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Geotechnical Instrumentation of Embankment Dams

الأجهزة الجيوتقنية لرصد السدود

Submitted for the partial fulfillment of the MSc in Civil Engineering

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Dedication

To my parents, my family, and all those I learnt from.
To all those who have given me a helping hand, a sincere advice or an encouraging word.

المستخلص

تهدف هذه الأطروحة للتعريف بأهمية أجهزة الرصد والقياس باعتبارها الأداة التي يستخدمها مهندسو سلامة السدود لتوفير المعلومات المطلوبة لتقييم أداء السدود و للإنذار المبكر في حالة حدوث أي تغيرات قد تهدد سلامتها. كما تتطرق باختصار لمتطلبات التخطيط لأنظمة الرصد والقياس والتي تشمل تحديد أهداف النظام وتطبيق نهج التخطيط والذي يتضمن، من بين خطوات أخرى، تحديد العوامل التي ينبغي رصدها واختيار الجهاز أو الأداة المناسبة لعملية الرصد و تسجيل البيانات. كما تم التطرق ببعض الإيجاز لبعض المفاهيم الجيوتقنية الأساسية المتعلقة بهندسة السدود والتي يجب على مهندسي الرصد والقياس المعرفة الكافية بها حتى يتسنى لهم تحليل البيانات التي تم جمعها عن طريق نظم الرصد والقياس بشكل صحيح. ونسبة لضرورة الحفاظ على سلامة السدود على الدوام فقد تم توضيح دور أجهزة الرصد والقياس خلال مرحلة التصميم, مرحلة التشييد, مرحلة الملء الأول ومرحلة التشغيل مع بيان أهم القراءات التي يتم رصدها في المرحلة الأخيرة. وقد تم تقديم وصف تفصيلي لأنواع أجهزة الرصد والقياس المستخدمة في كلٍ من مشروع تعلية سد الروصيرص و مشروع مجمع سدي أعالي عطبرة لإعطاء فكرة عن كيفية تركيب هذه الأجهزة وكيفية أخذ القراءات والقياسات وكيفية عرض النتائج بالإضافة لنظرة عامة عن نظام الرصد الآلي بسد الروصيرص. كما تم استخدام بيانات الرصد والقياس من قطاع نموذجي من كلا السدين بالإضافة لبيانات خلايا الضغط بسد الروصيرص و قراءات التسرب بسدي أعالي عطبرة لتوضيح كيفية تحليل هذه البيانات من أجل تقييم أداء الردميات الترابية بكلا السدين. وقد اختتمت هذه الأطروحة ببعض التوصيات.

Abstract

The thesis aims at highlighting the importance of geotechnical instrumentation as the tool of dam safety engineers which provides the information necessary to evaluate the performance of embankment dams and gives early warnings of changes that could endanger their safety. It also gives a brief of the requirements of planning an embankment instrumentation system which include defining the objectives of the system and applying the systematic planning approach which contains, among other steps, defining the parameters to be monitored and the selection of the appropriate device or instrument for monitoring and recording data. Data collected form monitoring systems should be properly analyzed by instrumentation engineers which necessitates familiarity with the basic geotechnical concepts related to dam design and engineering. The thesis gives a brief of some these concepts. It is emphasized that the safety of the dam should be maintained throughout its deferent life phases i.e. design phase; construction phase; first filling phase; and operation phase, with some detail of the types of readings to be taken during the last one. A detailed description of the types of instruments used for the monitoring of *Roseires Dam Heightening Project (RDHP)* and *Dam Complex of Upper Atbara Project (DCUAP)* is presented to give an idea of how instruments are installed, how readings and measurements are taken and how results are presented as well as an overview of the automatic monitoring system of *(RDHP)*. Instrumentation readings of two example sections from the two projects together with readings of the pressure cells in *(RDHP)* and seepage observations in *(DCUAP)* are used to illustrate how instrumentation data are analyzed to come to an evaluation of the safety of the embankments in terms of the different monitored parameters. The thesis is concluded by some recommendations from the author which he believes to be of high importance to be considered.

Table of Contents

LIST OF FIGURES

LIST OF TABLES

Chapter One: INTRODUCTION

1. INTRODUCTION

1.1. General

The purpose of instrumentation and monitoring is to maintain and improve dam safety by providing information to evaluate whether a dam is performing as expected, and warn of changes that could endanger the safety of a dam.

Though only a small percentage of dams develop problems, it is impossible to predict those that will develop problems because of the highly indeterminate nature of the structures and the infinite number of possible variations in conditions that could affect the safety of a dam or appurtenant structures. Therefore, it is prudent that any dam that may affect the public safety has basic instrumentation to monitor vital signs.

Minimum recommended instrumentation is separated into categories of existing and proposed dams and further subdivided depending on the hazard potential classification and the type of structure.

The type of information to be gathered by instrumentation varies with deferent dam life phases i.e. design phase; construction phase; first filling phase; and operation phase.

In the design phase instrumentation is used in investigation works to establish baseline conditions relative to design, construction and potential legal issues. In the construction phase instrumentation is used for confirmation of design assumptions and to monitor changes in groundwater and stability conditions both on site and at adjacent sites. It also monitors workers' safety and construction quality control. In the first reservoir filling phase performance monitoring activities (visual and instrumented monitoring) are most critical because the dam is being tested for the first time in terms of seepage resistance and structural stability. During this phase instrumentation typically is used for better understanding of the response of the dam to reservoir loads and confirmation of satisfactory performance. During operation phase information gained from instrumentation is used to make sure that the dam continues to perform satisfactorily throughout its design life and it helps in creating a data base that can be utilized in improving the field of geotechnical engineering.

1.2. Problem statement

As the number of dams is increasing, and damages associated to their failures are considered to be of a very large scale, it is vital to develop a very good and effective dam safety programs. Although instrumentation is the main tool for providing the information necessary for implementing dam safety programs, most of the engineers who are working in the field of dam engineering have less awareness of its important role.

1.3. Objectives

The main objective of this thesis is to illustrate the types of instrumentation used for monitoring the performance of embankment dams for the purpose of having early alarms for the signs of development of any of the causes of failure.

Moreover, the thesis is expected to enlarge the knowledge of engineers of instrumentation used for the monitoring of embankment dams and raise their awareness of its important role in the assurance of dam safety.

Chapter Two: LITERATURE PREVIEW

2. LITERATURE PREVIEW

2.1. Instrumentation and Dam Safety

Instrumentation necessary to assist in the evaluation of the safety of embankments is considered an integral part of the geotechnical design. The design of geotechnical engineering project contains much uncertainties and assumptions than most other branches of engineering; where designers have greater control over the materials utilized for construction; therefore, field instrumentation is more vital to geotechnical engineering project and geotechnical engineers must have more than casual knowledge of instrumentation.

Since failures of embankment dams are very costly in terms of money, properties, infrastructure and even the lives of inhabitants of downstream areas, embankment dams should be carefully designed and constructed to avoid the principal causes of failures and incidents which are:

1- Overtopping from inadequate spillway capacity, spillway blockage, or excessive settlement resulting in erosion of the embankment.
2- Erosion of embankments from failure of spillways, failure or deformation of outlet conduits causing leakage and piping, and failure of riprap.
3- Embankment leakage and piping along outlet conduits, abutment interfaces, contacts with concrete structures, or concentrated piping in the embankment itself.
4- Foundation leakage and piping in pervious strata, soluble lenses, and rock discontinuities.
5- Sliding of embankment slopes due to overly steep slopes, seepage forces, rapid drawdown, or rainfall.
6- Sliding along clay seams in foundations.
7- Cracking due to differential settlements.
8- Liquefaction.

A very robust and effective dam safety program must be adopted to monitor and evaluate the performance of embankment dams. Monitoring of performance requires the choice of the suitable instrumentation for evaluating the causes of failures. Table 2-1 shows typical instrumentation and monitoring used in evaluating causes of common dam safety problems.

Table 2-1 : Typical instrumentation and monitoring used in evaluating causes of common problems/concerns

PROBLEM/CONCERN	TYPICAL INSTRUMENTATION
Seepage or leakage	Visual observation, weirs, flowmeters, flumes, calibrated containers, observation wells, piezometers
Boils or piping	Visual observation, piezometers.
Uplift pressure, pore pressure, or phreatic surface	Visual observation, observation wells, piezometers
Total or surface movement (translation, rotation)	Visual observation, precise position and level surveys, plumb measurements, tiltmeters
Internal movement or deformation in embankments	Settlement plates, cross-arm devices, fluid leveling devices, pneumatic settlement sensors, vibrating wire settlement sensor, mechanical and electrical sounding devices, inclinometers, extensometers.
Foundation or abutment movement	Visual observation, precise surveys, inclinometers, extensometers, piezometers
Poor quality rock foundation or abutment	Visual observation, pressure and flow measurements, piezometers, precise surveys, extensometers, inclinometers
Slope stability	Visual observation, precise surveys, inclinometers, extensometers, observation wells, piezometers, shear strips.
Seismic loading	Accelerographs.

2.2. General Role of Instrumentation

The main purpose of instrumentation installed within an embankment dam is to study whether or not the dam is behaving according to design predictions. Generally, dams can be subdivided into two categories: category one, Dams with special foundation conditions or uncommon design features; category two, Dams without special foundation conditions or uncommon design features.

2.2.1. Dams of Category One

When instrumentation is used for this purpose, it helps to determine whether design assumptions for the special conditions and features are being realized during construction and operation. The design of the monitoring program is tuned directly for the special conditions and features and, because the designers of the dam know the weaknesses of the particular site and the sensitive features of the design, they play a leading role in the choice of type and location of instruments.

2.2.2. Dams of Category Two

The practice of placing instruments routinely in embankment dams where there is no special problem to be studied, for example, a conventional dam of moderate height on a good foundation, has seen several ups and downs in the last 30 years. In the 1960s, many important dams were constructed with essentially no internal instruments. Each failure or major problem tends to be widely publicized, and dam engineers feel pressure to install instruments routinely for their own protection, even if they do not believe it absolutely necessary.

Most old dams of less than 15 m height have no instruments other than V-notch weirs or other means of measuring leakage. There are very many up to 90 m built during 1960 ~ 1975 without additional instruments.

2.3. Implementation of an Instrumentation Program

An instrumentation program is a comprehensive approach that assures that all aspects of instrumentation from planning and design through maintenance and rehabilitation are commensurate with the overall purpose. An instrumentation program is an important contributing effort to the much larger concept of dam safety. As such, the characteristics of the instrumentation program must be consistent with the other entities of dam safety such as dam safety training, emergency response, periodic inspections, remedial studies, and structure modifications.

Qualified personnel, quality equipment, and timely information and assessment must be encouraged and supported. Without this level of attention and commitment, the importance of all entities of dam safety will deteriorate, wasted effort and expense will result, and areas that depend on input from the instrumentation program will also suffer.

2.4. Planning of instrumentation system

Planning an embankment instrumentation system is a team effort of the designers (or those responsible for evaluating existing projects) and personnel having expertise in the application of geotechnical instrumentation. Developing an instrumentation system should begin with a definition of an objective and proceed through a comprehensive series of logical steps that include all aspects of the system.

2.4.1. Objectives of geotechnical instrumentation

The principal objectives of a geotechnical instrumentation plan may be generally grouped into four categories which are:

2.4.1.1. Analytical assessment

Analysis of data obtained from geotechnical instrumentation may be utilized to verify design parameters, verify design assumptions and construction techniques, analyze adverse events, and verify apparent satisfactory performance.

2.4.1.2. Prediction of future performance

Instrumentation data should be used in such a manner that valid predictions of future behavior of an embankment can be made. Such predictions may vary from indicating continued satisfactory performance under normal operating conditions to an indication of potential future distress which may become threatening to life or safety, and necessitate remedial action.

2.4.1.3. Legal evaluation

Valid instrumentation data can be valuable for potential litigation relative to construction claims. It can also be valuable for evaluation of later claims relative to changed groundwater conditions downstream of a dam or landward of a levee project. In many cases, damage claims arising from adverse events can be of such great monetary value that the cost of providing instrumentation can be justified on this basis alone. Instrumentation data can be utilized as an aid in determining causes or extent of adverse events so that various legal claims can be evaluated.

2.4.1.4. Development and verification of future design theories.

Analysis of the performance of existing dams and levees, and instrumentation data generated during operation, can be used to advance the state-of-the-art of design and construction. Instrumentation data from existing projects can promote safer and more economical design and construction of future earth and rock fill embankments.

Instrumentation achieves these objectives by providing quantitative data to assess groundwater pressures, deformations, total stresses, temperatures, seismic events, leakage, and water levels. Total movements as well as relative movements between zones of an embankment and its foundation may also need to be monitored. A wide variety of instruments may be utilized in a comprehensive monitoring program to ensure that all critical conditions for a given project are covered sufficiently.

2.4.2. Steps of planning an instrumentation system

The systematic planning approach recommended by Dunnicliff (1988 and 1990) requires a series of steps to be gone through in order to plan a good instrumentation system. These steps are listed below:

1- Prediction of mechanisms that control behavior.
2- Definition of purpose of instrumentation.
3- Definition of geotechnical questions.
4- Selection of parameters to monitor.
5- Selection of instrument locations.
6- Selection of instruments.
7- Determination of need for automation.
8- Planning for recording of factors which influence measurements.
9- Establishment of procedures for ensuring data validity.
10- Determination of costs.
11- Planning installation.
12- Planning long-term protection.
13- Planning regular calibration and maintenance.
14- Planning data collection and management.
15- Coordination of resources.
16- Determination of life cycle costs.

2.5. Familiarity with geotechnical concepts

Familiarity of instrumentation engineers with basic geotechnical concepts is necessary to understand the application of geotechnical instrumentation devices and analyses of instrumentation data. These geotechnical aspects are related primarily to pore water pressures and deformations which can be found in detail in geotechnical textbooks. The following are some of the main important geotechnical definitions related to dam safety.

2.5.1. Hydrostatic piezometric conditions.

The groundwater level or table is defined as the elevation of the free water surface in permeable soils or rock because of equilibrium with atmospheric pressure in a hole extending a short distance below the capillary zone, as shown in Figure 2-1(a). The water-bearing stratum containing the groundwater is called an aquifer. The capillary zone is defined as the interval between the free water surface and the limiting height above which water cannot be drawn by capillarity. Normal behavior of groundwater conditions occurs when the pore water pressure increases hydrostatically with depth below the groundwater level. In this condition the pore water pressure, also called the hydrostatic pressure, can be calculated by multiplying the unit weight of water by the vertical distance from the point of interest (screen location, sensor location, etc.) to the groundwater surface.

2.5.2. Pore water pressure.

Figure 2-1 illustrates the groundwater condition soon after a layer of material is placed over existing soil layers and before consolidation is complete. Thus, excess pore water pressure exists in the clay and the groundwater is no longer in equilibrium. The second pipe (b) is perforated throughout its length while the other pipes are perforated only near the bottom. Due to the high permeability of the sand, the excess pore water pressure in the sand is dissipated immediately. Thus, pipe (a) indicates the groundwater level. Pipe (b) indicates the groundwater level because the permeability of the sand is such that the excess pore water pressure from the clay will be dissipated into the sand. Pipes (c) and (d) indicate the pore water pressures in the clay at two locations. More dissipation of the excess pore water pressure has occurred in pipe (c) than in pipe (d) because the drainage path for excess pore water pressure is shorter and the rate of dissipation is greater in pipe (c) than in pipe (d). Pipe (b) in Figure 2-1is an observation well, because there are no subsurface seals that prevent a vertical connection between multiple strata. A piezometer is a measuring device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressure at other elevations. Pipes (a), (c), and (d) are called piezometers because they indicate pore water pressure at one location (they are sealed above and below the perforated locations) and not to the groundwater pressure at other elevations. The piezometric elevation or piezometric level is the elevation to which water will rise in a piezometer.

