations are carried out separately for each one.

5.5.2 Assumptions for Calculations

First, a common unit cost is determined. All prices are calculated by using a fixed reference unit price (RUP). The RUP is selected as the price of one-meter length of PVC 20 mm pipe and the other prices are proportionally converted to RUP. Hence, each cost figure is given without a specific currency unit. In order to provide a better comprehension, the cost parameters are divided into four main groups. These are instrument, cabling, installation and operation cost. Packing, delivery and insurance costs are considered as 1% of total cost of instrument and cabling.

5.5.3 Instrument Cost

Instrument cost includes the total cost of instruments, data acquisition system components and accessories for installation. The prices of the instruments are determined based on the catalog prices of various manufacturers. Instruments having the same features are compared in terms of prices and an average cost is taken as final cost and converted to RUP.

5.5.4 Cabling Cost

Cabling cost comprises the total cost of PVC pipes, PVC cable trays and cables. Cables utilized at Cindere Dam are selected as two types; regular and multi-core. The cable lengths are calculated by considering a safety factor of 1.20.

5.5.5 Installation Cost

The installation of instruments is carried out in nine stages in accordance with a work breakdown schedule. As the elevation of construction rises, instruments are installed to the specified location at predetermined elevations. The installation cost mainly includes workmanship and drilling. In this study, the unit cost of workmanship breakdown is given below.

a. Labor Cost (man-hour) : 15 RUP
b. Travel Expense (price per person) : 40 RUP
c. Per Diem (price per person) : 20 RUP
d. Accommodation (price per person) : 40 RUP

The value of labor cost, travel expense, per diem and accommodation are determined according to the current prices used in a typical commercial private Turkish company. A typical one-day duty cost consists of 8 man-hours, a round-trip bus ticket, three days per diem and two days accommodation fee for per person. The reader is suggested to refer to Appendices B, C, D, E and F for details.

It is assumed that installation of each instrument will be implemented in two man-hours. Before installation, the required site modifications such as drilling, providing installation pits, installation of cable trays, etc., are assumed to be carried out by the contractor. The required man-hour to set up MDAS and ADAS are assumed as 32 and 80 man-hours, respectively. Drilling cost is calculated separately due to the requirement of special machinery equipment and staff. The drilling length is calculated by considering a safety factor as 1.05. Drilling length for the new free-field strong motion accelerometer is assumed as 25 m in cost calculations. Installation survey points (SP) are assumed to be carried out after the completion of the construction, during the set up of data acquisition systems, simultaneously.

5.5.6 Operating Cost

Operating cost of instrumentation system mainly includes cost of maintenance of equipment, man-hours required for taking measurements and training for new personnel. In this study, operating cost of ADAS is neglected since no stationary personnel are essential and some system components only require calibration for every two years. Operational cost calculations are performed only for MDAS systems. For MDAS systems, two technicians who have basic qualifications in civil engineering are assigned to manual data acquisition duty for long-term operation. It is assumed that a salary of 650 RUP per month will be paid to each technician for the first year. The initial salary is determined based on the current governmental regulations and increase of salary is taken as 5% per year. Prior to regular measurements commence at the dam, an extensive training program will be given to the technicians for two months. The training cost is 1000 RUP per technician.

Based on the above assumptions, all calculations regarding instruments, cabling, installation and operating costs are carried out according to the current documents of Cindere Dam including design drawings. As a result of this study, all cost breakdown tables for each configuration of systems are given in Appendices B, C, D, E, F and G. Summary tables based on these computations are presented in the following section.

5.6 Summary of Cost Calculations

Summary of cost calculations for each system is given in Table 5.2 through Table 5.5. The initial and life cycle cost of instrumentation systems are also presented in graphical form in Figures 5.4 through Figures 5.6

Table 5.2. Cost of the Current Instrumentation System with ADAS

Instrument ID	Instrument Cost	Cabling Cost	Total Cost
Pressure Cells	18.000	28.166	**46.167**
Piezometers	15.725	34.736	**50.461**
Strain Gauges	22.749	38.572	**61.322**
Joint Meters	34.368	38.216	**72.585**
Reservoir Level Gauges	2.200	1.681	**3.881**
V-Notch Weirs	4.500	1.544	**6.044**
Accelerometers	113.785	-	**113.785**
Subtotal	**211.327**	**142.915**	**354.245**
Data Acquisition System	97.790	-	**97.790**
Pack-Deliver-Insurance			**4.520**
Installation Cost			**28.095**
Grand Total (RUP)			**484.650**

Table 5.3. Cost of the Current Instrumentation System with MDAS

Instrument ID	Instrument Cost	Cabling Cost	Total Cost
Pressure Cells	18.000	20.030	**38.031**
Piezometers	15.725	25.741	**41.466**
Strain Gauges	22.749	25.589	**48.339**
Joint Meters	34.368	21.772	**56.140**
Reservoir Level Gauges	2.200	1.076	**3.277**
V-Notch Weirs	2.410	0	**2.410**
Accelerometers	113.785	0	**113.785**
Subtotal	**209.237**	**94.208**	**303.448**
Data Acquisition System	27.760	0	**27.760**
Pack-Deliver-Insurance			**3.312**
Installation Cost			**27.015**
Grand Total (RUP)			**361.535**

Table 5.4. Cost of Alternative Instrumentation System with ADAS

Instrument ID	Instrument Cost	Cabling Cost	Total Cost
Pressure Cells	18.000	28.166	**46.167**
Piezometers	15.725	34.736	**50.461**
Pressure Cells	22.749	38.572	**61.322**
Joint Meters	34.368	38.216	**72.585**
No-stress Strain Gauge	2.100	5.154	**7.254**
Resistance Temperature Detector	840	6.795	**7.635**
Reservoir Level Gauges	2.200	1.681	**3.881**
Parshall Flumes	4.410	1.544	**5.954**
Accelerometers	124.255	-	**124.255**
Subtotal	**224.647**	**91.962**	**379.514**
Data Acquisition System	101.575	-	**101.575**
Pack-Deliver-Insurance			**4.811**
Installation Cost			**30.068**
Grand Total (RUP)			**515.968**

Table 5.5. Cost of Alternative Instrumentation System with MDAS

Instrumentation System Cost			
Instrument ID	**Instrument Cost**	**Cabling Cost**	**Total Cost**
Pressure Cells	18.000	20.030	**38.031**
Piezometers	15.725	25.741	**41.466**
Pressure Cells	22.749	25.589	**48.339**
Joint Meters	34.368	21.772	**56.140**
No-stress Strain Gauge	2.100	3.235	**5.336**
Resistance Temperature Detector	840	4.360	**5.201**
Reservoir Level Gauges	2.200	1.076	**3.277**
Parshall Flumes	1.980	-	**1.980**
Accelerometers	124.255	-	**124.255**
Subtotal	**222.217**	**56.033**	**324.025**
Data Acquisition System	0	0	**27.760**
Pack-Deliver-Insurance			**3.518**
Installation Cost			**28.988**
Grand Total (RUP)			**384.291**