2.5.2.1. Positive pore water pressures.

Pore water pressure that is above atmospheric pressure is called positive pore water pressure. Pore water pressure can be increased by applying a compressive force to the soil or when a shearing force is applied to a soil that decreases its volume while preventing dissipation of pore water pressure. Excess pore water pressure resulting from any type of stress change can also be called induced pore water pressure.

2.5.2.2. Negative pore water pressures.

Negative pore water pressure occurs when the pore water pressure is less than atmospheric pressure. This condition can occur when a compressive load is removed or when a densely packed soil is sheared and increases in volume.

2.5.3. Non-hydrostatic groundwater conditions.

Pore water pressure does not always increase hydrostatically with depth below the groundwater level. These groundwater conditions include:

a. Perched water table.

Perched water tables are caused when a permeable material overlies a relatively impermeable stratum above the main groundwater level and retains some groundwater. A piezometer placed in a perched water table will indicate an elevated surface as illustrated in Figure 2-1, pipe (e).

b. Artesian pressure.

Artesian pressures are found in strata that are confined between impervious strata and are connected to a water source at a higher elevation. A well drilled to an artesian aquifer having pore water pressure above the ground surface will flow without pumping and is called a free-flowing artesian well. Artesian conditions are shown in Figure 2-1, pipes (c) and (d).

2.5.4. Variations in piezometric levels and pressures.

Piezometric levels and pressures are rarely constant over an extended period of time. Natural forces such as precipitation, evaporation, atmospheric pressure, and seepage may cause wide variations in the groundwater level.

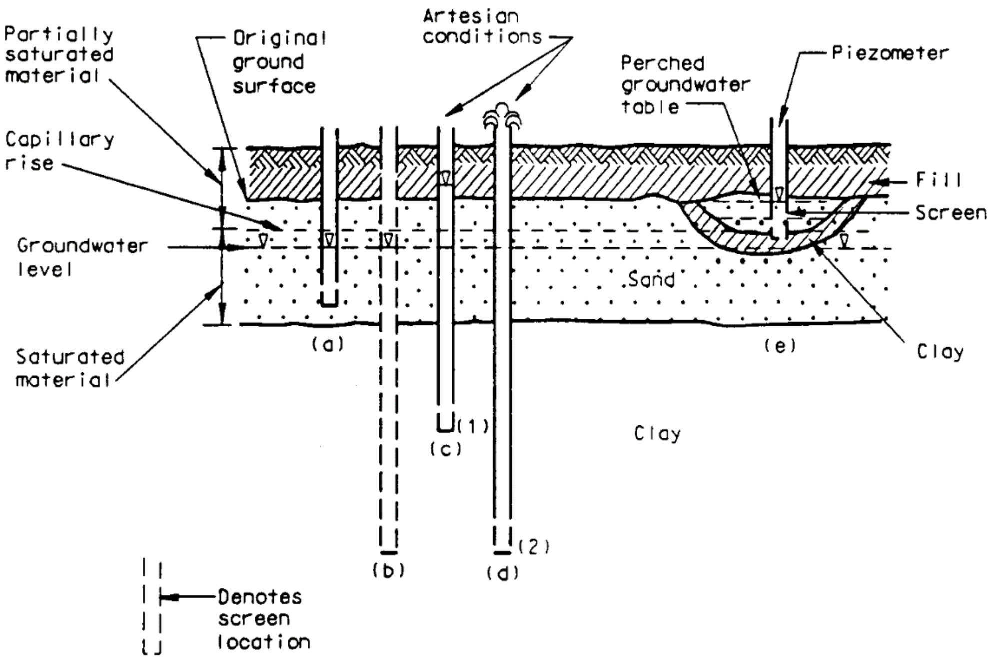


Figure 2-1 : Piezometric conditions in deferent soil strata after layer placing.

2.6. Long-Term Performance Monitoring of Embankment Dams

When the operation phase of the dam starts, most failures are the result of internal erosion, overtopping, or other unexpected events. The need for long-term instrumentation to monitor the performance of the dam against these failures depends greatly on the basic conservatism and foundation conditions of the dam.

2.6.1. Regular Visual Observations

Regular visual observations are an essential aspect of a program for monitoring long-term performance. In fact, visual observations by reservoir staff, trained to look for seeps, boils, shallow sloughing, cracks, or any other signs of distress, to log their observations, and to contact the responsible engineer when appropriate, is the primary approach to monitoring long-term performance of embankment dams. The visual observations should include the spillway and the abutments. If visual observations indicate a potential problem, it may be necessary to initiate a quantitative monitoring program to define the problem and assist in selecting a solution.

2.6.2. Monitoring of Piezometric Pressures

The term piezometer is used to indicate a device that is sealed within the ground so that it responds only to groundwater pressure around itself and not to groundwater pressures at other elevations. Piezometers are used to monitor pore water pressure and joint water pressure.

Applications for piezometers fal1 into two general categories. First, for monitoring the pattern of water flow and second, to provide an index of soil or rock mass strength. Examples in the first category include monitoring subsurface water flow during large-scale pumping tests to determine permeabi1ity in situ, monitoring the long-term seepage pattern in embankment dams and slopes, and monitoring uplift pressures in the dam toe area. In the second category, monitoring of pore or joint water pressure allows an estimate of effective stress to be made and thus an assessment of strength. Examples include assessing the strength along a potential failure plane behind a cut slope in soil or rock, and monitoring of pore water pressure to control staged construction over soft clay foundations.

2.6.2.1. Analysis for Uplift

When selecting the scenarios for analysis of hydrostatic uplift, it must be ensured that the worst-case interactions of the excavation and of the construction grades with the phreatic and piezometric surfaces are selected. Temporal changes in phreatic and piezometric surfaces must be taken into account. The highest temporal phreatic and piezometric surfaces must be used in the analysis. Using average depth of excavation or average elevation for the phreatic and piezometric surfaces is not acceptable (see Figure 2-2).

The purpose of the analysis is to find all areas of the facility, if any, that have a factor of safety less than specified for hydrostatic uplift.

Figure 2-3 illustrates a situation where a clay liner (or another soil layer) is constructed above a saturated layer. The piezometric head (H_P) is applying upward pressure on the liner.

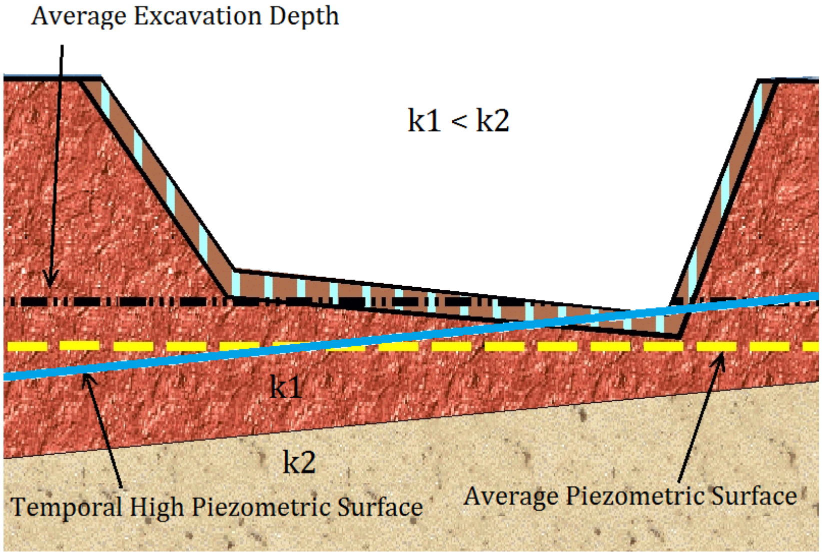


Figure 2-2 : Temporal and Average piezometric surfaces.

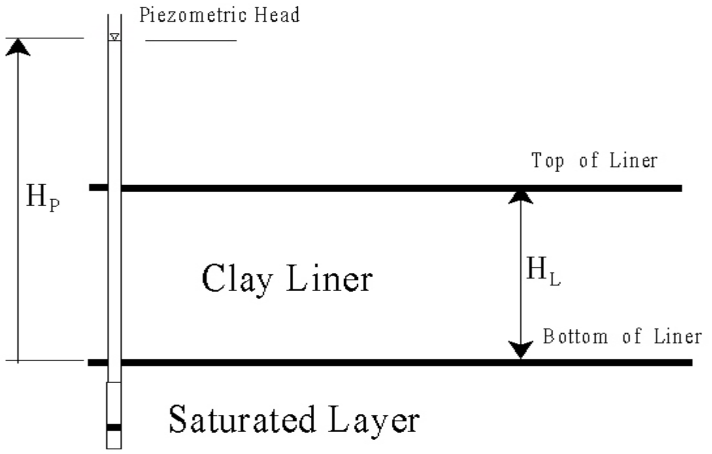


Figure 2-3 : An example of a potential for hydrostatic uplift.

If γ_L = field density of clay liner,

γ_W = density of water,

H_L = clay liner thickness, and

H_P = piezometric level (head),

Then, at some depth (for instance at the interface between the liner and the saturated layer)

$\gamma_L \times H_L$ would represent the total stress (σ), and

$\gamma_W \times H_P$ would represent the pore water pressure (u).

An unstable (or point of failure) situation could then be described as:

$$\sigma = u$$

i.e., $\gamma_L \times H_L = \gamma_W \times H_P$

or as a stress ratio: $\frac{\gamma_L \times H_L}{\gamma_W \times H_P} = 1$

Conversely, the total stress required to achieve the specified of safety is:

$$\gamma_L \times H_L > FS(\gamma_W \times H_P)$$

An unstable condition caused by hydrostatic uplift may develop when the hydrostatic uplift force overcomes the downward force created by the weight of the soil layer(s). If an area acted upon by the hydrostatic force is sufficiently great, excess water pressure may cause overlying soil to rise, creating a failure known as "heave." Although heave can take place in any soil, it will most likely occur at an interface between a relatively impervious layer (such as a clay liner) and a saturated, relatively pervious base.

Water percolation through a soil layer affects hydrostatic uplift force. As a result, considering seepage may theoretically be a more accurate approach. The shear resistance of the soil could also be theoretically taken into account. However, for practical purposes, a conservative evaluation of the resistance created by a soil layer against hydrostatic uplift can be accomplished by calculating a maximum uplift force based on a maximum measured piezometric head and comparing it to the normal stress created by the overlying soil layers. This is especially true when checking an interface between a sub-base and a clay (or plastic) liner, where any significant seepage through the liner material is not anticipated nor wanted.

2.6.3. Monitoring of Deformations

Both horizontal and vertical deformations of the embankment dams and its foundation can be monitored by surveying methods, geotechnical instruments or a combination of both. Surveying methods are used to monitor the magnitude and rate of horizontal and vertical deformations of structures, the ground surface, and accessible parts of subsurface instruments in a wide variety of construction situations.

Monitoring by survey involves the establishing of the dam survey network which comprises a group of leveling pins and mounting plates installed in various levels in sections of interest to monitor their movements relative to a set of fixed reference points located downstream of the dam.

The most common surveying methods used for monitoring the horizontal and vertical movements of embankment dams include:

1- Elevations by Optical leveling.
2- Distance Measurements by Taping.
3- Offsets from a baseline Using Theodolite and Scale.
4- Traverse Lines.
5- Triangulation.
6- Laser Beam Leveling and Offsets.
7- Electronic Distance Measurement (EDM).
8- Trigonometric leveling.
9- Photogrammetric Methods.
10- Global Positioning System (GPS).

If greater accuracy is required or if measuring points are inaccessible to surveying methods, as is the case for subsurface measurements, geotechnical instruments like inclinometers and magnetic settlement plates can be used to measure the internal deformations of embankment layers as well as the foundation. In general, whenever geotechnical instruments are used to monitor deformation, surveying methods are also used to relate measurements to a reference datum.

2.6.4. Monitoring contact pressures in the Embankments/Concrete interfaces.

The interfaces between the embankments and any concrete structure; adjacent or embedded into it; is considered as one of the most critical areas which requires special attention in its design, selection of material and construction. Due to the difference in elasticity, the rate of settlement in the embankment is greater than that of the concrete structure and if the suitable material is not used in that zone, this may lead to horizontal cracking in different elevations within the embankment structure.

During periods of low reservoir levels, the material used in the core of the embankment tends to contract, due shrinkage, forming a gap between the embankment and the concrete surface.

The horizontal cracking, due to differential settlement, together with the gap, due to shrinkage, will provide seepage paths from the reservoir to the downstream face of the embankment, and if the core material is dispersive this may lead to failure due to progressive backward erosion.

Since differential settlement and shrinkage are inevitable, the core should be protected with a carefully-designed filter zone to prevent migration of fine clay particles with the seepage water and if the core material is dispersive it can be chemically stabilized, by the addition of gypsum for example, to reduce its dispersivity.

2.6.5. Monitoring of Seepage

All earth and rock-fill dams are subject to seepage through the embankment, foundation, and abutments. Seepage control is necessary to prevent excessive uplift pressures, sloughing of the downstream slope, piping through the embankment and foundation, and erosion of material by loss into open joints in the foundation and abutments.

Several factors which can be monitored that can lead to a conclusion regarding the safety of a dam are:

1- Progressive increase in the volume of seepage flow.
2- Removal of solids by the seepage.
3- Increased uplift pressures or locally depressed gradients.
4- Soft or wet areas on the downstream embankment.

The most common and easiest monitoring is to rely on visual observations along with careful surface inspections at predetermined intervals. Another type of monitoring which should be completed before construction is the installation of piezometers, observation wells, and drainage collection systems to determine a site dependent pattern of behavior. Finally, the actual structure should be monitored by the installation of a site specific network of piezometers, observation wells and drainage collection systems with flow measurements designed for the anticipated seepage problems. A regular review of the data collected will generally detect major changes between subsequent readings but equally as important are the long-range trends manifested by steady changes.

2.6.6. Monitoring of Seismic Events

Seismic instrumentation is necessary for moderate to high dams in seismic areas. It is also desirable in traditionally non-seismic areas.

2.6.6.1. Strong Motion Monitoring

Strong motion monitoring is used to measure the response of the dam to ground shaking. Although measurements can be used to validate seismic design assumptions, the most important benefit is to guide decisions on inspection and repair after the dam has been subjected to a seismic event. For example, if an earthquake has caused structural damage to the dam or to its appurtenant structures, the following questions must be answered immediately:

1- Was the earthquake larger or smaller than the design earthquake?
2- What will be the performance of the dam in the event of a larger shock?
3- What repair or strengthening is required?

If no obvious earthquake damage has occurred, a decision must be made on the extent of elaborate and expensive inspection operations, and data provided by strong motion monitoring equipment provide vital input to answering questions and reaching decisions. Deformation data provided by static instrumentation are also valuable, because seismic events often cause regional tectonic movements and also deformation within cohesion-less soil deposits. On special occasions the pulses of seismically induced excess pore water pressures can be monitored during ground shaking, requiring the installation of piezometers with high dynamic response.

Instruments for strong motion monitoring are normally called strong motion accelerographs or seismographs.

Strong motion accelerographs should, as a minimum, be located at the base and crest of the dam, and additional instruments may be located on the downstream slope. If possible, an additional instrument should be placed on a rock outcrop as close as possible to the dam to record bedrock motion.

2.6.6.2. Reservoir Seismicity Networks

Seismicity networks are used to measure local small earthquakes in the area of the project site. Measured data provide information on the frequency of local earthquakes, the location and depth of seismic activity, and the magnitude and mechanisms of ground shaking.

Instruments for reservoir seismicity networks are normally called microseismographs. They should be located around the reservoir rim at intervals of from 5 to 30 km. They should, wherever possible, be located on rock in shallow pits away from noise from quarries, streams, and spillways.

2.7. Types of Instrumentation Measuring devices

Most electronic instrumentation measurement methods consist of three components: a transducer, a data acquisition system, and a linkage between these two components. A transducer is a component that converts a physical change into a corresponding electrical output signal. Data acquisition systems range from simple portable readout units to complex automatic systems.

The most common type of transducers used are the Vibrating wire devices which are used in pressure sensors for piezometers, earth pressure cells, and liquid level settlement gages, and in numerous deformation gauges.

In a vibrating wire device, a length of steel wire is clamped at its ends and tensioned so that it is free to vibrate at its natural frequency. The frequency of vibration of the wire varies with the wire tension. Thus with small relative movements between the two end clamps of the vibrating wire device, the frequency of the vibration of the wire varies. The wire can therefore be used as a pressure sensor as shown in Figure 3-1.

Other common types of transducers are, Pneumatic devices, electrical resistance strain gage devices and electrical transducers for measuring linear displacement. For more details, the reader is advised to refer to Dunnicliff (1988 and 1990).

2.8. Automation of monitoring

During the past decades, there have been significant efforts to advance the state of practice in automating dam-safety instrumentation. These efforts were initially targeted towards high hazard dams that posed significant potential risk to downstream communities. The objective is to make automated data acquisition more reliable, cost-effective, and readily available for broader applications in dam safety monitoring through advances in sensor technology, data acquisition equipment, and data management systems.

The main reasons for adopting an Automatic Data Acquisition System (ADAS) are:

1- The challenge of taking regular accurate measurements from big quantity of instruments and measuring points, which may be broadly distributed over long distance and at various levels.
2- The difficulty of dealing with the expected large quantity of collected measurement data and process, analyze and evaluate it to assess the dam performance and to fulfill all dam monitoring requirements and plans.
3- To provide usable and useful data about the performance and behavior of the structures for any future use.
4- To reduce the number of personnel for doing all dam monitoring tasks, compared to the tens of employee that were involved in dam monitoring activities during construction time.

An automated data-acquisition system (or ADAS) can range from a simple data logger temporarily connected to one or more instruments to a permanent system that automates up to several hundred instruments at a dam. Generally, an ADAS for dam-safety monitoring includes the following key components:

- One or more electronic sensors (for water levels, displacements, etc.).
- A remote data logger (permanent or portable).
- A communication link to the dam for remote access (cell phone, landline, radio, or satellite).

An ADAS usually consists of one or more remote monitoring units (RMUs) located on the dam connected to instruments to be automated. The RMUs communicate via radio, hardwire, or cell phone with a central desktop PC with vendor-supplied interface and communication software to provide access to the on-site RMUs by remote users. Typically, the central desktop PC is located onsite; however, it can be located at a remote location (such as a district or administration building). Instruments readings are stored in memory for either manual or automatic downloading for plotting and tabular reporting.

These systems can send out an alarm via cell phone or email if user-defined instrument thresholds are exceeded.

Since these systems are installed outdoors, it is important that equipment used is designed for geotechnical instrumentation and dam-safety monitoring. Special attention should be paid to lightning protection and grounding, surge protection, and backup power supplies.

A properly designed and installed ADAS can provide cost-effective and reliable instrumentation data acquisition and presentation to assist dam safety personnel in both long-term monitoring and during safety events. These systems provide the ability to adjust the frequency of instrument readings and provide the ability to quickly assess trends from remote locations.

Chapter Three: METHODOLOGY

3. METHODOLOGY

Illustration of instrumentation used for the monitoring of embankment dams will be through examples taken from the most resent constructed dams in the country, namely, Roseires Dam Heightening Project (RDHP) and Dam Complex of Upper Atbara Project (DCUAP).

Data taken from these projects will be analyzed either by proprietary software specially designed by instruments manufacturers for specific types of observations; e.g. ***In-Site*** software for the analyses and presentation of inclinometer readings; or by Excel spreadsheets prepared for the purpose of recording, analyzing and presentation of results.

3.1. Description of the two projects

3.1.1. Roseires Dam Heightening Project (RDHP)

Roseires dam is a hydropower dam built in the Blue Nile in two phases. Phase one, which have been completed in the year 1966, consisted of a concrete dam of 1 km length in the river section, an embankment dam of 4.5 km length in the right bank and an embankment dam of 8.5 km length in the left bank. Phase two, which is called Roseires Dam Heightening Project (RDHP), was completed in January 2013 and aimed to heightening the old dam by 10 meters to increase the storage capacity of the reservoir. It also comprised of increasing the lengths of both the right bank and left bank embankment dams to 8.5 kilometers and 12.5 kilometers respectively. The work involved stripping the downstream shell of the old dam to expose the old core so as to connect it to the core of the heightening. It also involved extension of the downstream berms.

Depending on their heights, the embankments in the two banks are divided into four types. Within these types 14 representative sections were selected where almost all types of measurements and readings are taken to monitor the performance of the embankment dam as a whole.

3.1.2. Dam Complex of Upper Atbara Project (DCUAP)

The Dam Complex of Upper Atbara Project is situated on the Atbara River and the Setit River, approximately 20 km upstream of their confluence, 80 km south of Khashm el Girba and approximately 30 km upstream of the small town of Showak in the Gedaref State in East of Sudan.

The DCUAP encompasses the Rumela Project, situated on the Upper Atbara River, and the Burdana Project, situated on the Setit River. Each of these two projects consists in the construction of an earth fill embankment dam and a system of dykes at either side of their river banks, i.e. on left bank of Upper Atbara River, in between the two dams on the Fashaga with a joint interface in dyke construction and on right bank of Setit River. The combined embankments of both Projects with crest level at El. 524.80 m will impound both rivers, Upper Atbara and Setit, and will create the Upper Atbara Reservoir with a total storage volume of 2,763 million m^3 at Maximum Operating Level (MOL=El. 521.00 m) and an active storage volume of 1,580 million m^3 between Minimum Operating Level (LOL=509.00 m) and MOL. Through this impoundment a maximum gross head of 38.85 m is created.

3.2. Long-term monitoring of RDHP and DCUAP

For the purpose of long-term monitoring of RDHP and DCUAP a number of instruments and devices were installed to confirm the safety of the dams in terms of the main parameters shown in 2.6 above. The following is a detailed description of these installations, how readings and measurements are being taken, and how results are calculated and presented.

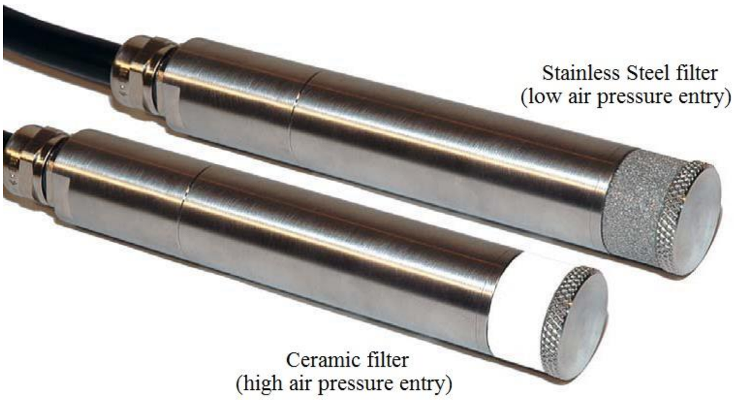
3.2.1. Monitoring of Piezometric Pressures

In both RDHP and DCUAP piezometric water levels and pore water pressures are measured using the most common two types of piezometers, i.e. vibrating wire piezometers and stand pipe piezometers. Vibrating wire piezometers are distributed inside the embankment cross sections to measure piezometric pressures in the deferent embankment cross section components i.e. foundation, core and filters. Stand pipe piezometers are installed alongside the downstream toe drain to monitor the uplift pressures in the downstream side of the dam.

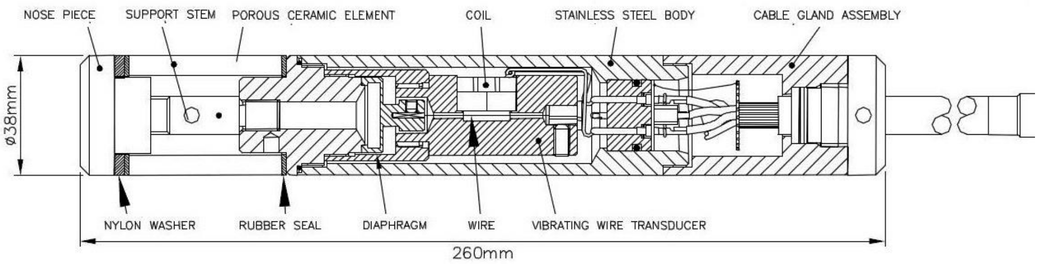
3.2.1.1. Installation

3.2.1.1.1. Vibrating Wire Piezometers

As can be seen in the instrumented sections in both dams shown in Appendix B, Vibrating Wire Piezometers are installed in deferent elevations in the same line to measure piezometric pressures in deferent levels. First, after excavations reached the design depth, a borehole is drilled from the foundation level down to the lowest elevation and sensors are lowered to the design levels and borehole grouted. Then, during the course of construction, sensors are embedded in the embankment whenever the filling reaches a design sensor elevation. The sensors are surrounded with sand filter to prevent clogging of sensor filter by ingress of fine soil particles.



(a) Types of filters



(b) Assembly of the sensor

Figure 3-1 : Vibrating Wire Piezometers.

Table 3-1 : Differences between types of filters.

Stainless steel filter (low air pressure entry)	**Ceramic filter** (high air pressure entry)
Bore diameter: ~50 μm	Bore diameter: ~1 μm
Filter generally used.	Filter usually installed for use in unsaturated fine grain material.
Does not allow suction measurements. If water level drops below the piezometer and that a suction builds up, the filter can de-saturate. But as soon as the water level comes up, it will re-saturate easily.	Allow measuring suction to ~100 kPa. If negative pressure is very high, the filter will de-saturate and readings will become incorrect.
Air entry pressure: ~10 kPa	Air entry pressure: ~450 kPa
Small time lag.	More important time lag.

Easy to saturate and install.	Need to be saturated under vacuum.
Allows fine grain infiltration.	Helps prevent fine grain infiltration.

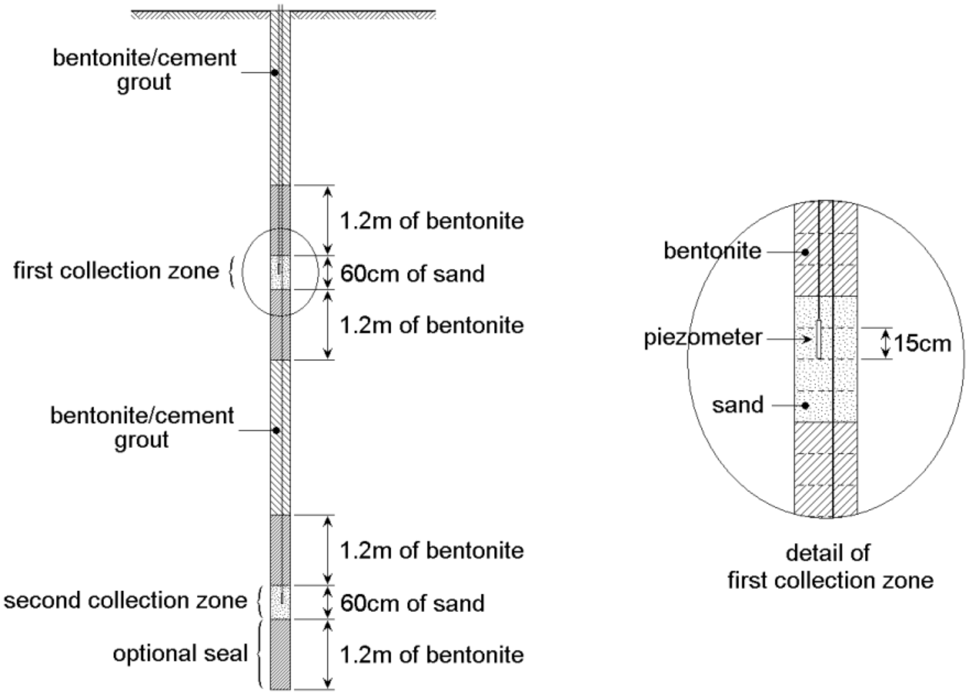


Figure 3-2 : Typical installation of piezometers in a borehole.

3.2.1.2. Readings and Measurements

3.2.1.2.1. Vibrating Wire Piezometers

In RDHP, all Vibrating Wire Piezometers in the main sections are connected to the Automatic Monitoring System (AMS), hence, readings are taken, stored in the data-loggers and sent to the main computer automatically. In DCUAP, the (AMS) is not finalized yet, so, readings are taken manually using a portable VW readout unit, Figure 3-3.

When taking manual readings, the alligator clips of the reading cable is connected to the gage lead wires and the readout unit is switched on. The readout unit will excite the vibrating wire of the sensor and the vibrations of the sensor will induce an electric current in the coil. The readout unit records either the LINEAR reading (LU) or the vibration FREQUENCY (F).

The linear reading is converted to pressure values by the following equation:

$$\boldsymbol{P = C_f(L - L_0)} \qquad \text{……..……………………………} \quad 1$$

where: P = pressure in kilopascal

C_f = calibration factor (provided in the calibration sheet)

L = current reading in linear units (LU)

L_0 = initial reading in linear units (LU)

If the frequency is measured, it can be converted into linear units using the following equation:

$$\boldsymbol{L = K\frac{F^2}{1000}} \quad \text{……..……………………………} \quad 2$$

where: L = reading in linear units

K = gage constant for piezometer

F = frequency in Hz

Material used in the vibrating wire sensors are specially chosen to minimize the temperature effects on the measurements. The thermal coefficient of expansion of the sensor body is very close to the wire's one, so that the temperature effects are self-compensated. However, a slight temperature coefficient still exists. If maximum accuracy is desired or if huge temperature variations are suspected, a correction can be applied.

In any case, especially for low range sensor, variations of barometric pressure have to be corrected as well.

The following equation can be used to correct for both temperature and barometric pressure variations:

$$\boldsymbol{P_c = P - C_T(T - T_0) - (S - S_0)} \quad \text{……..……………………………} \quad 3$$

where: P_c = corrected pressure in kilopascal

P = pressure previously calculated in kilopascal

C_T = thermal coefficient (see calibration sheet), in kPa/°C

T = current temperature reading in degrees Celsius

T_0 = initial temperature reading in degrees Celsius

S = current barometric pressure reading in kilopascal

S_0 = initial barometric pressure reading in kilopascal

Finally, the corrected pressure in kilopascal is converted to a pressure head of water in meters and added to the sensor tip elevation to get the piezometric water level.

(a) (b)

Figure 3-3 : (a) Portable VW recorder (b) Hand Held VW readout unit.

3.2.1.2.2. Stand pipe Piezometers

Measurements of Stand pipe piezometers in both projects are taken using a water level indicator, Figure 3-4, to measure the depth of water level inside the open pipe and deduct it from the elevation of the top of pipe to get the Piezometric water level.

Figure 3-4 : Water Level Indicators.

3.2.1.3. Presentation of Results

Both piezometric water levels and reservoir levels are plotted against dates of readings to monitor the response of pore water pressure in the deferent parts of the dam section to changes in upstream water level as shown in Figure 4-2 for the contentious monitoring of the changes in piezometric pressures, and Figure 4-3 for comparison of piezometric pressures in successive periods of impounding.

3.2.2. Monitoring of Deformations

In RDHP, both horizontal and vertical deformations are measured using a number of inclinometer tubes with settlement targets embedded around them. In DCUAP only vertical deformations are measured using a system of settlement targets embedded around plain tubes installed the same way as in RDHP.

3.2.2.1. Installation

The tubes are installed in boreholes drilled from the foundation level down to the design depth and then extended with the course of filling. The base plate (reference) is installed around the bottom of the tube in the stable foundation and the other magnet targets (Spider magnets) are installed whenever the filling reaches a prescribed elevation.