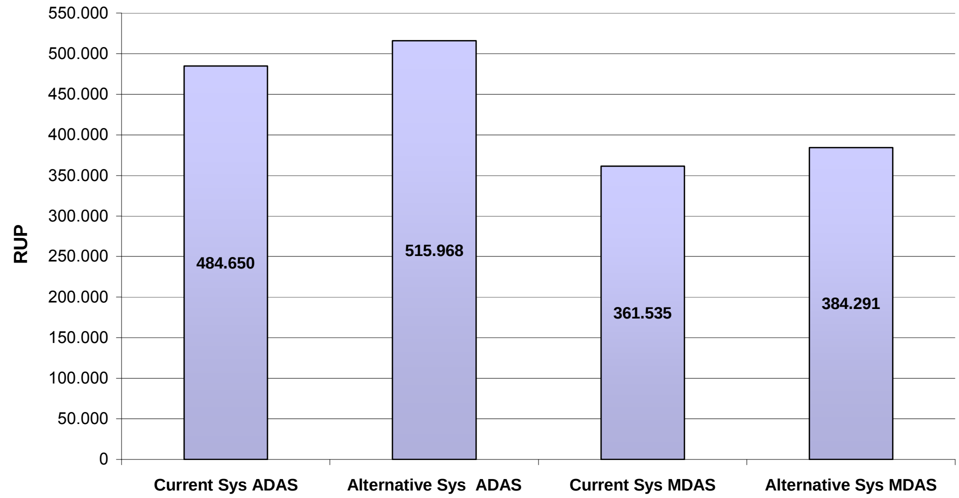

Figure 5.4. Initial cost of the systems

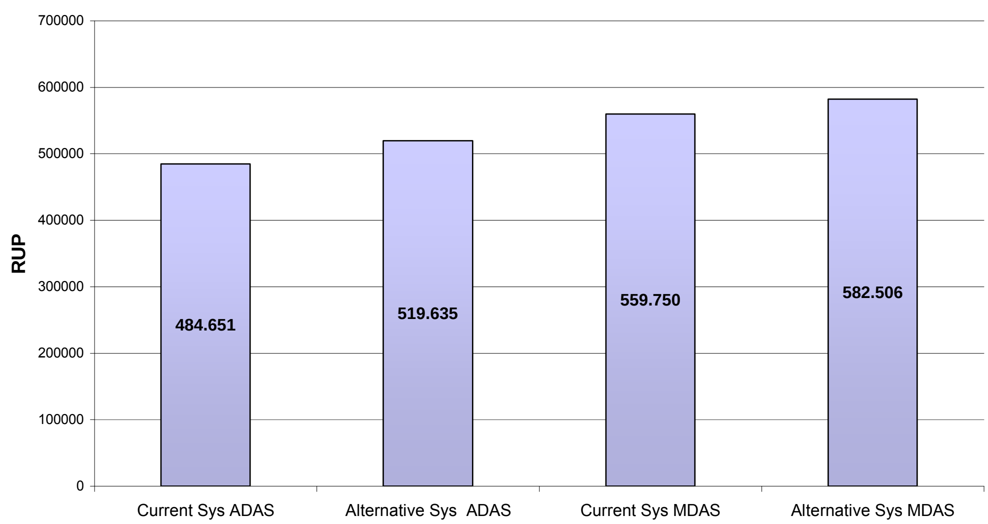

Figure 5.5. Life cycle cost of the systems on the 10th year

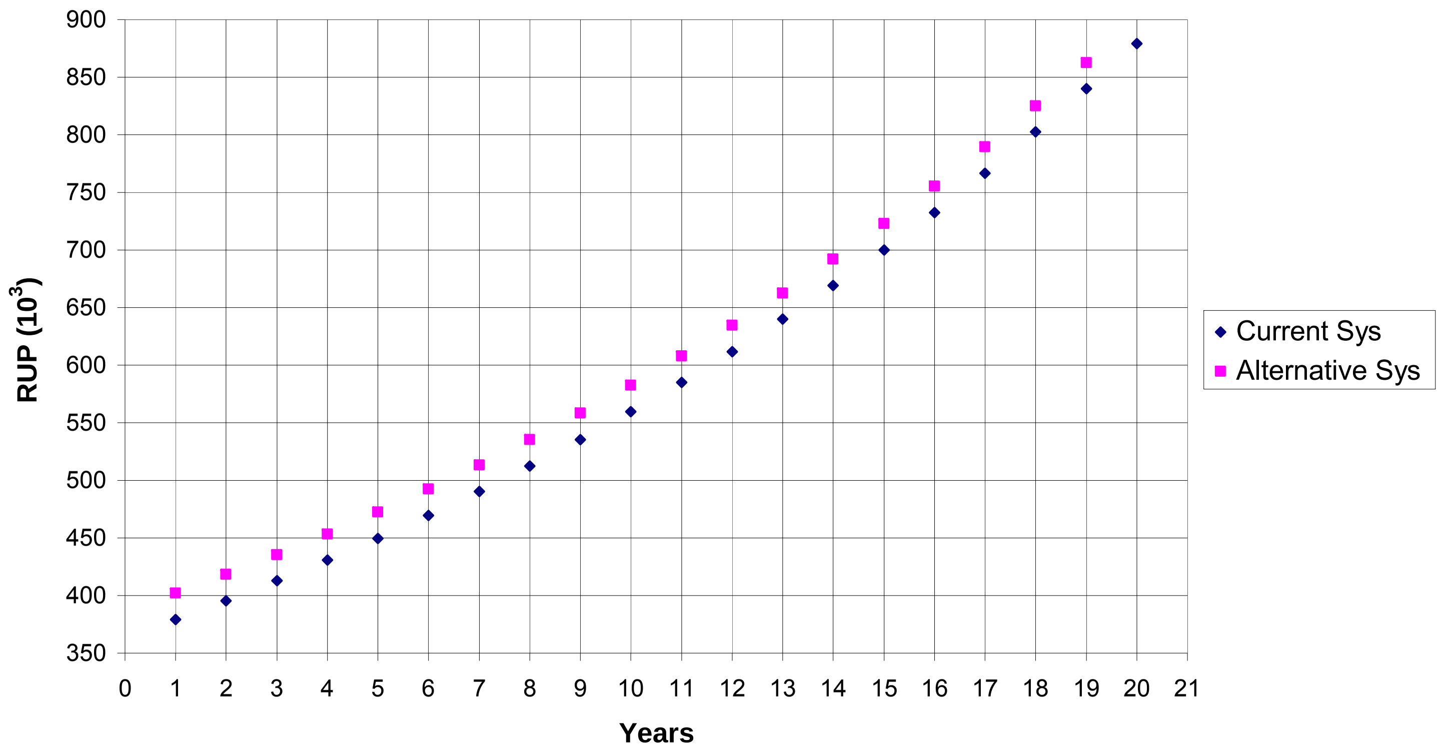

Figure 5.6. Life cycle Cost of Instrumentation Systems with MDAS

As it can be seen from Tables 5.2 through 5.5, cabling costs are significantly higher than instrument costs disregarding the type of data acquisition systems. Therefore, during the design phase cabling cost must be seriously taken into account. As illustrated in Figures 5.4 and 5.5, initial costs of manual data acquisition systems are relatively low compared to the automatic data acquisition systems. The main cost difference between ADAS and MDAS is resulted from operational efforts. The MDAS requires stationary personnel to take regular measurements with a predetermined schedule while ADAS not. The readers are suggested to refer Section 5.5.6 for more detailed information.

In order to obtain a representative picture, it is essential to make a comparison between systems within a certain period. Hence, the operating cost of manual data acquisition systems are calculated for twenty years (See Figure 5.6). It is observed that, life-cycle costs of manual systems are considerably high compared to automatic systems.

As illustrated in Figure 5.6 life-cycle cost of current and alternative MDAS reach the cost of ADAS within seven years and eight years, respectively. In addition, the total man-hour cost of current and alternative MDAS become almost equal to initial instrumentation investment cost of MDAS at the 16th and 17th years, respectively. As a consequence of this comparison, it can be stated that, although, the initial cost of ADAS is higher than MDAS, it pays back the additional investment cost within less than ten years.

CHAPTER 6

CONCLUSIONS

In this study, an alternative instrumentation system is designed based on current instrumentation system of Cindere Dam. The two systems are equipped with both manual and automatic data acquisition systems. Totally, four different instrumentation systems are achieved including the current one. From the evaluation and comparison of these systems, the following conclusions can be drawn:

1. The current instrumentation system of Cindere Dam has been found to be satisfactory and maintainable.

2. The alternative instrumentation systems promote the current system in many aspects without increasing the cost in a great deal.

3. Although the initial cost of manual data acquisition systems is relatively low, life-cycle cost is found considerably high compared to properly designed automatic data acquisition systems in this study. However, this result cannot be generalized and similar analyses should be carried out for each dam project, which may exhibit entirely different features than those of Cindere Dam.

4. Installation of no-stress strain gauges adjacent to conventional strain gauges in a line not only increases the accuracy of the strain measurements but also provides additional valuable data to develop a specific concrete expansion coefficient for hardfill, which is a relatively new construction material.

5. The uniform arrangement of temperature instruments provides a valuable data that demonstrates better picture for the temperature distributions and orientations. Therefore, the number of temperature sensors must be utilized in sufficient quantities especially for such a new developing dam type.

6. Parshall flumes can be superior to V-notch weirs due to low maintenance requirements especially for the instrumentation systems equipped with automatic data acquisition.

7. A proper arrangement of strong motion accelerometers is quite important to obtain meaningful data, which represent dam behaviors regarding seismic incidents.

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APPENDIX A

A convenient record form is required for manual data acquisition task. Technicians performing the measurements are suggested to fill out such forms to obtain proper data collection. A typical record form is shown in Table A.1.