(a)

(b)

(c)

Figure 3-5 : (a) Inclinometer tube. (b) Embedding of Magnetic Plate. (c) Spider Magnets.

3.2.2.2. Measurements and calculations of Vertical Deformations

For measuring the vertical deformations, the magnetic probe extensometer (Figure 3-6) is dropped to the bottom of the casing, the sensor is raised slowly till the buzzer indicates the location of the first spider magnet. To make sure the accurate depth, the sensor is lowered a little down until the buzzer stops and then raised again much slower until the buzzer sounds again. The step is repeated to record the depths of all the magnetic targets.

The distances between the base plate and all other magnets are calculated and subtracted from the distances at the zero reading to give the settlements of all the targets at the measurement date.

Figure 3-6 : Magnetic probe extensometer.

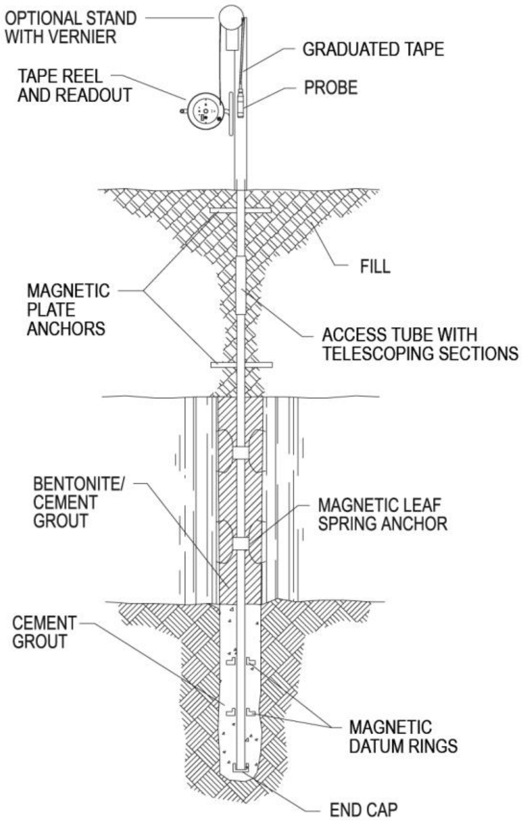


Figure 3-7 : Settlement targets.

3.2.2.3. Presentation of Vertical Deformations

An excel spreadsheet is made to calculate the vertical deformations, as stated in 3.2.2.2 above, and plot the elevations of the magnets versus their deformations for every measurement date to monitor the rate of change in deformations during construction and during reservoir filling in the operation stage. Figure 4-5 shows an example of the Excel model for the analysis and presentation of vertical deformations in main monitoring sections at DCUAP.

3.2.2.4. Measurements and calculations of Horizontal Deformations

For measuring the horizontal deformations, the inclinometer probe (Figure 3-8) is carefully lowered to the bottom of the casing and raised until the next 0.5 m mark rests in the seating cap. The casing has two pairs of alignment keyways in which the probe guide wheels run. The accelerometers measure the angular difference between the probe’s axis and the vertical X and Y planes. The angles are converted to horizontal displacement in millimeters, over the probe gauge length of 500mm.

Displacement readings are taken at regular intervals of (0.5m) within the casing; this is measured and controlled by graduation markers on the cable. An initial or ‘base’ set of inclinometer readings are obtained at each increment within the casing. Summation of each incremental reading provides a profile of horizontal displacement of the casing as a function of depth. Subsequent readings are taken at identical depths. Comparison of successive casing profiles indicates the depth, direction, magnitude and the rate of change of movement. The clearest indication of movement is given by plotting the change in displacement of the casing against depth.

Readings are taken by ***I-n_Port*** software installed in a PDA through a Bluetooth connection with the cable reel and stored in the PDA’s memory and then downloaded through ***In-Site*** software to the calculation PC.

Figure 3-8 : Inclinometer probe with PDA.

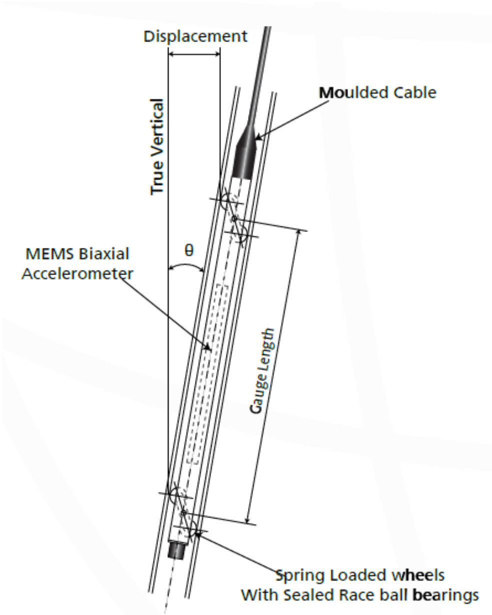


Figure 3-9 : Displacement between two ends of inclinometer probe.

3.2.2.5. Presentation of Horizontal Deformations

The horizontal deformations are either presented using the ***In-Site*** Inclinometer Data Presentation Software or by using Excel spreadsheet to calculate the horizontal deviations from measurements and present the results graphically.

In either cases, deviations in the upstream-downstream and west-east directions, A and B directions respectively, are plotted separately in two adjacent charts. Figure 4-10 illustrates the graphical presentation of the deviations from the zero reading in both A and B directions as well as the map of deviation in the deformation tube in section D+065 in RDHP for various measuring dates.

3.2.3. Monitoring Contact Pressures in Embankment/Concrete interfaces.

For the purpose of monitoring contact pressure in RDHP, nine pressure cells were installed in the two interfaces with the concrete dam, five in the right bank interface and four in the left bank interface. A circular groove is formed in the finished concrete surface where the cell is fixed using epoxy mortar. The cable is embedded in a groove in the concrete surface and routed to the nearest junction box where the readings are automatically taken by the ADAS.

Figure 3-10 Fixation of pressure cells in the concrete face.

The sensor used is a vibrating wire type where the linear reading is converted to pressure values by the following equation:

$$\boldsymbol{P = G(R_0 - R_1)} \qquad \text{..} \quad 4$$

where: P = pressure in kilopascal

G = calibration factor (provided in the calibration sheet)

R_0 = current reading in linear units (LU)

R_1 = initial reading in linear units (LU)

If the frequency is measured, it can be converted into linear units using the following equation:

$$\boldsymbol{R = \frac{F^2}{1000}} \qquad \text{..} \quad 5$$

where: R = reading in linear units

F = frequency in Hz

Then; results are plotted against time together with the upstream water level to monitor changes in contact pressure relative to changes in reservoir levels as shown in Figure 4-11.

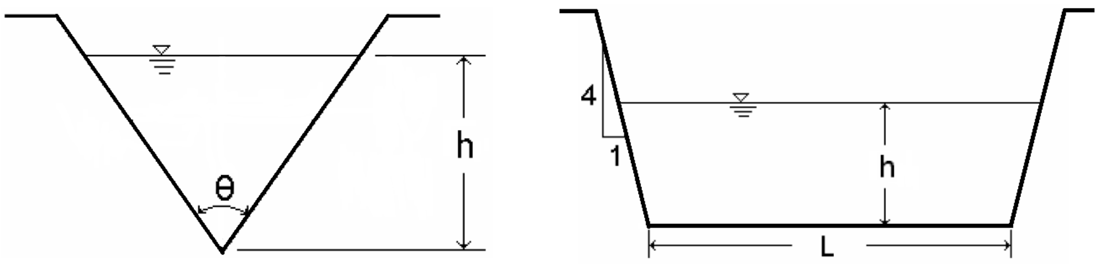
3.2.4. Monitoring of Seepage

Seepage water is collected in the toe drain and led through some discharge channels to the downstream. Some of the seepage measuring stations are located in points along the toe drain, while others are located at the discharge channels.

3.2.4.1. Installation

The measuring weirs and measuring pipes are installed in a concrete wall blocking the path of the water flowing in the toe drain.

In RDHP only V-notch weirs are used to measure seepage flow while in DCUAP trapezoidal weirs and seepage pipes are used. The side slopes of the trapezoidal weir are splayed out at an angle of 14° with the vertical (Cipoletti type).



a b

Figure 3-11 : V-notch weir (a) and Cipoletti weir (b).

3.2.4.2. Measurements

In the case of measuring pipes, seepage flow rate is simply measured by taking the time needed for the water to fill a calibrated bucket, and then dividing the volume of water (in liters) by time (in seconds) to give the flow rate (in liters/s).

Figure 3-12 : Measurement of seepage pipes.

In the case of seepage weirs, the seepage flow rate is calculated by the equation relative to the weir type i.e. V-notch or Cipoletti as follows:

For V-notch weirs: $Q = Ch^{5/2}$

For Cipoletti weirs: $Q = CLh^{3/2}$

where:

Q = Seepage flow rate (l/s).

C = Appropriate factor for the type of weir.

h = Upstream head of water (m).

L = Length of the bottom of the Cipoletti weir.

Figure 3-13 : Measurement of water head at seepage weirs.

3.2.4.3. Presentation of Results

Results are presented graphically by plotting seepage flow rates in the primary Y-axis and the reservoir water level in the secondary Y-axis against dates in the X-axis as shown in Figure 4-14.

3.2.5. Monitoring of Seismic Events

In RDHP the accelerometer is installed in a concrete plinth founded deep in the rock of the foundation of the concrete dam. The concrete plinth is isolated from the walls of the protecting concrete room so that it will only record seismic events. No seismic event is recorded till now no.

3.2.6. Automation of Monitoring

As can be seen in Appendix B the quantity and distribution of the dam instrumentation makes taking measurements and collecting data from these instruments is not an easy job, and here rises the need to adopt a proper long-term monitoring program and a well-controlled data collection and management system.

In RDHP embankments, instruments are distributed mainly in seven sections in the right bank and seven sections in the left bank. Each section consists of a number of vibrating wire piezometers installed in different elevations within the section and two deformation tubes, one in the dam toe and another in the downstream slope. Only the vibrating wire piezometers are incorporated in the AMS. The vertical and horizontal deformations are read manually, because, according to types of readout devices used, the process of reading involves human interference to lower the probes to different elevations in order to complete the readings.

At each section, a junction box is installed in the AMS shaft to collect all the cables of the vibrating wire piezometers and to facilitate manual reading of sensors, one by one, through connecting the vibrating wire readout device (Figure 3-3) to the reading plug and switch between the channels.

a. Cables connected to the junction box.

b. Manual reading panel.

Figure 3-14 : Junction box.

Each junction box is connected through a data cable either to a dedicated data-logger panel, as the case with widely apart sections, or to a shared data-logger panel with close neighboring sections. Figure 3-15 shows the enclosures of the data-logger panel, which are:

a. A rechargeable battery (to provide AC power supply).
b. The data-logger (to control taking and storing both scheduled and random readings).
c. An AVW (to convert signals).
d. One or more Multiplexers (to expand the number of sensors controlled by one data-logger).
e. A telephone modem (to provide communications between the data-logger and the main computer).

Usually, the data-logger is programed to take and store readings at predefined time intervals according to the significance of the measurements. At the time of reading, the data-logger sends the signal to the ***AVW*** which converts it to a signal ***understandable*** by the sensors and sends it to the ***multiplexer*** which spreads it among all the connected sensors at the same time. When receiving the signal, the tensioned wires of the sensors are excited and the frequency of their vibrations are read and sent back to the ***multiplexer*** which collects it and forwards it to the ***AVW***. Again, the ***AVW*** converts the signal to a signal ***understandable*** by the data-logger which stores the readings into the selected file format.

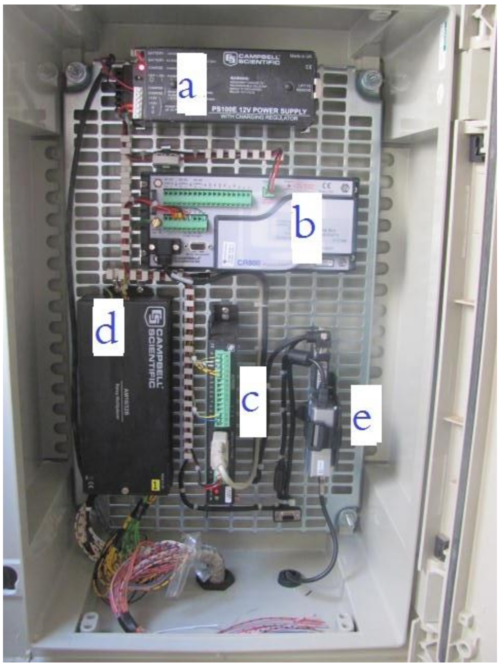


Figure 3-15 : Components of the Data-logger panel.

A portable PC can be used to make direct connection with the data-logger at site to download the data file. Direct connection is also needed for the purpose of programing the data-logger as well as performing diagnosing and troubleshooting operations. All these operations are made through ***Loggernet*** software.

In the central PC, located in the control room, ***Loggernet*** is used to perform scheduled connection with the data-loggers, through mobile network, to download and store the data files in the specified data directory.

Chapter Four: ANALYSIS OF INSTRUMENTATION DATA

4. ANALYSIS OF INSTRUMENTATION DATA

As mentioned before, one of the main goals of geotechnical instrumentation is to verify the performance of the embankment dam in terms of the deferent geotechnical parameters, and to monitor signs of adverse behaviors in order to take prevention measures against predicted potential failures. This can be achieved by the proper analysis of the data gained by the instrumentation system. For the purpose of demonstration, readings of piezometers and settlement tubes of DCUAP will be used for verifying the performance of the embankments regarding piezometric pressures, uplift pressures and vertical deformations. Analysis of horizontal deformations will be demonstrated by taking an example section from RDHP.

4.1. Piezometric Pressures

In the example section, six vibrating wire piezometers plus one standpipe piezometer are located in such a way to give the information needed to evaluate the performance of the main section components and to monitor the changes in piezometric pressures corresponding to changes in reservoir levels.

4.1.1. Efficiency of the cutoff wall

In DCUAP a mixed in place cutoff wall (MIP) is introduced under the embankment section to lengthen the path of seepage water and reduce the pressure at the exit point in the downstream side and hence reduce the potential of failure by piping and erosion of downstream toe. Piezometers PP1.1 and PP1.2 are used to evaluate the performance of the mixed in place cutoff wall by monitoring the reduction in piezometric pressures between the upstream and downstream.

In the user form for monitoring piezometric pressures in Figure 4-1, the vertical blue lines upstream and downstream of the (MIP) indicates the heads in piezometers PP1.1 and PP1.2 respectively of which values and the reduction in pressure for the selected date are displayed in the upper part of the form. During the past two impounding seasons it was found that the reduction percent varies with the value of the reservoir level and ranges from 25% to 40%.

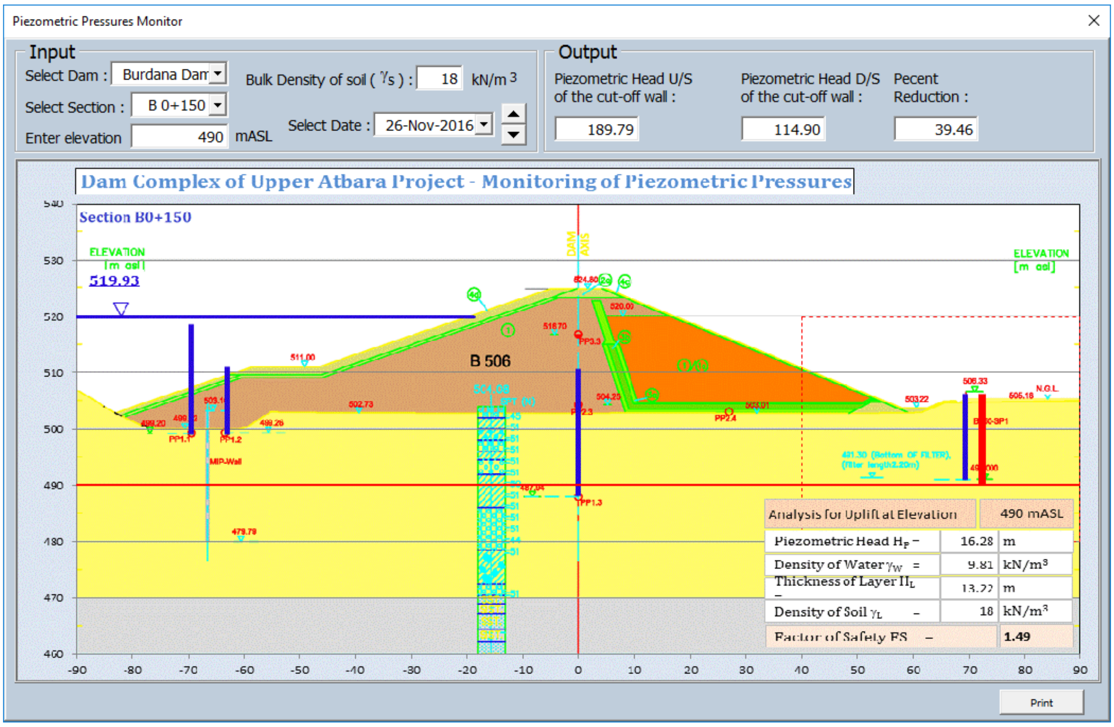


Figure 4-1 : Monitoring piezometric pressures in DCUAP.

4.1.2. Monitoring changes in piezometric pressures

The readings of every single piezometer is plotted, together with the reservoir levels, to monitor the changes of piezometric pressures in the deferent locations of the section related to changes of reservoir levels. The chart is done in to models. In the first chart (Figure 4-2) the dates from the first reading up to the last one are plotted in the x-axis while piezometer readings and reservoir levels are plotted in the y-axis. In the other chart (Figure 4-3) dates of the year are plotted in the x-axis and the readings of every year together with reservoir levels are plotted are plotted in the y-axis, so readings are plotted in a comparative way, i.e. readings of the current impounding can easily be compared with previous ones.

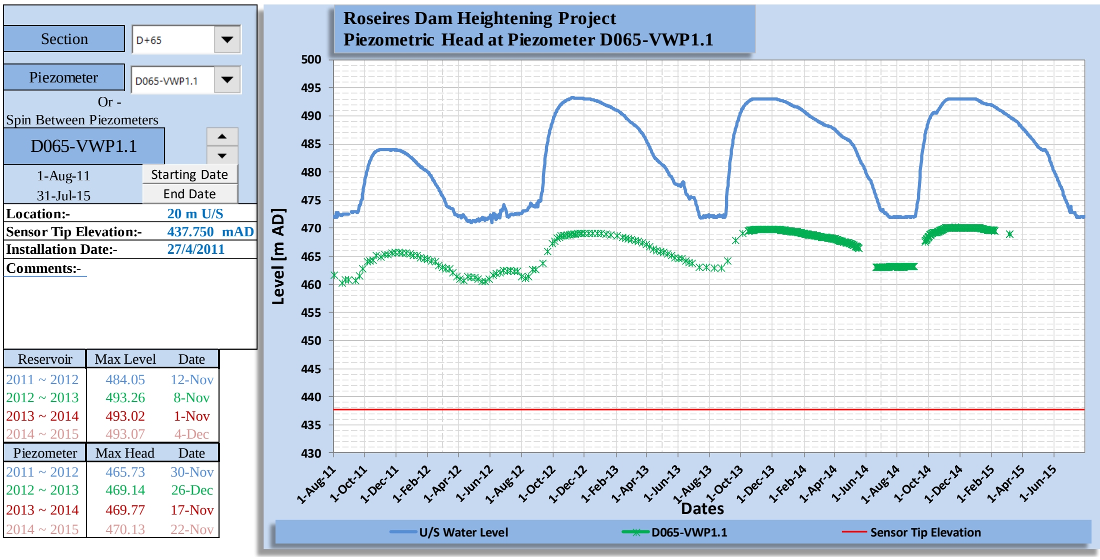


Figure 4-2 : Continuous plot of piezometric readings in RDHP.

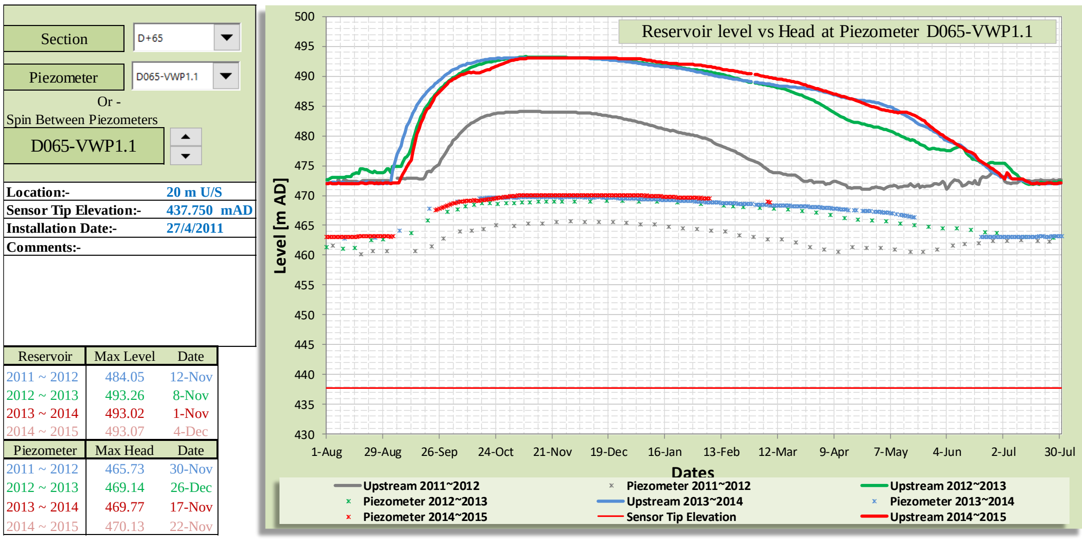


Figure 4-3 : Comparative plot of piezometric readings in RDHP.

4.1.3. Analysis for Uplift

In both RDHP and DCUAP, the part of the section of the dam in the vicinity of the toe drain is considered to be the most critical location for uplift pressures because normally the soil in this location is striped to form the section of the drain, hence, the downward force to act against hydrostatic uplift force is reduced.

For the analysis of uplift in DCUAP an excel spreadsheet is designed to monitor the change in piezometric pressures in the area of the toe drain and calculate the factor of safety throughout the period of impounding. Figure 4-4 displays the user form for monitoring uplift in the toe drain in which the continuous red line indicates the elevation where the section is analyzed for uplift (normally the interface between the top two layers). The spreadsheet calculates the piezometric head (H_P) from the deference between the piezometric head of the standpipe located near the toe drain and the elevation of the interface between the top two layers and multiply it by the density of water (γ_W), to give the uplift force. It also calculates the thickness of the top layer (H_L), from the deference between the elevation of the bottom of the toe drain and the elevation of the interface and multiply it by the density of the top layer (γ_L), to give the downward force. Then the factor of safety (FS) is calculated by dividing the downward force by the uplift force.

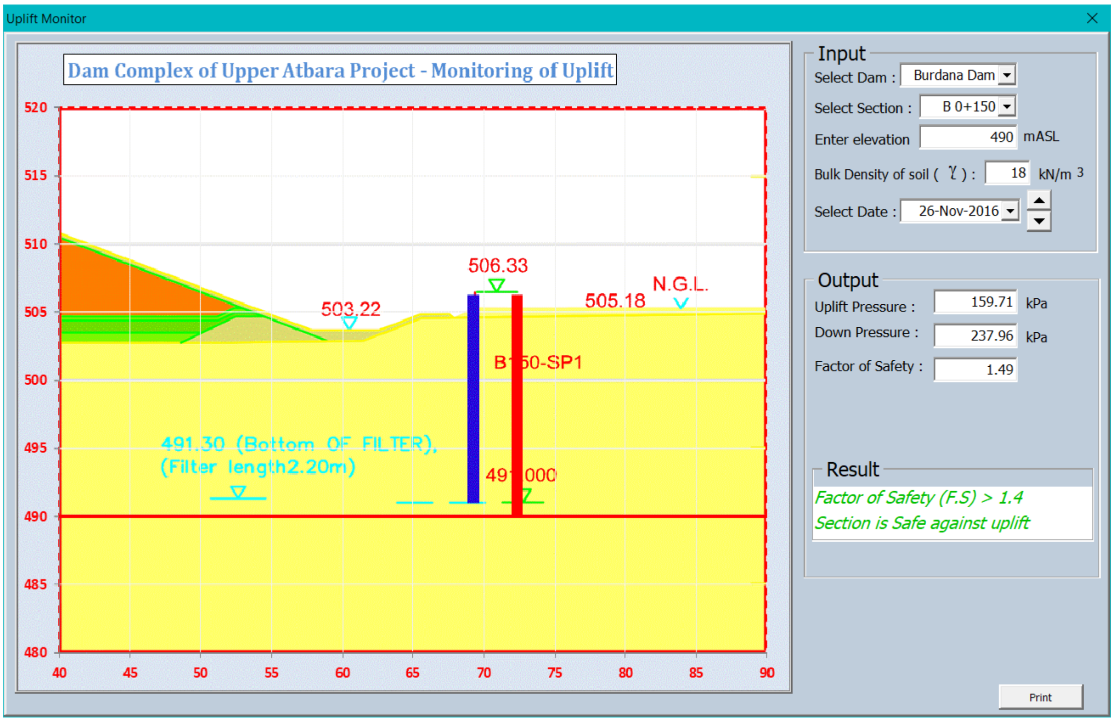


Figure 4-4 : Monitoring uplift in the toe drain.

4.2. Vertical deformations

As mentioned in 3.2.2.1 above, vertical deformations are monitored by recording the change in elevations of settlement targets installed around the deformation tube. In the first chart, Figure 4-5, the deformations of all the targets are plotted in one series where deformations (in millimeters) are in the x-axis and elevations (in meters) are in the y-axis. In the second chart, Figure 4-9, all the deformations of one targets are plotted in one series where deformations (in millimeters) are in the y-axis and measurement dates are in the y-axis.

After installing the top magnetic target, the rate of settlement continued to be relatively high (1.84 mm/day) due to additional loading from construction activities of the top embankment layers. After completion of construction, the rate of deformation started to decrease gradually until it reached 0.12 mm/day prior to the first impounding in August 2015, and the maximum settlement value in the top magnet plate reached 361.5 mm. During the impounding, the rate of settlement increased to (0.31 mm/day) and the maximum settlement value in the top magnet plate reached 383 mm. Before the second impounding, the rate of settlement decreased to (0.07 mm/day) and the maximum settlement value in the top magnet plate reached 415.5 mm.

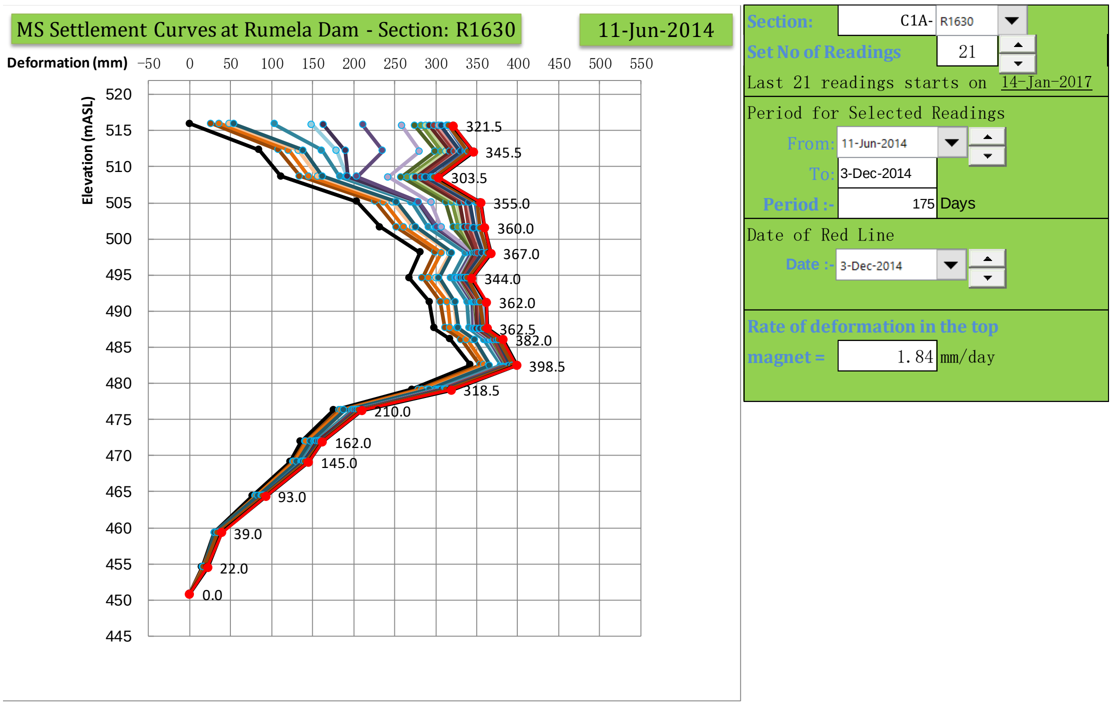


Figure 4-5: Vertical deformations after installing the top target.

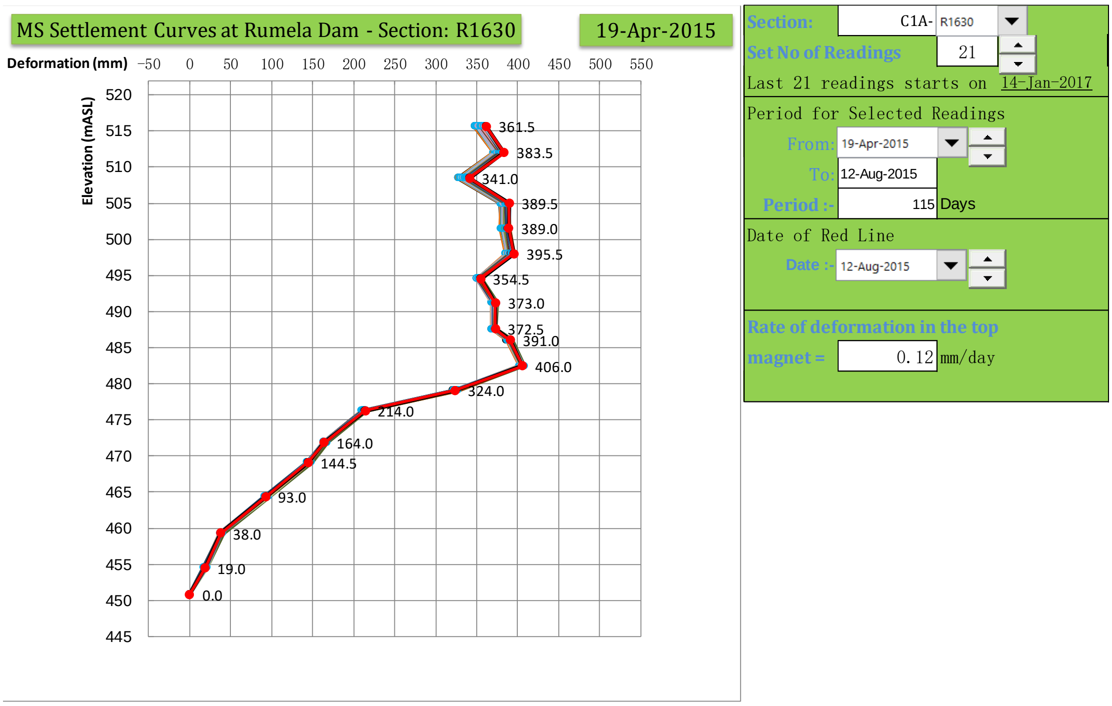


Figure 4-6: Vertical deformations prior to the first impounding.