Table A. 1. Typical Record Form for Manual Data Acquisition (SIS, 2008)

VIBRATING WIRE STRAIN GAUGE RECORD FORM

Estensimetro a corda vibrante *Vibrating wire strain gauge*	S/N:	Sigla di identificazione: *Identification code:*
Cantiere: *Site:*	Ubicazione: *Location:*	Data installazione: *Installation date:*

DATI STRUMENTO/ *INSTRUMENT FEATURES*

Modello: *Type:*	Campo di misura: *Range:*
Fattore di conversione (GF): *Gauge factor (GF):*	Unità misurata: *Measured unit:*
Valore iniziale di temperatura (T_0): *Initial value of temperature (T_0):*	
Lettura iniziale prima dell'installazione: *Initial reading before installation:*	Lettura di zero dopo l'installazione (L_0): *Initial reading after installation (L_0):*

UNITA' DI LETTURA/ *READOUT UNIT*

Modello: *Type:*	Numero di matricola: *Serial number:*

DATA *DATE*	**LETTURA** *READING Le*	**MICROSTRAIN** *Le-L_0*	**TEMPERATURA TEMPERATURE**	**NOTE** *NOTES*
		[$\Delta\mu\varepsilon$]	[°C]	

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SCHEDA RILEVAMENTO DATI ***FIELD DATA SHEET***	Rev. n. 1	Maggio 2000 *May 2000*	Pagina n: ..../.... *Page n: ..../....*

Estensimetri corda vibrante e barrette estensimetriche
Vibrating wire strain gauges and resistive strain gauges SISGEO

APPENDIX B

A complete price list of all instruments are given in RUP currency unit in Table B.1 and prices considered for the installation cost calculations are given in Table B.2. The abbreviations used in cost calculation tables are also listed in Table B.1.

Table B. 1. Prices of Instruments

	Instrument	Abbreviations	Price (RUP)
1	Pressure Cell	**PC**	**1.200**
2	Piezometer	**PM**	**925**
3	Strain Gauge	**SG**	**280**
4	Joint meter	**JM**	**860**
5	No-Stress Strain Gauge	**NS**	**350**
6	Resistance Temperature Detector	**RTD**	**105**
7	Reservoir Level Gauges	**RLG**	**1.100**
8	V-Notch Weirs	**VN**	**1.205**
9	V-Notch Weirs (Automatic)	**VN-AMU**	**2.250**
10	Parshall Flumes	**PF**	**990**
11	Parshall Flumes (Automatic)	**PF-AMU**	**2.205**
12	Regular Accelerometer	**ACC**	**10.230**
13	Submersible Accelerometer		**20.700**
14	Common Triggering Unit	**Cmmn Trggr**	**21.625**
15	Accelerometer Digital Recorder	**ACC DigREC**	**17.000**
16	PVC Pipe 20mm (1m)	**PVC20**	**1**
17	PVC Pipe 200mm (1m)	**PVC200**	**80**
18	Regular Cable (1m)	**RC(*)**	**4,2**
19	Multi-core Cable (1m)	**MC(*)**	**7**
20	Junction Box	**JNCBX**	**98**
21	PVC Cable Tray (3m)	**CT**	**60**
22	Switch Terminal Box 18 POS.	**SWBX-18**	**2.325**
23	Switch Terminal Box 24 POS.	**SWBX-24**	**2.930**
24	Multiplexer 32 Channels	**MUX**	**3.785**
25	Data Logger (Automatic)	**ADAS-Datalogger**	**12.900**
26	Multi-logger Software	**Multi-logger Software**	**13.950**
27	Portable Readout Unit-VW	**Prtble Reeadout VW**	**5.115**
28	Portable Readout Unit-ER	**Prtble Reeadout ER**	**3.140**
29	Staff Gauge for VN and PF	**STG for VN and PF**	**150**
30	Rosette Mount for SG	**RM**	**185**
31	Fixing Plate for JM	**FP**	**330**

* In cost calculation tables, “C-SF” is cable length calculated by considering a safety factor (1.20) for both regular and multi-core type cables.

Table B. 2. Cost of Installation

Labour Cost (LC)	man-hour	**15**
Travel Expense(TE)	price/person	**40**
Per Diem (PD)	price/person	**20**
Accommodation (A)	price/person	**40**
Drilling Cost (DC)	price/meter	**50**

APPENDIX C

All cost calculations performed in the design phase are presented in tabular form for current instrumentation system equipped with ADAS. The cost tables include all the details regarding instrument, cabling and installation costs.

Table C. 1. Installation Plan and Cost Table for the Current System with ADAS

Stage	Elevation m	Instrument ID	Instrument Quantity	Number of Technicians	Work Hrs	LC	TE	A	PD	DC	Total Cost (RUP)
1	160-165	PC	15	2	30	450	80	240	160	0	930
2	170,5	PM	15	2	30	450	80	240	160	18.724	20.130
		ACC	1	2	2	30				446	
3	185,0	SG	8	2	16	240	80	160	120	0	600
4	195,5	PM	2	2	4	60	80	160	120	525	945
5	205,0	SG	6	2	12	180	80	160	120	0	540
6	225,0	SG	5	2	10	150	80	160	120	0	540
		ACC	1	2	2	30					
7	245,0	SG	4	2	8	120	80	160	120	0	480
8	272,0	JM	16	2	32	480	80	320	200	0	1.230
		VN	2	2	4	60				0	
		ACC	1	2	2	30				0	
		RLG	2	2	4	60				0	
9	272,0	ADAS	40	2	80	1.200	80	640	360	0	2.700
		SP	14	2	28	420				0	
Total Installation Cost (RUP)											28.095

Table C. 2. Cabling and Instrument Cost of Pressure Cells

Instrument ID	Elevation m	Length (m)					Cabling Cost (RUP)				Instrument Cost (RUP)
		PVC20	PVC200	CT	RC	C-SF	PVC20	PVC200	CT	RC	
PC1	161,0	36,2			256,8	308,1	36,2			1.294	1.200
PC2	164,0	41,5			256,6	307,86	41,5			1.293	1.200
PC3	164,0	74,1			289,2	346,98	74,1			1.457	1.200
PC4	164,0	111,8			326,9	392,28	111,8			1.648	1.200
PC5	164,0	149,6			364,7	437,58	149,6			1.838	1.200
PC6	161,0	19,1			234,2	281,04	19,1			1.180	1.200
PC7	164,0	24,4			239,5	287,4	24,4			1.207	1.200
PC8	164,0	57,0	5,5	215,1	272,1	326,52	57,0	440	4.302	1.371	1.200
PC9	164,0	94,8			309,9	371,82	94,8			1.562	1.200
PC10	164,0	132,5			347,6	417,12	132,5			1.752	1.200
PC11	161,0	41,7			256,8	308,1	41,7			1.294	1.200
PC12	164,0	47,0			262,1	314,46	47,0			1.321	1.200
PC13	164,0	79,6			294,7	353,58	79,6			1.485	1.200
PC14	164,0	117,3			332,4	398,88	117,3			1.675	1.200
PC15	164,0	155,1			370,2	444,18	155,1			1.866	1.200
Σ		**1181,3**	**5,5**	**215,1**	**4413,3**	**5295,9**	**1181,3**	**440**	**4.302**	**22.243**	**18.000**

Table C. 3. Cabling and Instrument Cost of Piezometers

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			Instrument Cost (RUP)
		PVC20	CT	RC	C-SF	PVC20	CT	RC	
PM1	140,0	0,0		283,1	339,72	0		1.427	925
PM2	140,0	0,0		284,6	341,52	0		1.434	925
PM3	120,0	0,0		304,3	365,16	0		1.534	925
PM4	120,0	0,0		301,9	362,28	0		1.522	925
PM5	160,0	0,0		226,3	271,56	0		1.141	925
PM6	130,0	0,0		258,2	309,84	0		1.301	925
PM7	130,0	0,0		256,2	307,44	0		1.291	925
PM8	100,0	0,0		290,1	348,12	0		1.462	925
PM9	100,0	0,0	505,7	286,7	344,04	0	10.114	1.445	925
PM10	162,0	56,1		276,7	332,04	56		1.395	925
PM11	162,0	129,5		350,1	420,12	130		1.765	925
PM12	140,0	0,0		286,1	343,32	0		1.442	925
PM13	140,0	0,0		284,6	341,52	0		1.434	925
PM14	120,0	0,0		307,3	368,76	0		1.549	925
PM15	120,0	0,0		304,9	365,88	0		1.537	925
PM16	190,5	5,0		273,2	327,84	5		1.377	925
PM17	190,5	5,0		272,2	326,64	5		1.372	925
Σ		**195,6**	**505,7**	**4846,5**	**5815,8**	**196**	**10.114**	**24.426**	**15.725**