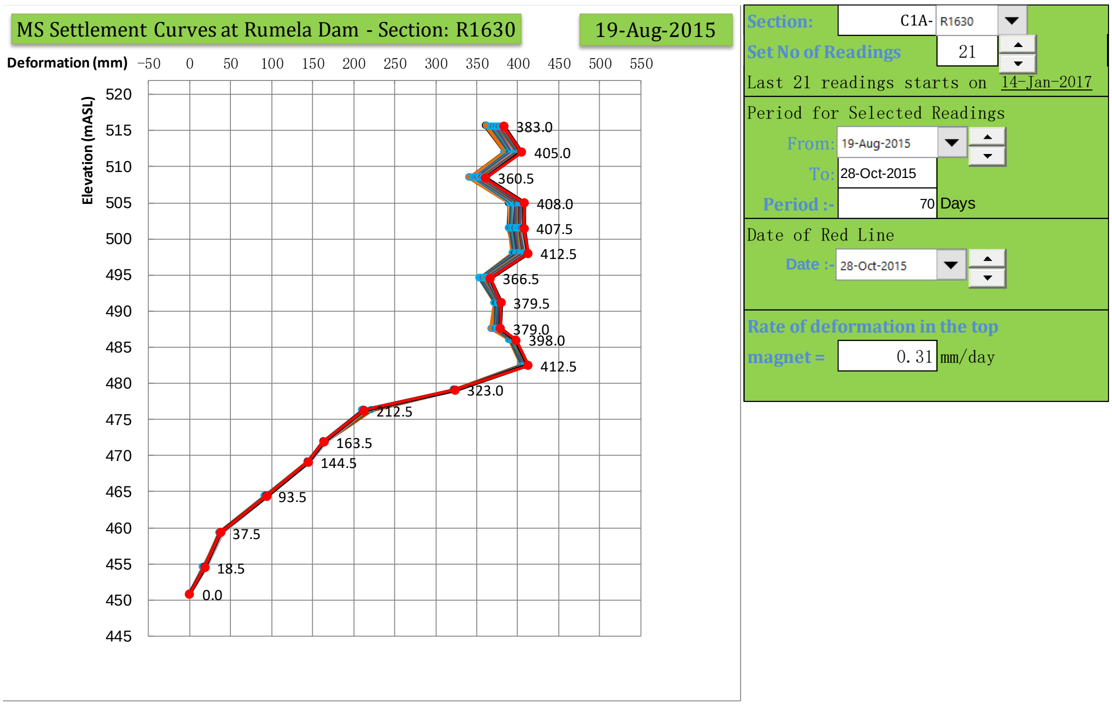


Figure 4-7: Vertical deformations during the first impounding.

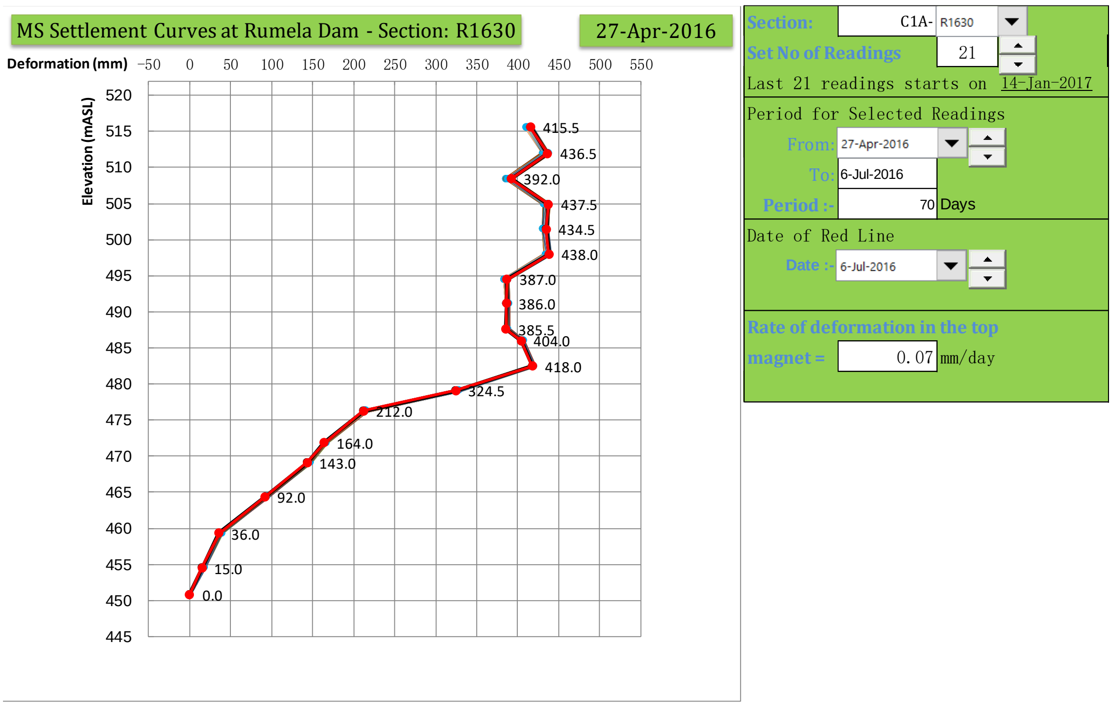


Figure 4-8: Vertical deformations before the second impounding.

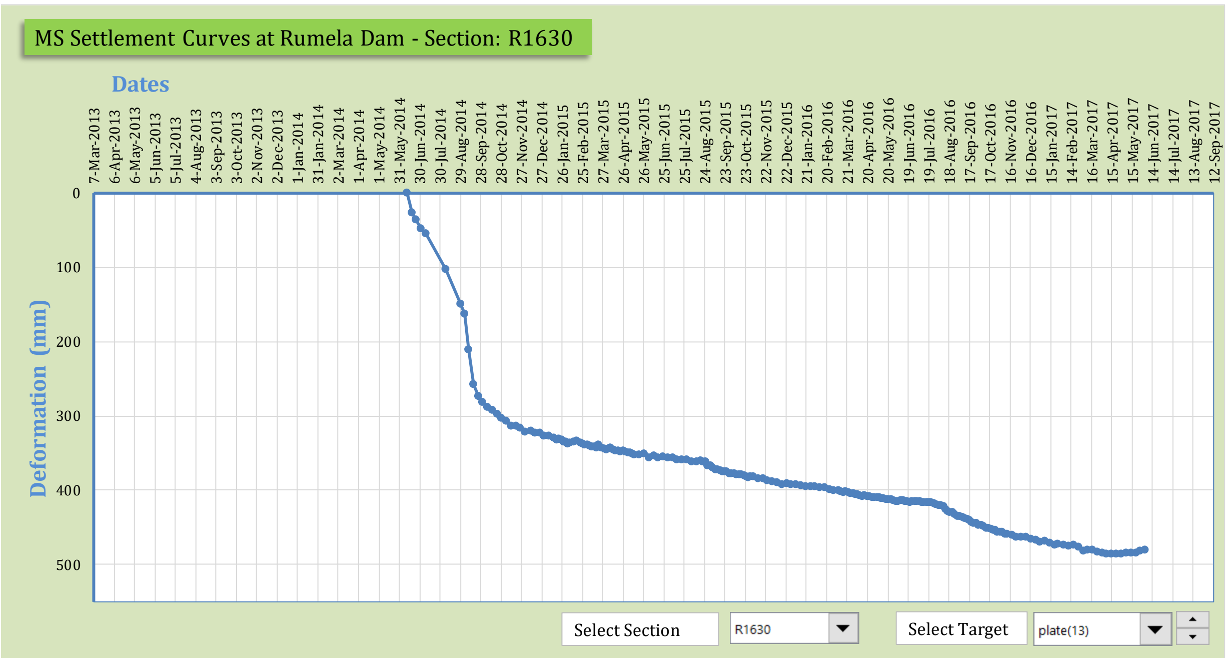


Figure 4-9: Vertical deformations of the topmost settlement target.

4.3. Horizontal deformations

The inclinometer readings are plotted as described in 3.2.2.5 above. From the plot it can be seen that the pattern of horizontal deformation does not indicate any unfavorable movement in the embankments nor in the foundation. It can also be seen that during the first impounding to the new full supply level, the horizontal deformations ranged from -6 mm (upstream) to 18 mm (downstream) in the A-axis, and ranged from -6.8 mm (east) to 10 mm (west) in the B-axis.

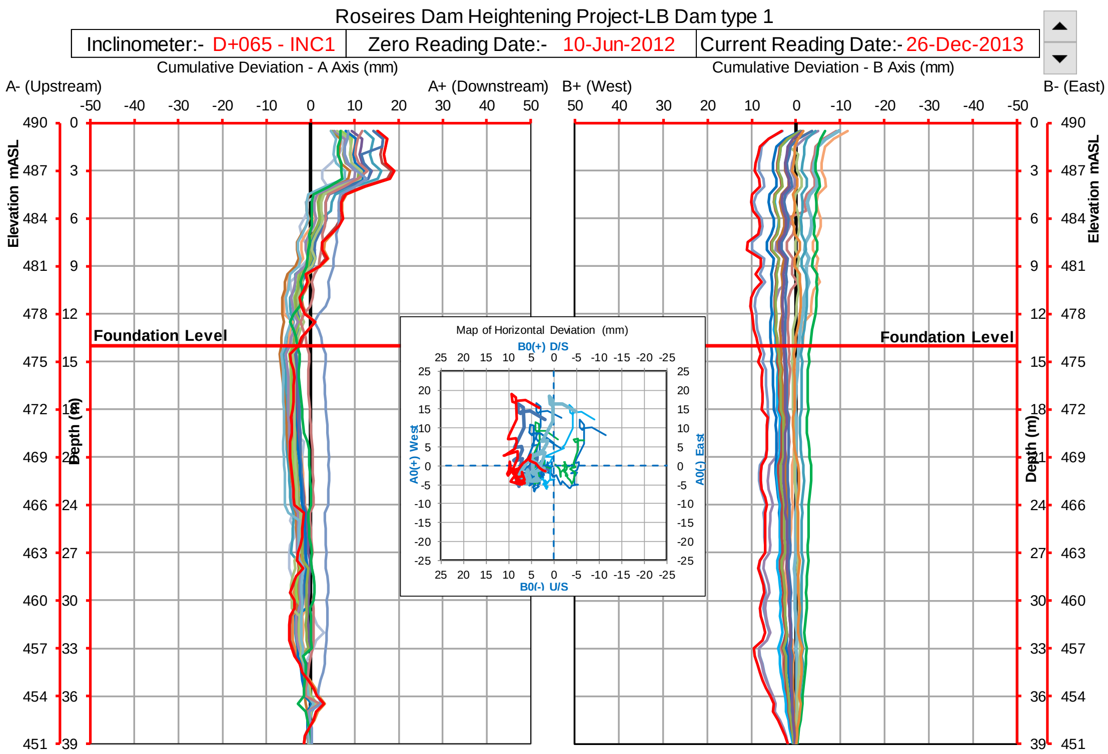


Figure 4-10 Analysis of horizontal deformations in RDHP.

4.4. Contact Pressures in Embankment/Concrete interfaces

In the plotted readings of the pressure values in the interface between the embankments and the concrete dam, Figure 4-11 below, it can be seen that the embankments are always in contact with the concrete surface to the extent not to form a gap to provide path for the seepage water in this critical area.

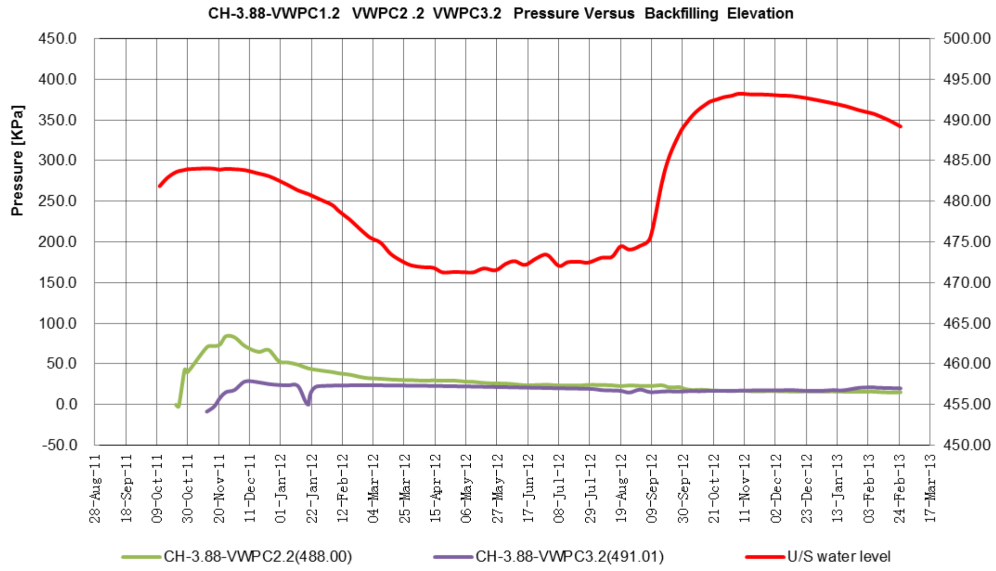


Figure 4-11 Presentation of Contact pressure results.

4.5. Seepage observations.

Figure E-1 in Appendix E shows that no seepage is observed in Rumela left bank, and the seepage of the river section is collected and measured in the seepage pipes installed in section R1630 of which the maximum recorded quantity during the last impounding is 0.53 l/s. Seepage recording in this section is interrupted by downstream water from spillway operations during reservoir release.

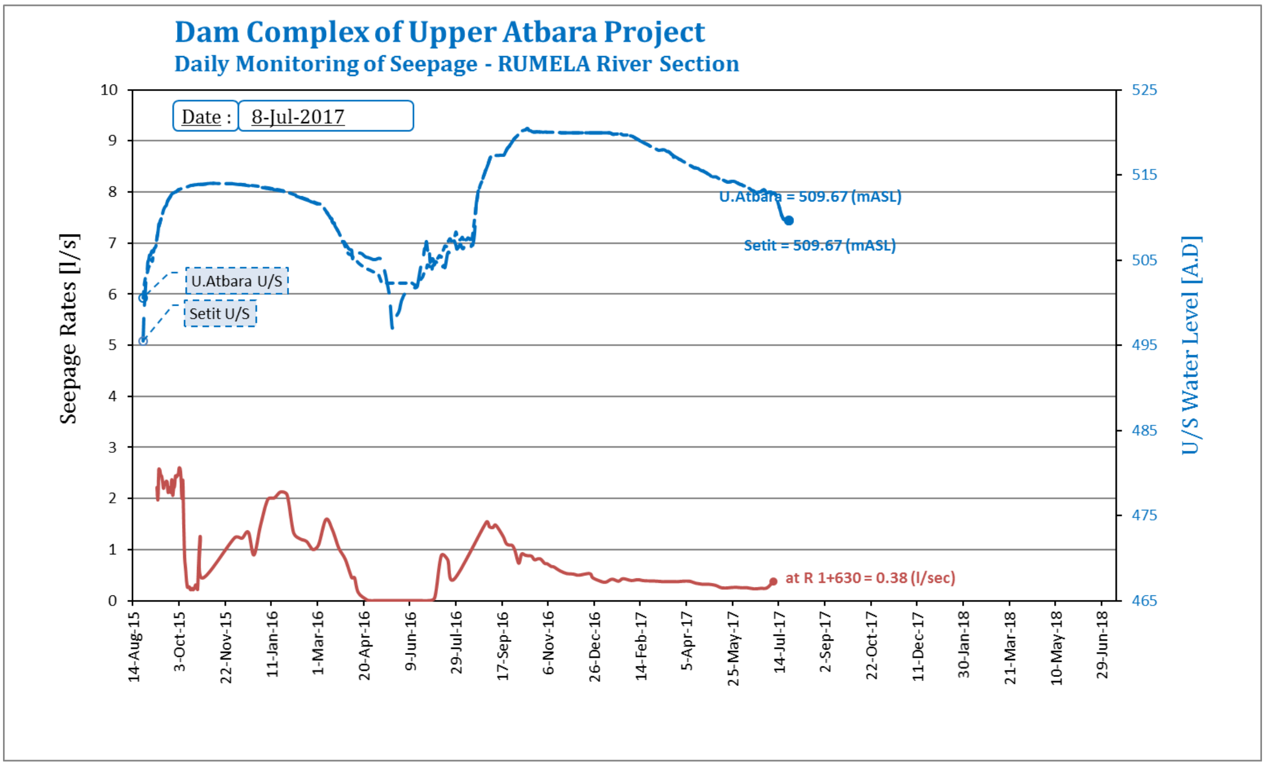


Figure 4-12 : Plot of seepage at Rumela River Section.

In the Fashaga area; Figure E-2 in Appendix E; seepage is observed in three locations in the toe drain and in the bottom of the Fashaga Wadi i.e. the area downstream between the two rivers. The three locations in the toe drain are R0+288, B0+045 and B0+840. As seen in Figure 4-13 below seepage pipes at R0+288 recorded very small rates of seepage for a very short period of time. The two other weirs records seepage for the whole impounding period. During the second impounding it was decided to raise the bottom of the toe drain at B0+045 to increase the resistance against uplift in the Fashaga area. This resulted changes in the catchment areas of the two weirs and some water changed its flow from B0+045 to B0+840. This justifies the drops in plotted rates of B0+045 accompanied by the rise in plotted rates of B0+840 in Figure 4-13 below.

For the purpose of estimating the rates of seepage from the bottom of the Fashaga Wadi, three weirs are installed in a dyke constructed to collect and measure the rate of seepage from the whole Fashaga area and deduct the seepage rates measured by the weirs of the toe drain to get the seepage quantity of the bottom of the Wadi.

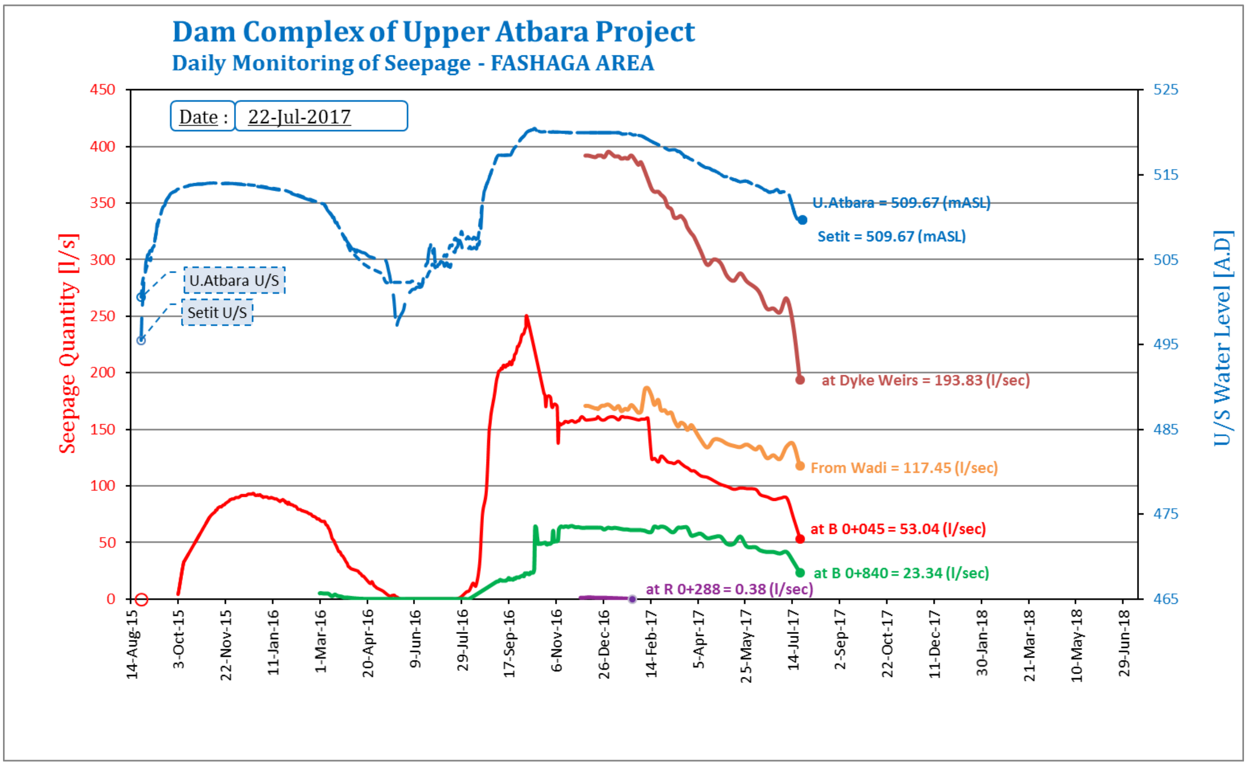


Figure 4-13 : Plot of seepage at Fashaga Area.

In Burdana Right Bank; Figure E-3 in Appendix E; seepage is observed in three locations, in the river section; which is collected and measured in the temporary pipes in B1+460, in the toe drain between chainages B1+560 and B1+870; which is collected and measured in the weir at B1+675 and near B2+460. Seepage recording in the river section is interrupted by downstream water from spillway operations during reservoir release.

Figure 4-14 below shows the plot of seepage rates in these locations.

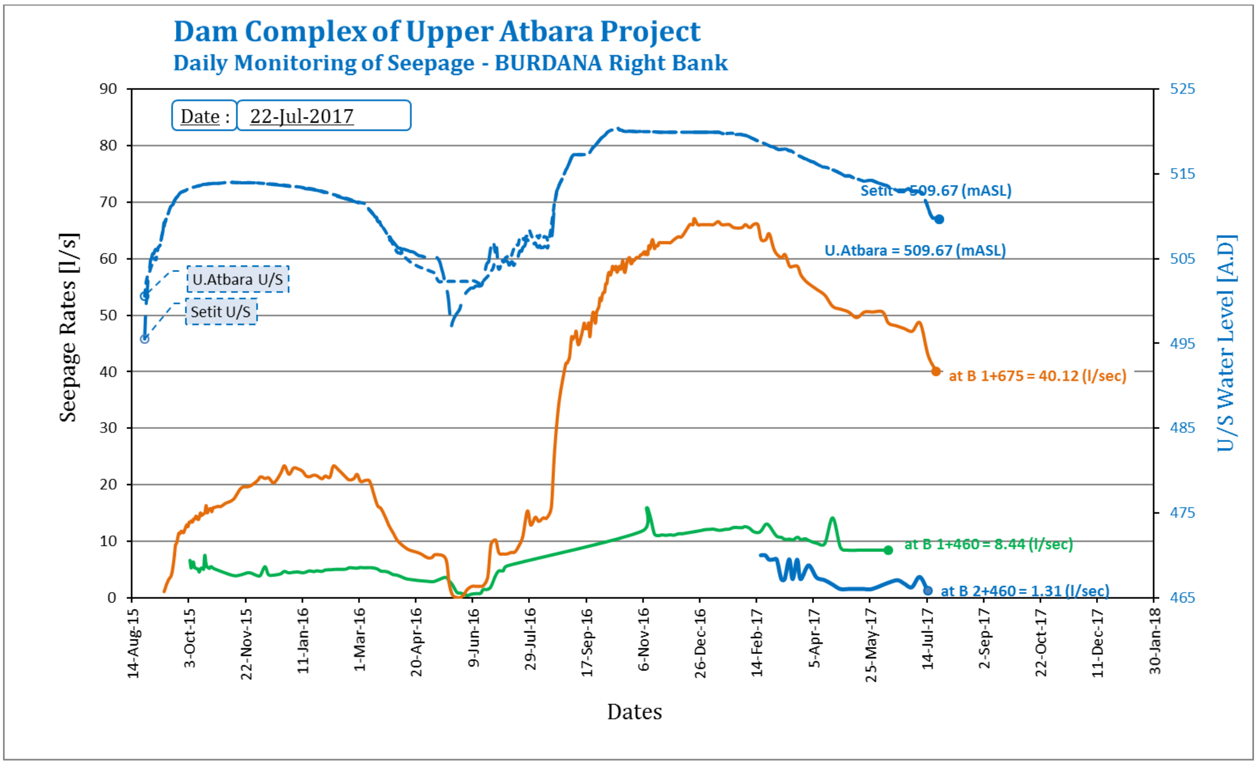


Figure 4-14 : Plot of seepage at Burdana Right Bank.

4.5.1. Evaluation of Seepage observations

In terms of cubic meters, the maximum seepage rate; which continues only for a very short period of time; is about (42,000 m^3/day) which does not constitute a significant loss to the water storage in the reservoir.

In terms of water clarity, in all seepage locations, the observed water is very clear (no turbidity and no fine soil particles). This indicates that the foundation is stable and fine soil particles are not being eroded by seepage flow.

Table 4-1: Maximum recorded seepage rates (impounding of 2016).

Area	Location	Rate (l/s)
Rumela River Section	R1+630	0.53
Fashaga Area	Fashaga Weirs	395.40
Burdana River Section	B1+460	12.79
Burdana Right Bank	B1+675	67.11
	B2+460	7.56

Figure 4-15 : Clear water of seepage pipe.

Figure 4-16 : Clear water of seepage weir.

Chapter Five: CONCLUSION AND RECOMMENDATIONS

5. CONCLUSION AND RECOMMENDATIONS

5.1. Conclusion

Based on the analysis and evaluation of instrumentation data made in the previous chapter it can be concluded that: -

5.1.1. Piezometric Pressures

a- The mixed in place cutoff wall (MIP) in the example section from DCUAP is functioning as expected in reducing the head.

b- In the example section from RDHP, no significant changes are observed in piezometric pressures in successive impounding periods.

c- In the example section from DCUAP, the factor of safety against uplift is always found to be higher than 1.4 which indicates that the section is safe against adverse uplift pressures.

5.1.2. Vertical deformations

In the example section from DCUAP, readings of vertical deformations show that the rate of settlement has decreased significantly and indicates that the foundation and the fill material are in the secondary consolidation stage. It can also be seen that the impounding has no significant effect in the rate of settlement.

5.1.3. Horizontal deformations

In the example section from RDHP, readings indicate that the response of the embankments to the reservoir filling and emptying is normal and no progressive horizontal deformation is developing; which might indicate sliding or slope failure either to the downstream or to the upstream direction.

5.1.4. Contact Pressures in Embankment/Concrete interfaces

Readings show that embankments are always in contact with the concrete face without providing path to the seepage water.

5.1.5. Seepage observations

In terms of quantity, the recorded seepage does not constitute a significant loss to the water storage in the reservoir.

In terms of water clarity, observations indicate that the foundation is stable and fine soil particles are not being eroded by seepage flow.

From the above, it can be concluded that the performance of the embankments; in terms of the monitored parameters; is considered satisfactory and failure modes related to these parameters are less likely to happen.

It should be highly emphasized that this conclusion should not be considered to be valid for ever, because no dam is *immune* against changes in safety parameters due to changes in geotechnical conditions. This is why dam safety monitoring is a continuous operation throughout the whole live time of the dam.

It should also be emphasized that monitoring staff should not rely only on instrumentation reading for the verification of the performance of the dam because instruments are only installed in selected representing sections to give a general idea of the performance. Routine visual inspections should be made by experienced staff and observations and incidents should be related to instrumentation readings for the purpose of conducting a reliable review of the safety of the dam.

5.2. Recommendations

Geotechnical instrumentation is not a branch of civil engineering. It should be considered as a tool used by geotechnical engineers to help them better understand geotechnical problems in order to suggest the most effective solutions and to have better control over implementation of these solutions. Hence, it is highly recommended that the staff responsible for instrumentation have a good geotechnical background to enable them understand instrumentation readings and relate them to the geotechnical characteristics of the structure.

It is also highly recommended that universities should introduce dam engineering including, design, construction and dam safety in the geotechnical courses and coordinate with dams' operation companies in the country to share their knowledge and practical experience with the students of civil engineering.

I also recommend that someone should make benefit of the data available in the recently constructed dams and consider making his research on instrumentation of concrete dams.

REFERENCES

1- Geotechnical Instrumentation for Monitoring Field Performance – 1988, By John Dunnicliff and Gordon E. Green.
2- Instrumentation of Embankment Dams and Levees – 1995, U.S. Army Corps of Engineers.
3- State of Colorado Dam Safety Manual – 2002, John Schurer, Eric Wilkinson, James Norfleet, John Van Sciver, Clint Huntington, Chin Lee and Allan Rogers.
4- Earth Manual, Third Edition – 1998, U.S. Department of The Interior Bureau of Reclamation.
5- Why Include Instrumentation in Dam Monitoring Programs – 2008, U.S. Society on Dams.
6- Seepage Analysis and Control for Dams – 1993, U.S. Army Corps of Engineers.
7- Guidelines for Operation and Maintenance of Dams in Texas -

APPENDICES

Appendix A : Layouts of RDHP and DCUAP.

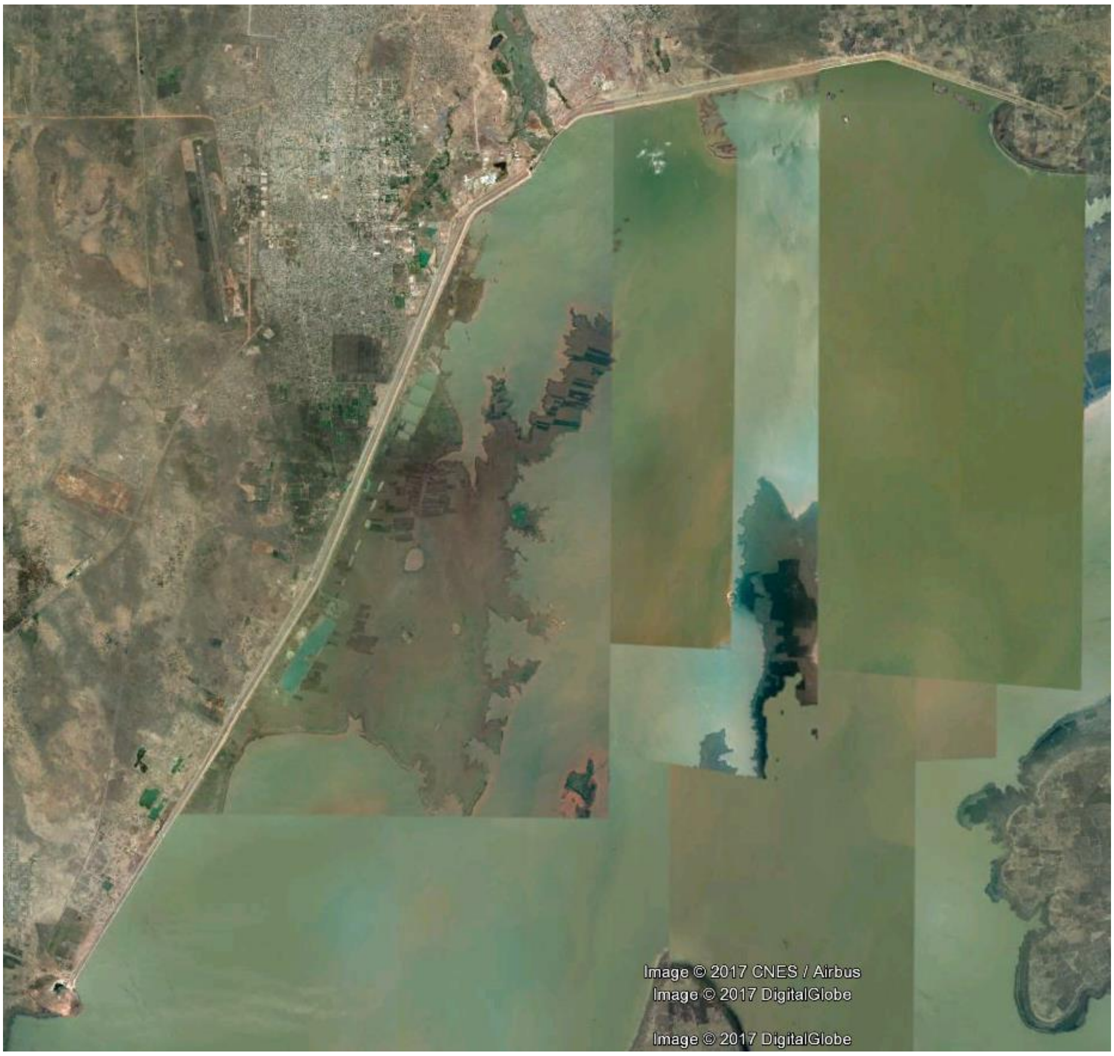


Figure A-1: Satellite image of RDHP.