Table C. 4. Cabling and Instrument Cost of Strain Gauges

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX	RM	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC	(RUP)	(RUP)	
SG1 - 3D	245,0	37,4	102,5	139,9	167,9	37	2.050	1.175	98	185	840
SG5 - 3D	245,0	24,0		126,5	151,8	24		1.063	98	185	840
SG9 - 2D	245,0	20,7		123,2	147,8	21		1.035	98	185	560
SG20 - 3D	245,0	45,0		147,5	177,0	45		1.239	98	185	840
SG2 - 3D	225,0	46,3		128,8	154,6	46		1.082	98	185	840
SG6 - 3D	225,0	23,5		106,0	127,2	24		890	98	185	840
SG10 - 2D	225,0	18,0		100,5	120,6	18		844	98	185	560
SG11 - 2D	225,0	22,8		105,3	126,4	23		885	98	185	560
SG21 - 3D	225,0	57,0		139,5	167,4	57		1.172	98	185	840
SG3 - 3D	205,0	67,9	169,9	199,4	239,3	68	3.398	1.675	98	185	840
SG7 - 3D	205,0	23,9		155,4	186,5	24		1.305	98	185	840
SG12 - 2D	205,0	1,0		132,5	159,0	1		1.113	98	185	560
SG13 - 2D	205,0	23,9		155,4	186,5	24		1.305	98	185	560
SG14 - 2D	205,0	48,1		179,6	215,5	48		1.509	98	185	560
SG22 - 3D	205,0	46,4		177,9	213,5	46		1.494	98	185	840
SG4 - 3D	185,0	70,7		240,6	288,7	71		2.021	98	185	840
SG8 - 3D	185,0	21,7		191,6	229,9	22		1.609	98	185	840
SG15 - 2D	185,0	1,0		170,9	205,1	1		1.436	98	185	560
SG16 - 2D	185,0	20,6		190,5	228,6	21		1.600	98	185	560
SG17 - 2D	185,0	41,2		211,1	253,3	41		1.773	98	185	560
SG18 - 2D	185,0	66,7		236,6	283,9	67		1.987	98	185	560
SG19 - 2D	185,0	92,2		262,1	314,5	92		2.202	98	185	560
SG23 - 3D	185,0	49,2		219,1	262,9	49		1.840	98	185	840
Σ		**869,2**	**272,4**	**3839,9**	**4607,9**	**869**	**5.448**	**32.255**	**2.254**	**4.255**	**16.240**

Table C. 5. Cabling and Instrument Cost of Joint meters

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX	FP	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC	(RUP)	(RUP)	
JM1 - 2D	247,2	4,0	239,7	162,4	194,9	4	4.794	1.364	98	330	1.720
JM2 - 2D	247,2	4,0		143,1	171,7	4		1.202	98	330	1.720
JM3 - 2D	247,2	4,0		123,9	148,7	4		1.041	98	330	1.720
JM4 - 2D	247,2	4,0		114,2	137,0	4		959	98	330	1.720
JM5 - 2D	247,2	4,0		137,5	165,0	4		1.155	98	330	1.720
JM6 - 2D	247,2	4,0		188,5	226,2	4		1.583	98	330	1.720
JM7 - 2D	222,2	0,0	204,8	125,0	150,0	0	4.096	1.050	98	330	1.720
JM8 - 2D	222,2	0,0		104,6	125,5	0		879	98	330	1.720
JM9 - 2D	222,2	0,0		112,9	135,5	0		948	98	330	1.720
JM10 - 2D	222,2	0,0		137,0	164,4	0		1.151	98	330	1.720
JM11 - 2D	222,2	0,0		158,3	190,0	0		1.330	98	330	1.720
JM12 - 2D	222,2	0,0		173,3	208,0	0		1.456	98	330	1.720
JM13 - 2D	195,5	0,0	205,8	176,1	211,3	0	4.116	1.479	98	330	1.720
JM14 - 2D	195,5	0,0		189,3	227,2	0		1.590	98	330	1.720
JM15 - 2D	170,5	0,0	230,6	200,9	241,1	0	4.612	1.688	98	330	1.720
JM16 - 2D	170,5	0,0		202,3	242,8	0		1.699	98	330	1.720
Σ		**24,0**	**880,9**	**2449,3**	**2939,2**	**24**	**17.618**	**20.574**	**1.568**	**5.280**	**27.520**

Table C. 6. Cabling and Instrument Cost of Reservoir Level Gauges

Instrument ID	Elevation m	Length (m)		Cabling Cost (RUP)	Instrument Cost (RUP)
		RC	C-SF	RC	
RLG1	255,0	158,7	190,4	800	1.100
RLG2	243,5	174,8	209,8	881	1.100
Σ		**333,5**	**400,2**	**1.681**	**2.200**

Table C. 7. Cabling and Instrument Cost of V-Notch Weirs

Instrument ID	Elevation m	Length (m)		Cabling Cost (RUP)	Instrument Cost (RUP)
		RC	C-SF	RC	
VN1	220,5	140,7	168,8	709	2.250
VN2	195,5	165,6	198,7	835	2.250
Σ		**306,3**	**367,6**	**1.544**	**4.500**

Table C. 8. Instrument Cost of Strong Motion Accelerometers

Instrument ID	Elevation m	Cmmn Trggr (RUP)	ACC DigiREC (RUP)	Instrument Cost (RUP)
ACC1	272,0	21.625	17.000	10.230
ACC2	220,5		17.000	10.230
ACC3	162,0		17.000	20.700
Σ		**21.625**	**51.000**	**41.160**

Table C. 9. Data Acquisition System Cost

Data Acquisition System Cost			
Instrument ID	**Quantity (unit)**	**Instrument Price (RUP)**	**Total Cost (RUP)**
SWBX-18	7	2.325	16.275
SWBX-24	1	2.930	2.930
MUX	8	3.785	30.280
ADAS-Datalogger	2	12.900	25.800
Multi-logger Software	1	13.950	13.950
Prtble Readout VW	1	5.115	5.115
Prtble Readout ER	1	3.140	3.140
STG for VN and PF	2	150	300
Σ			**97.790**

Table C. 10. Drilling Cost of the Current Systems

Drilling Cost for Current Systems				
Elevation m	**Instrument ID**	**Drilling Length m**	**DL-SF(*) m**	**Total Cost (RUP)**
140	**PM1**	0,0	0,0	2.821
120	**PM3**	53,7	56,4	
140	**PM2**	0,0	0,0	2.692
120	**PM4**	51,3	53,8	
160	**PM5**	0,0	0,0	3.758
130	**PM7**	0,0	0,0	
100	**PM9**	71,6	75,2	
130	**PM6**	0,0	0,0	3.939
100	**PM8**	75,0	78,8	
140	**PM12**	0,0	0,0	2.821
120	**PM14**	53,7	56,4	
140	**PM13**	0,0	0,0	2.692
120	**PM15**	51,3	53,8	
Subtotal				**18.724**
190,5	**PM16**	5,0	5,3	263
190,5	**PM17**	5,0	5,3	263
Subtotal				**525**
162	**ACC3**	8,5	8,9	**446**
Total Drilling Cost (RUP)				**19.695**

* DL-SF is the drilling length calculated by considering a safety factor of 1.05.

APPENDIX D

All cost calculations performed in the design phase are presented in tabular form for current instrumentation system equipped with MDAS. The cost tables include all the details regarding instrument, cabling and installation costs.

Table D. 1. Installation Plan and Cost Table for the Current System with MDAS

Stage	Elevation m	Instrument ID	Instrument Quantity	Number of Technicians	Work Hrs	LC	TE	A	PD	DC	Total Cost (RUP)
1	160-165	PC	15	2	30	450	80	240	160	0	930
2	170,5	PM	15	2	30	450	80	240	160	18.724	20.130
		ACC	1	2	2	30				446	
3	185,0	SG	8	2	16	240	80	160	120	0	600
4	195,5	PM	2	2	4	60	80	160	120	525	945
5	205,0	SG	6	2	12	180	80	160	120	0	540
6	225,0	SG	5	2	10	150	80	160	120	0	540
		ACC	1	2	2	30					
7	245,0	SG	4	2	8	120	80	160	120	0	480
8	272,0	JM	16	2	32	480	80	320	200	0	1.230
		VN	2	2	4	60				0	
		ACC	1	2	2	30				0	
		RLG	2	2	4	60				0	
9	272,0	MDAS	16	2	32	480	80	400	240	0	1.620
		SP	14	2	28	420					
Total Installation Cost (RUP)											27.015

Table D. 2. Cabling and Instrument Cost of Pressure Cells

Instrument ID	Elevation m	Length (m)					Cabling Cost (RUP)				Instrument Cost (RUP)
		PVC20	PVC200	CT	RC	C-SF	PVC20	PVC200	CT	RC	
PC1	161,0	36,2			171,7	205,98	36			865	1.200
PC2	164,0	41,5			171,5	205,74	41			864	1.200
PC3	164,0	74,1			204,1	244,86	74			1.028	1.200
PC4	164,0	111,8			241,8	290,16	112			1.219	1.200
PC5	164,0	149,6			279,6	335,46	150			1.409	1.200
PC6	161,0	19,1			149,1	178,92	19			751	1.200
PC7	164,0	24,4			154,4	185,28	24			778	1.200
PC8	164,0	57,0	5,5	130,0	187,0	224,4	57	440	2.600	942	1.200
PC9	164,0	94,8			224,8	269,7	95			1.133	1.200
PC10	164,0	132,5			262,5	315	133			1.323	1.200
PC11	161,0	41,7			171,7	205,98	42			865	1.200
PC12	164,0	47,0			177,0	212,34	47			892	1.200
PC13	164,0	79,6			209,6	251,46	80			1.056	1.200
PC14	164,0	117,3			247,3	296,76	117			1.246	1.200
PC15	164,0	155,1			285,1	342,06	155			1.437	1.200
Σ		**1181,3**	**5,5**	**130,0**	**3136,8**	**3764,1**	**1.181**	**440**	**2.600**	**15.809**	**18.000**

Table D. 3.Cabling and Instrument Cost of Piezometers

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			Instrument Cost (RUP)
		PVC20	CT	RC	C-SF	PVC20	CT	RC	
PM1	140,0	0,0		198,0	237,6	0		998	925
PM2	140,0	0,0		199,5	239,4	0		1.005	925
PM3	120,0	0,0		219,2	263,04	0		1.105	925
PM4	120,0	0,0		216,8	260,16	0		1.093	925
PM5	160,0	0,0		141,2	169,44	0		712	925
PM6	130,0	0,0		173,1	207,72	0		872	925
PM7	130,0	0,0		171,1	205,32	0		862	925
PM8	100,0	0,0		205,0	246	0		1.033	925
PM9	100,0	0,0	420,6	201,6	241,92	0	8.412	1.016	925
PM10	162,0	56,1		191,6	229,92	56		966	925
PM11	162,0	129,5		265,0	318	130		1.336	925
PM12	140,0	0,0		201,0	241,2	0		1.013	925
PM13	140,0	0,0		199,5	239,4	0		1.005	925
PM14	120,0	0,0		222,2	266,64	0		1.120	925
PM15	120,0	0,0		219,8	263,76	0		1.108	925
PM16	190,5	5,0		187,9	225,48	5		947	925
PM17	190,5	5,0		186,9	224,28	5		942	925
Σ		**195,6**	**420,6**	**3399,4**	**4079,28**	**196**	**8.412**	**17.133**	**15.725**

Table D. 4. Cabling and Instrument Cost of Strain Gauges

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX (RUP)	RM (RUP)	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC			
SG1 - 3D	245,0	37,4	44,5	81,9	98,3	37	890	688	98	185	840
SG5 - 3D	245,0	24,0		68,5	82,2	24		575	98	185	840
SG9 - 2D	245,0	20,7		65,2	78,2	21		548	98	185	560
SG20 - 3D	245,0	45,0		89,5	107,4	45		752	98	185	840
SG2 - 3D	225,0	46,3		70,8	85,0	46		595	98	185	840
SG6 - 3D	225,0	23,5		48,0	57,6	24		403	98	185	840
SG10 - 2D	225,0	18,0		42,5	51,0	18		357	98	185	560
SG11 - 2D	225,0	22,8		47,3	56,8	23		397	98	185	560
SG21 - 3D	225,0	57,0		81,5	97,8	57		685	98	185	840
SG3 - 3D	205,0	67,9	84,6	199,4	239,3	68	1.692	1.675	98	185	840
SG7 - 3D	205,0	23,9		155,4	186,5	24		1.305	98	185	840
SG12 - 2D	205,0	1,0		132,5	159,0	1		1.113	98	185	560
SG13 - 2D	205,0	23,9		155,4	186,5	24		1.305	98	185	560
SG14 - 2D	205,0	48,1		179,6	215,5	48		1.509	98	185	560
SG22 - 3D	205,0	46,4		177,9	213,5	46		1.494	98	185	840
SG4 - 3D	185,0	70,7		155,3	186,4	71		1.305	98	185	840
SG8 - 3D	185,0	21,7		106,3	127,6	22		893	98	185	840
SG15 - 2D	185,0	1,0		85,6	102,7	1		719	98	185	560
SG16 - 2D	185,0	20,6		105,2	126,2	21		884	98	185	560
SG17 - 2D	185,0	41,2		125,8	151,0	41		1.057	98	185	560
SG18 - 2D	185,0	66,7		151,3	181,6	67		1.271	98	185	560
SG19 - 2D	185,0	92,2		176,8	212,2	92		1.485	98	185	560
SG23 - 3D	185,0	49,2		133,8	160,6	49		1.124	98	185	840
Σ		**869,2**	**129,1**	**2635,5**	**3162,6**	**869**	**2.582**	**22.138**	**2.254**	**4.255**	**16.240**

Table D. 5. Cabling and Instrument Cost of Joint Meters

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX	FP	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC	(RUP)	(RUP)	
JM1 - 2D	247,2	4,0	177,7	100,4	120,5	4	3.554	843	98	330	1.720
JM2 - 2D	247,2	4,0		81,1	97,3	4		681	98	330	1.720
JM3 - 2D	247,2	4,0		61,9	74,3	4		520	98	330	1.720
JM4 - 2D	247,2	4,0		52,2	62,6	4		438	98	330	1.720
JM5 - 2D	247,2	4,0		75,5	90,6	4		634	98	330	1.720
JM6 - 2D	247,2	4,0		126,5	151,8	4		1.063	98	330	1.720
JM7 - 2D	222,2	0,0	122,5	42,7	51,2	0	2.450	359	98	330	1.720
JM8 - 2D	222,2	0,0		22,3	26,8	0		187	98	330	1.720
JM9 - 2D	222,2	0,0		30,6	36,7	0		257	98	330	1.720
JM10 - 2D	222,2	0,0		54,7	65,6	0		459	98	330	1.720
JM11 - 2D	222,2	0,0		76,0	91,2	0		638	98	330	1.720
JM12 - 2D	222,2	0,0		91,0	109,2	0		764	98	330	1.720
JM13 - 2D	195,5	0,0	120,3	90,6	108,7	0	2.406	761	98	330	1.720
JM14 - 2D	195,5	0,0		103,8	124,6	0		872	98	330	1.720
JM15 - 2D	170,5	0,0	145,3	115,6	138,7	0	2.906	971	98	330	1.720
JM16 - 2D	170,5	0,0		117,0	140,4	0		983	98	330	1.720
Σ		**24,0**	**565,8**	**1241,9**	**1490,3**	**24**	**11.316**	**10.432**	**1.568**	**5.280**	**27.520**

Table D. 6. Cabling and Instrument Cost of Reservoir Level Gauges

Instrument ID	Elevation m	Length (m)		Cabling Cost (RUP)	Instrument Cost (RUP)
		RC	C-SF	RC	
RLG1	255,0	98,7	118,4	497	1.100
RLG2	243,5	114,8	137,8	579	1.100
Σ		**213,5**	**256,2**	**1.076**	**2.200**

Table D. 7. Cabling and Instrument Cost of V-Notch Weirs

Instrument ID	Elevation m	Instrument Cost (RUP)
VN1	220,5	1.205
VN2	195,5	1.205
Σ		**2.410**

Table D. 8. Instrument Cost of Strong Motion Accelerometers

Instrument ID	Elevation m	Cmmn Trggr (RUP)	ACC DigiREC (RUP)	Instrument Cost (RUP)
ACC1	272,0	21.625	17.000	10.230
ACC2	220,5		17.000	10.230
ACC3	162,0		17.000	20.700
Σ		**21.625**	**51.000**	**41.160**

Table D. 9. Data Acquisition System Cost

Data Acquisition System Cost			
Instrument ID	**Quantity (unit)**	**Instrument Price (RUP)**	**Total Cost (RUP)**
SWBX-18	7	2.325	16.275
SWBX-24	1	2.930	2.930
Prtble Readout VW	1	5.115	5.115
Prtble Readout ER	1	3.140	3.140
STG for VN and PF	2	150	300
Σ	12		**27.760**

APPENDIX E

All cost calculations performed in the design phase are presented in tabular form for alternative instrumentation system equipped with ADAS. The cost tables include all the details regarding instrument, cabling and installation costs.

Table E. 1. Installation Plan and Cost Table for the Alternative System with ADAS

Stage	Elevation M	Instrument ID	Instrument Quantity	Number of Technicians	Work Hrs	LC	TE	A	PD	DC	Total Cost (RUP)
1	160-165	PC	15	2	30	450	80	240	160	0	930
2	170,5	PM	15	2	30	450	80	240	160	18724	20.130
		ACC	1	2	2	30				446	
3	185,0	SG	8	2	16	240	80	240	160	0	870
		NS	2	2	4	60					
		TH	3	2	6	90					
4	195,5	PM	2	2	4	60	80	160	120	525	945
5	205,0	SG	6	2	12	180	80	240	160	0	780
		NS	2	2	4	60					
		TH	2	2	4	60					
6	225,0	SG	5	2	10	150	80	160	120	0	600
		NS	1	2	2	30				0	
		TH	2	2	4	60				0	
7	245,0	SG	4	2	8	120	80	160	120	0	540
		NS	1	2	2	30					
		TH	1	2	2	30					
8	272,0	JM	16	2	32	480	80	320	200	0	2.573
		PF	2	2	4	60				0	
		ACC	2	2	4	60				1312,5	
		RLG	2	2	4	60				0	
9	272,0	ADAS	40	2	80	1200	80	640	360	0	2.700
		SPs	14	2	28	420				0	
Total Installation Cost (RUP)											30.068

Table E. 2. Cabling and Instrument Cost of Pressure Cells

Instrument ID	Elevation m	Length (m)					Cabling Cost (RUP)				Instrument Cost (RUP)
		PVC20	PVC200	CT	RC	C-SF	PVC20	PVC200	CT	RC	
PC1	161,0	36,2	5,5	215,1	256,8	308,1	36	440	4.302	1.294	1.200
PC2	164,0	41,5			256,6	307,86	41			1.293	1.200
PC3	164,0	74,1			289,2	346,98	74			1.457	1.200
PC4	164,0	111,8			326,9	392,28	112			1.648	1.200
PC5	164,0	149,6			364,7	437,58	150			1.838	1.200
PC6	161,0	19,1			234,2	281,04	19			1.180	1.200
PC7	164,0	24,4			239,5	287,4	24			1.207	1.200
PC8	164,0	57,0			272,1	326,52	57			1.371	1.200
PC9	164,0	94,8			309,9	371,82	95			1.562	1.200
PC10	164,0	132,5			347,6	417,12	133			1.752	1.200
PC11	161,0	41,7			256,8	308,1	42			1.294	1.200
PC12	164,0	47,0			262,1	314,46	47			1.321	1.200
PC13	164,0	79,6			294,7	353,58	80			1.485	1.200
PC14	164,0	117,3			332,4	398,88	117			1.675	1.200
PC15	164,0	155,1			370,2	444,18	155			1.866	1.200
Σ		**1181,3**	**5,5**	**215,1**	**4413,3**	**5295,9**	**1.181**	**440**	**4.302**	**22.243**	**18.000**

Table E. 3. Cabling and Instrument Cost of Piezometers

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			Instrument Cost (RUP)
		PVC20	CT	RC	C-SF	PVC20	CT	RC	
PM1	140,0	0,0	505,7	283,1	339,72	0	10.114	1.427	925
PM2	140,0	0,0		284,6	341,52	0		1.434	925
PM3	120,0	0,0		304,3	365,16	0		1.534	925
PM4	120,0	0,0		301,9	362,28	0		1.522	925
PM5	160,0	0,0		226,3	271,56	0		1.141	925
PM6	130,0	0,0		258,2	309,84	0		1.301	925
PM7	130,0	0,0		256,2	307,44	0		1.291	925
PM8	100,0	0,0		290,1	348,12	0		1.462	925
PM9	100,0	0,0		286,7	344,04	0		1.445	925
PM10	162,0	56,1		276,7	332,04	56		1.395	925
PM11	162,0	129,5		350,1	420,12	130		1.765	925
PM12	140,0	0,0		286,1	343,32	0		1.442	925
PM13	140,0	0,0		284,6	341,52	0		1.434	925
PM14	120,0	0,0		307,3	368,76	0		1.549	925
PM15	120,0	0,0		304,9	365,88	0		1.537	925
PM16	190,5	5,0		273,2	327,84	5		1.377	925
PM17	190,5	5,0		272,2	326,64	5		1.372	925
Σ		**195,6**	**505,7**	**4846,5**	**5815,8**	**196**	**10.114**	**24.426**	**15.725**

Table E. 4. Cabling and Instrument Cost of Strain Gauges

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX	RM	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC	(RUP)	(RUP)	
SG1 - 3D	245,0	37,4	102,5	139,9	167,9	37	2.050	1.175	98	185	840
SG5 - 3D	245,0	24,0		126,5	151,8	24		1.063	98	185	840
SG9 - 2D	245,0	20,7		123,2	147,8	21		1.035	98	185	560
SG20 - 3D	245,0	45,0		147,5	177,0	45		1.239	98	185	840
SG2 - 3D	225,0	46,3		128,8	154,6	46		1.082	98	185	840
SG6 - 3D	225,0	23,5		106,0	127,2	24		890	98	185	840
SG10 - 2D	225,0	18,0		100,5	120,6	18		844	98	185	560
SG11 - 2D	225,0	22,8		105,3	126,4	23		885	98	185	560
SG21 - 3D	225,0	57,0		139,5	167,4	57		1.172	98	185	840
SG3 - 3D	205,0	67,9	169,9	199,4	239,3	68	3.398	1.675	98	185	840
SG7 - 3D	205,0	23,9		155,4	186,5	24		1.305	98	185	840
SG12 - 2D	205,0	1,0		132,5	159,0	1		1.113	98	185	560
SG13 - 2D	205,0	23,9		155,4	186,5	24		1.305	98	185	560
SG14 - 2D	205,0	48,1		179,6	215,5	48		1.509	98	185	560
SG22 - 3D	205,0	46,4		177,9	213,5	46		1.494	98	185	840
SG4 - 3D	185,0	70,7		240,6	288,7	71		2.021	98	185	840
SG8 - 3D	185,0	21,7		191,6	229,9	22		1.609	98	185	840
SG15 - 2D	185,0	1,0		170,9	205,1	1		1.436	98	185	560
SG16 - 2D	185,0	20,6		190,5	228,6	21		1.600	98	185	560
SG17 - 2D	185,0	41,2		211,1	253,3	41		1.773	98	185	560
SG18 - 2D	185,0	66,7		236,6	283,9	67		1.987	98	185	560
SG19 - 2D	185,0	92,2		262,1	314,5	92		2.202	98	185	560
SG23 - 3D	185,0	49,2		219,1	262,9	49		1.840	98	185	840
Σ		**869,2**	**272,4**	**3839,9**	**4607,9**	**869**	**5.448**	**32.255**	**2.254**	**4.255**	**16.240**

Table E. 5.Cabling and Instrument Cost of Joint Meters

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX (RUP)	FP (RUP)	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC			
JM1 - 2D	247,2	4,0	239,7	162,4	194,9	4,0	4794,0	1.364	98	330	1.720
JM2 - 2D	247,2	4,0		143,1	171,7	4,0		1.202	98	330	1.720
JM3 - 2D	247,2	4,0		123,9	148,7	4,0		1.041	98	330	1.720
JM4 - 2D	247,2	4,0		114,2	137,0	4,0		959	98	330	1.720
JM5 - 2D	247,2	4,0		137,5	165,0	4,0		1.155	98	330	1.720
JM6 - 2D	247,2	4,0		188,5	226,2	4,0		1.583	98	330	1.720
JM7 - 2D	222,2	0,0	204,8	125,0	150,0	0,0	4096,0	1.050	98	330	1.720
JM8 - 2D	222,2	0,0		104,6	125,5	0,0		879	98	330	1.720
JM9 - 2D	222,2	0,0		112,9	135,5	0,0		948	98	330	1.720
JM10 - 2D	222,2	0,0		137,0	164,4	0,0		1.151	98	330	1.720
JM11 - 2D	222,2	0,0		158,3	190,0	0,0		1.330	98	330	1.720
JM12 - 2D	222,2	0,0		173,3	208,0	0,0		1.456	98	330	1.720
JM13 - 2D	195,5	0,0	205,8	176,1	211,3	0,0	4116,0	1.479	98	330	1.720
JM14 - 2D	195,5	0,0		189,3	227,2	0,0		1.590	98	330	1.720
JM15 - 2D	170,5	0,0	230,6	200,9	241,1	0,0	4612,0	1.688	98	330	1.720
JM16 - 2D	170,5	0,0		202,3	242,8	0,0		1.699	98	330	1.720
Σ		**24,0**	**880,9**	**2449,3**	**2939,2**	**24,0**	**17618,0**	**20.574**	**1.568**	**5.280**	**27.520**

Table E. 6. Cabling and Instrument Cost of No-stress Strain Gauges

Instrument ID	Elevation m	Length (m)			Cabling Cost (RUP)		Instrument Cost (RUP)
		PVC20	RC	C-SF	PVC20	RC	
NS1	245,0	30,7	133,2	159,8	31	671	350
NS2	225,0	29,0	111,5	133,8	29	562	350
NS3	205,0	11,9	143,4	172,1	12	723	350
NS4	205,0	60,1	191,6	229,9	60	966	350
NS5	185,0	10,0	179,9	215,9	10	907	350
NS6	185,0	54,2	224,1	268,9	54	1.129	350
Σ		**195,9**	**983,7**	**1180,4**	**196**	**4.958**	**2.100**

Table E. 7. Cabling and Instrument Cost of RTDs

Instrument ID	Elevation m	Length (m)			Cabling Cost (RUP)		Instrument Cost (RUP)
		PVC20	RC	C-SF	PVC20	RC	
RTD1	245,0	20,0	122,5	147,0	20	617	105
RTD2	225,0	28,0	110,5	132,6	28	557	105
RTD3	225,0	32,8	115,3	138,4	33	581	105
RTD4	205,0	13,0	144,5	173,4	13	728	105
RTD5	205,0	35,9	167,4	200,9	36	844	105
RTD6	185,0	17,5	187,4	224,9	18	944	105
RTD7	185,0	30,6	200,5	240,6	31	1.011	105
RTD8	185,0	79,2	249,1	298,9	79	1.255	105
Σ		**257,0**	**1297,2**	**1556,6**	**257**	**6.538**	**840**

Table E. 8. Cabling and Instrument Cost of Reservoir Level Gauges

Instrument ID	Elevation m	Length (m)		Cabling Cost (RUP)	Instrument Cost (RUP)
		RC	C-SF	RC	
RLG1	255,0	158,7	190,4	800	1.100
RLG2	243,5	174,8	209,8	881	1.100
Σ		**333,5**	**400,2**	**1.681**	**2.200**

Table E. 9. Cabling and Instrument Cost of Parshall Flumes

Instrument ID	Elevation m	Length (m)		Cabling Cost (RUP)	Instrument Cost (RUP)
		RC	C-SF	RC	
PF1	220,5	140,7	168,8	709	2.205
PF2	195,5	165,6	198,7	835	2.205
Σ		**306,3**	**367,6**	**1.544**	**4.410**

Table E. 10. Instrument Cost of Strong Motion Accelerometers

Instrument ID	Elevation m	Cmmn Trggr (RUP)	ACC DigiREC (RUP)	Instrument Cost (RUP)
ACC1	272,0	21.625	17.000	10.230
ACC2	Suitable Elevation		17.000	20.700
ACC3	162,0		17.000	20.700
Σ		**21.625**	**51.000**	**51.630**

Table E. 11. Drilling Cost for Alternative Systems

Drilling Cost for Alternative Systems				
Elevation m	**Instrument ID**	**Drilling Length m**	**DL-SF(*) m**	**Total Cost (RUP)**
140	**PM1**	0,0	0,0	2.821
120	**PM3**	53,7	56,4	
140	**PM2**	0,0	0,0	2.692
120	**PM4**	51,3	53,8	
160	**PM5**	0,0	0,0	3.758
130	**PM7**	0,0	0,0	
100	**PM9**	71,6	75,2	
130	**PM6**	0,0	0,0	3.939
100	**PM8**	75,0	78,8	
140	**PM12**	0,0	0,0	2.821
120	**PM14**	53,7	56,4	
140	**PM13**	0,0	0,0	2.692
120	**PM15**	51,3	53,8	
Subtotal				**18.724**
190,5	**PM16**	5,0	5,3	263
190,5	**PM17**	5,0	5,3	263
Subtotal				**525**
162	**ACC3**	8,5	8,9	**446**
Suitable Elevation	**ACC2**	25	26,3	**1.313**
Total Drilling Cost (RUP)				**21.007**

* DL-SF is the drilling length calculated by considering a safety factor of 1.05.

Table E. 12. Cost of Data Acquisition System

Data Acquisition System			
Instrument ID	**Quantity (unit)**	**Instrument Price (RUP)**	**Total Cost (RUP)**
SWBX-18	7	2.325	16.275
SWBX-24	1	2.930	2.930
MUX	9	3.785	34.065
ADAS-Datalogger	2	12.900	25.800
Multi-logger Software	1	13.950	13.950
Prtble Readout VW	1	5.115	5.115
Prtble Readout ER	1	3.140	3.140
STG for VN and PF	2	150	300
Σ			**101.575**

APPENDIX F

All cost calculations performed in the design phase are presented in tabular form for alternative instrumentation system equipped with MDAS. The cost tables include all the details regarding instrument, cabling and installation costs.

Table F. 1. Installation Plan and Cost for the Alternative System with MDAS

Stage	Elevation m	Instrument ID	Instrument Quantity	Number of Technicians	Work Hrs	LC	TE	A	PD	DC	Total Cost (RUP)
1	160-165	PC	15	2	30	450	80	240	160	0	930
2	170,5	PM	15	2	30	450	80	240	160	18724	20.130
		ACC	1	2	2	30				446	
3	185,0	SG	8	2	16	240	80	240	160	0	870
		NS	2	2	4	60					
		TH	3	2	6	90					
4	195,5	PM	2	2	4	60	80	160	120	525	945
5	205,0	SG	6	2	12	180	80	240	160	0	780
		NS	2	2	4	60					
		TH	2	2	4	60					
6	225,0	SG	5	2	10	150	80	160	120	0	600
		NS	1	2	2	30				0	
		TH	2	2	4	60				0	
7	245,0	SG	4	2	8	120	80	160	120	0	540
		NS	1	2	2	30					
		TH	1	2	2	30					
8	272,0	JM	16	2	32	480	80	320	200	0	2.573
		PF	2	2	4	60				0	
		ACC	2	2	4	60				1312,5	
		RLG	2	2	4	60				0	
9	272,0	MDAS	16	2	32	480	80	400	240	0	1.620
		SPs	14	2	28	420				0	
Total Installation Cost (RUP)											28.988

Table F. 2. Cabling and Instrument Cost of Pressure Cells

Instrument ID	Elevation m	Length (m)					Cabling Cost (RUP)				Instrument Cost (RUP)
		PVC20	PVC200	CT	RC	C-SF	PVC20	PVC200	CT	RC	
PC1	161,0	36,2	5,5	130,0	171,7	205,98	36,2	440	2.600	865	1.200
PC2	164,0	41,5			171,5	205,74	41,5			864	1.200
PC3	164,0	74,1			204,1	244,86	74,1			1.028	1.200
PC4	164,0	111,8			241,8	290,16	111,8			1.219	1.200
PC5	164,0	149,6			279,6	335,46	149,6			1.409	1.200
PC6	161,0	19,1			149,1	178,92	19,1			751	1.200
PC7	164,0	24,4			154,4	185,28	24,4			778	1.200
PC8	164,0	57,0			187,0	224,4	57,0			942	1.200
PC9	164,0	94,8			224,8	269,7	94,8			1.133	1.200
PC10	164,0	132,5			262,5	315	132,5			1.323	1.200
PC11	161,0	41,7			171,7	205,98	41,7			865	1.200
PC12	164,0	47,0			177,0	212,34	47,0			892	1.200
PC13	164,0	79,6			209,6	251,46	79,6			1.056	1.200
PC14	164,0	117,3			247,3	296,76	117,3			1.246	1.200
PC15	164,0	155,1			285,1	342,06	155,1			1.437	1.200
Σ		**1181,3**	**5,5**	**130,0**	**3136,8**	**3764,1**	**1181,3**	**440**	**2.600**	**15.809**	**18.000**

Table F. 3. Cabling and Instrument Cost of Piezometers

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			Instrument Cost (RUP)
		PVC20	CT	RC	C-SF	PVC20	CT	RC	
PM1	140,0	0,0		198,0	237,6	0		998	925
PM2	140,0	0,0		199,5	239,4	0		1.005	925
PM3	120,0	0,0		219,2	263,04	0		1.105	925
PM4	120,0	0,0		216,8	260,16	0		1.093	925
PM5	160,0	0,0		141,2	169,44	0		712	925
PM6	130,0	0,0		173,1	207,72	0		872	925
PM7	130,0	0,0		171,1	205,32	0		862	925
PM8	100,0	0,0		205,0	246	0		1.033	925
PM9	100,0	0,0	420,6	201,6	241,92	0	8.412	1.016	925
PM10	162,0	56,1		191,6	229,92	56		966	925
PM11	162,0	129,5		265,0	318	130		1.336	925
PM12	140,0	0,0		201,0	241,2	0		1.013	925
PM13	140,0	0,0		199,5	239,4	0		1.005	925
PM14	120,0	0,0		222,2	266,64	0		1.120	925
PM15	120,0	0,0		219,8	263,76	0		1.108	925
PM16	190,5	5,0		187,9	225,48	5		947	925
PM17	190,5	5,0		186,9	224,28	5		942	925
Σ		**195,6**	**420,6**	**3399,4**	**4079,28**	**196**	**8.412**	**17.133**	**15.725**

Table F. 4. Cabling and Instrument Cost of Strain Gauges

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX	RM	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC	(RUP)	(RUP)	
SG1 - 3D	245,0	37,4	44,5	81,9	98,3	37	890	688	98	185	840
SG5 - 3D	245,0	24,0		68,5	82,2	24		575	98	185	840
SG9 - 2D	245,0	20,7		65,2	78,2	21		548	98	185	560
SG20 - 3D	245,0	45,0		89,5	107,4	45		752	98	185	840
SG2 - 3D	225,0	46,3		70,8	85,0	46		595	98	185	840
SG6 - 3D	225,0	23,5		48,0	57,6	24		403	98	185	840
SG10 - 2D	225,0	18,0		42,5	51,0	18		357	98	185	560
SG11 - 2D	225,0	22,8		47,3	56,8	23		397	98	185	560
SG21 - 3D	225,0	57,0		81,5	97,8	57		685	98	185	840
SG3 - 3D	205,0	67,9	84,6	199,4	239,3	68	1.692	1.675	98	185	840
SG7 - 3D	205,0	23,9		155,4	186,5	24		1.305	98	185	840
SG12 - 2D	205,0	1,0		132,5	159,0	1		1.113	98	185	560
SG13 - 2D	205,0	23,9		155,4	186,5	24		1.305	98	185	560
SG14 - 2D	205,0	48,1		179,6	215,5	48		1.509	98	185	560
SG22 - 3D	205,0	46,4		177,9	213,5	46		1.494	98	185	840
SG4 - 3D	185,0	70,7		155,3	186,4	71		1.305	98	185	840
SG8 - 3D	185,0	21,7		106,3	127,6	22		893	98	185	840
SG15 - 2D	185,0	1,0		85,6	102,7	1		719	98	185	560
SG16 - 2D	185,0	20,6		105,2	126,2	21		884	98	185	560
SG17 - 2D	185,0	41,2		125,8	151,0	41		1.057	98	185	560
SG18 - 2D	185,0	66,7		151,3	181,6	67		1.271	98	185	560
SG19 - 2D	185,0	92,2		176,8	212,2	92		1.485	98	185	560
SG23 - 3D	185,0	49,2		133,8	160,6	49		1.124	98	185	840
Σ		**869,2**	**129,1**	**2635,5**	**3162,6**	**869**	**2.582**	**22.138**	**2.254**	**4.255**	**16.240**

Table F. 5. Cabling and Instrument Cost of Joint Meters

Instrument ID	Elevation m	Length (m)				Cabling Cost (RUP)			JNCBX	FP	Instrument Cost (RUP)
		PVC20	CT	MC	C-SF	PVC20	CT	MC	(RUP)	(RUP)	
JM1 - 2D	247,2	4,0	177,7	100,4	120,5	4	3.554	843	98	330	1.720
JM2 - 2D	247,2	4,0		81,1	97,3	4		681	98	330	1.720
JM3 - 2D	247,2	4,0		61,9	74,3	4		520	98	330	1.720
JM4 - 2D	247,2	4,0		52,2	62,6	4		438	98	330	1.720
JM5 - 2D	247,2	4,0		75,5	90,6	4		634	98	330	1.720
JM6 - 2D	247,2	4,0		126,5	151,8	4		1.063	98	330	1.720
JM7 - 2D	222,2	0,0	122,5	42,7	51,2	0	2.450	359	98	330	1.720
JM8 - 2D	222,2	0,0		22,3	26,8	0		187	98	330	1.720
JM9 - 2D	222,2	0,0		30,6	36,7	0		257	98	330	1.720
JM10 - 2D	222,2	0,0		54,7	65,6	0		459	98	330	1.720
JM11 - 2D	222,2	0,0		76,0	91,2	0		638	98	330	1.720
JM12 - 2D	222,2	0,0		91,0	109,2	0		764	98	330	1.720
JM13 - 2D	195,5	0,0	120,3	90,6	108,7	0	2.406	761	98	330	1.720
JM14 - 2D	195,5	0,0		103,8	124,6	0		872	98	330	1.720
JM15 - 2D	170,5	0,0	145,3	115,6	138,7	0	2.906	971	98	330	1.720
JM16 - 2D	170,5	0,0		117,0	140,4	0		983	98	330	1.720
Σ		**24,0**	**565,8**	**1241,9**	**1490,3**	**24**	**11.316**	**10.432**	**1.568**	**5.280**	**27.520**

Table F. 6. Cabling and Instrument Cost of No-Stress Strain Gauges

Instrument ID	Elevation m	Length (m)			Cabling Cost (RUP)		Instrument Cost (RUP)
		PVC20	RC	C-SF	PVC20	RC	
NS1	245,0	30,7	75,2	90,2	31	379	350
NS2	225,0	11,0	55,5	66,6	11	280	350
NS3	205,0	12,0	96,6	115,9	12	487	350
NS4	205,0	60,1	144,7	173,6	60	729	350
NS5	185,0	11,0	95,6	114,7	11	482	350
NS6	185,0	54,2	138,8	166,6	54	700	350
Σ		**179,0**	**606,4**	**727,7**	**179**	**3.056**	**2.100**

Table F. 7. Cabling and Instrument Cost of RTDs

Instrument ID	Elevation m	Length (m)			Cabling Cost (RUP)		Instrument Cost (RUP)
		PVC 20	RC	C-SF	PVC20	RC	
RTD1	245,0	20,6	65,1	78	21	328	105
RTD2	225,0	28,0	72,5	87	28	365	105
RTD3	225,0	32,8	77,3	93	33	390	105
RTD4	205,0	13,0	97,6	117	13	492	105
RTD5	205,0	35,9	120,5	145	36	607	105
RTD6	185,0	17,5	102,1	123	18	515	105
RTD7	185,0	30,6	115,2	138	31	581	105
RTD8	185,0	79,2	163,8	197	79	826	105
Σ		**257,6**	**814,1**	**977**	**258**	**4.103**	**840**

Table F. 8. Cabling and Instrument Cost of Reservoir Level Gauges

Instrument ID	Elevation m	Length (m)		Cabling Cost (RUP)	Instrument Cost (RUP)
		RC	C-SF	RC	
RLG1	255,0	98,7	118,4	497	1.100
RLG2	243,5	114,8	137,8	579	1.100
Σ		**213,5**	**256,2**	**1.076**	**2.200**

Table F. 9. Cabling and Instrument Cost of Parshall Flumes

Instrument ID	Elevation m	Instrument Cost (RUP)
PF1	220,5	990
PF2	195,5	990
Σ		**1.980**

Table F. 10. Instrument Cost of Strong Motion Accelerometers

Instrument ID	Elevation m	Cmmn Trggr (RUP)	ACC DigiREC (RUP)	Instrument Cost (RUP)
ACC1	272,0	21.625	17.000	10.230
ACC2	Suitable Elevation		17.000	20.700
ACC3	162,0		17.000	20.700
Σ		**21.625**	**51.000**	**51.630**

Table F. 11. Cost of Data Acquisition Systems

Data Acquisition System			
Instrument ID	**Quantity (unit)**	**Instrument Price (RUP)**	**Total Cost (RUP)**
SWBX-18	7	2.325	16.275
SWBX-24	1	2.930	2.930
Prtble Readout VW	1	5.115	5.115
Prtble Readout ER	1	3.140	3.140
STG for VN and PF	2	150	300
Σ	12		**27.760**

APPENDIX G

Operating cost calculations are only carried out for MDAS systems and presented in Table G.1. The reader is suggested to refer Section 5.5.6 for detailed information.

Table G. 1. Operating Cost Calculations for Manual Data Acquisition Systems

Operating Cost for Manual Data Acquisition Systems							Total Cost of MDAS Systems	
Year	Number of	Training Cost	Salary Per Month	Increase of Salary	Total	Operating Cost	Current System	Alternative System
	Technicians	per tech	per tech	per year (avg=5%)	Training Cost	per year	MDAS	MDAS
1	2	1000	650	683	2000	17600	379135	401891
2	2	0	683	717	0	16380	395515	418271
3	2	0	717	752	0	17199	412714	435470
4	2	0	752	790	0	18059	430773	453529
5	2	0	790	830	0	18962	449735	472491
6	2	0	830	871	0	19910	469645	492401
7	2	0	871	915	0	20905	**490550**	513306
8	2	0	915	960	0	21951	512501	**535257**
9	2	0	960	1008	0	23048	535549	558305
10	2	0	1008	1059	0	24201	559750	582506
11	2	0	1059	1112	0	25411	585161	607917
12	2	0	1112	1167	0	26681	611842	634598
13	2	0	1167	1226	0	28015	639858	662614
14	2	0	1226	1287	0	29416	669274	692030
15	2	0	1287	1351	0	30887	700161	722917
16	2	0	1351	1419	0	32431	732592	755348
17	2	0	1419	1490	0	34053	766645	789401
18	2	0	1490	1564	0	35755	802400	825156
19	2	0	1564	1643	0	37543	839943	862699
20	2	0	1643	1725	0	39420	879364	902120