Figure A-2: Satellite image DCUAP.

Appendix B : Example Cross Sections from RDHP and DCUAP.

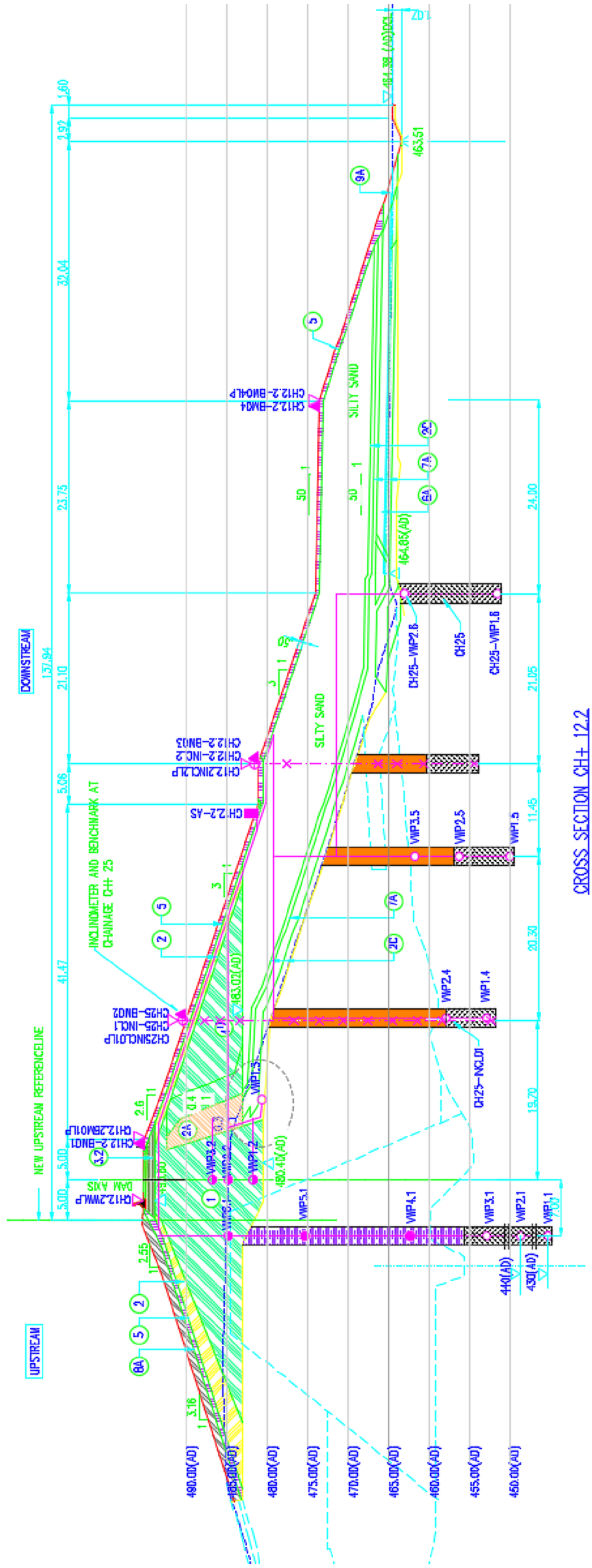


Figure B-1: RDHP - Section CH+12.2.

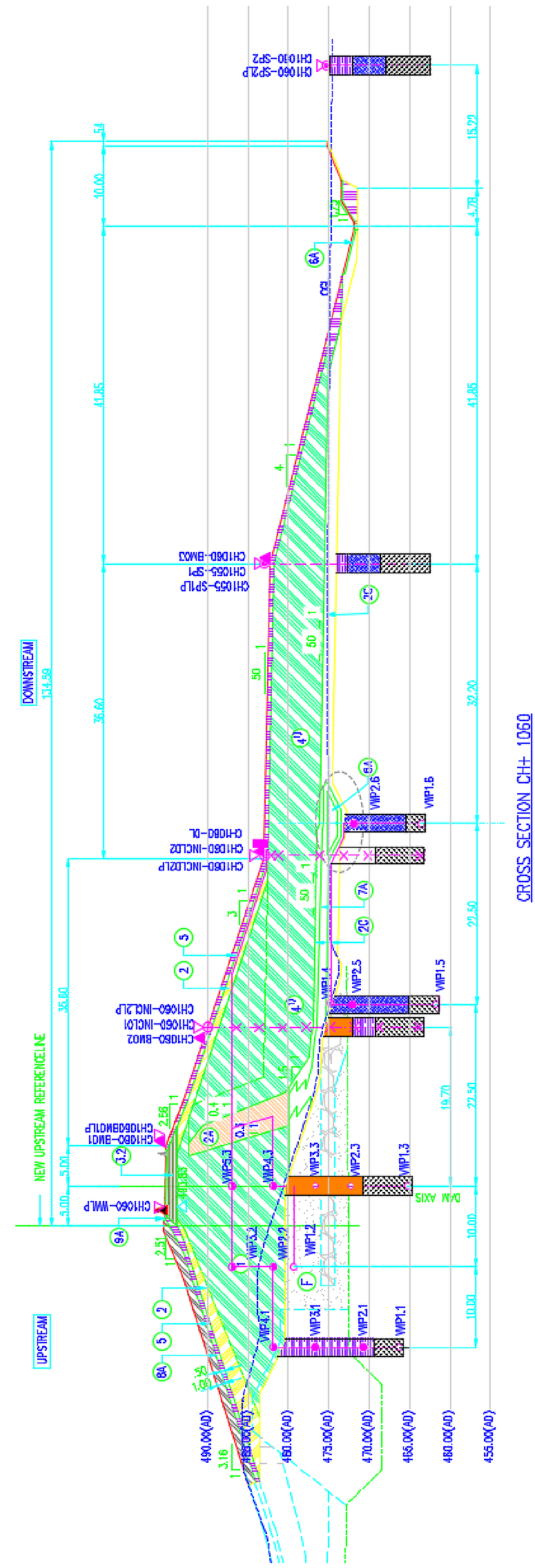


Figure B-2: RDHP - Section CH+1060.

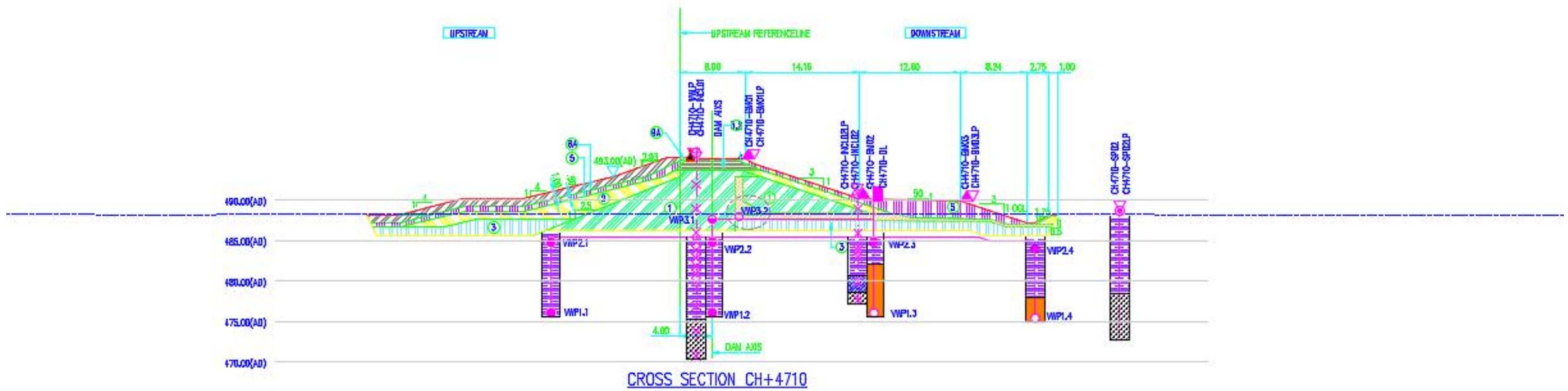


Figure B-3: RDHP - Section CH+4710.

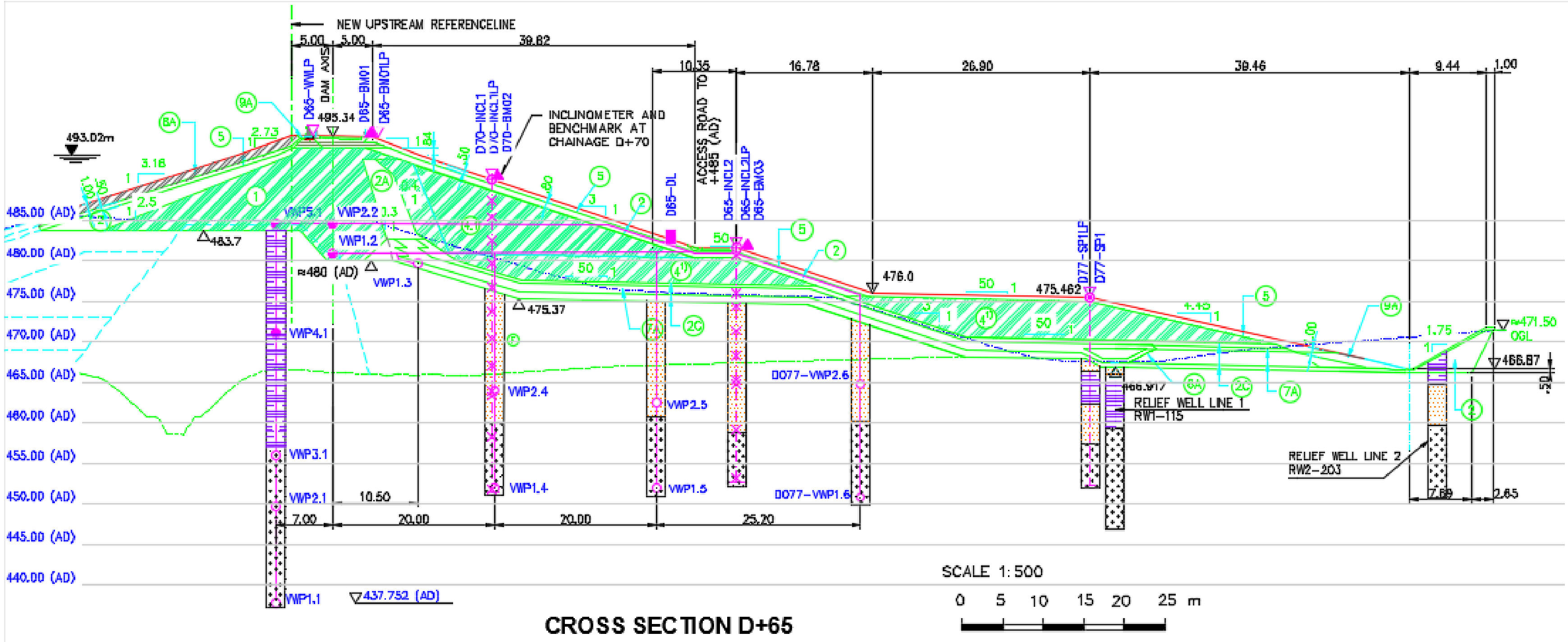


Figure B-4: RDHP – Section D+65.

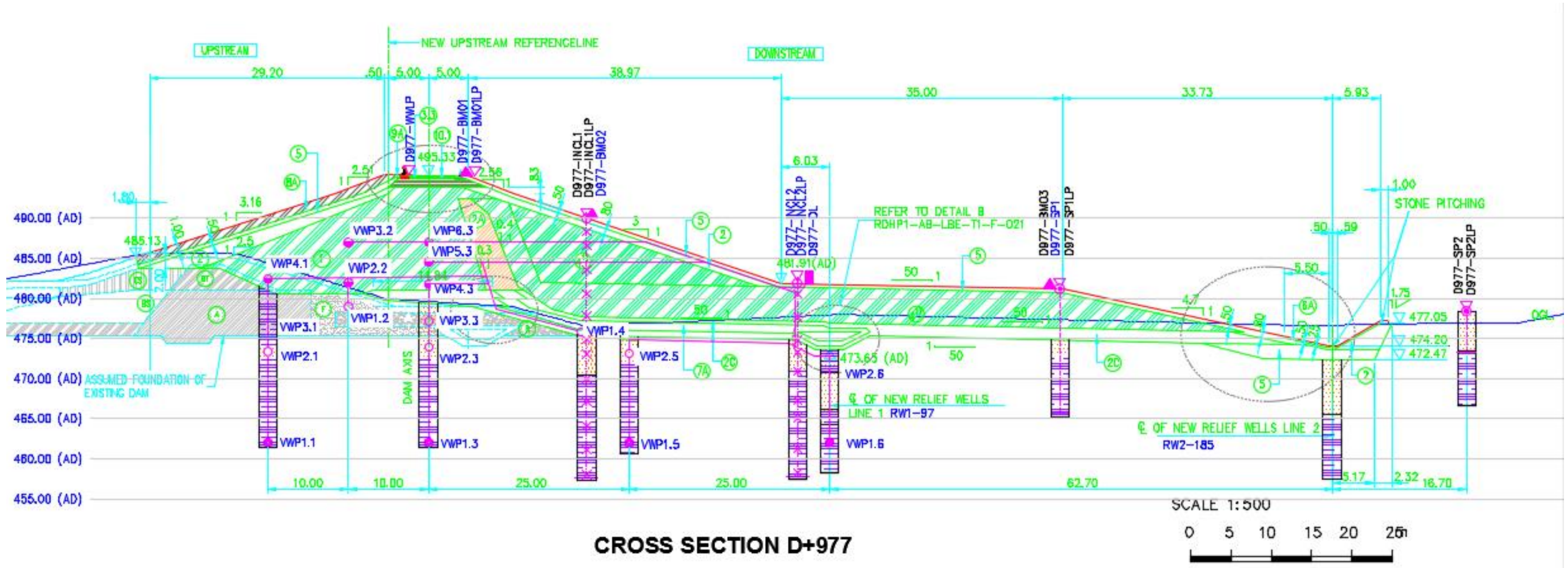


Figure B-5: RDHP – Section D+977.

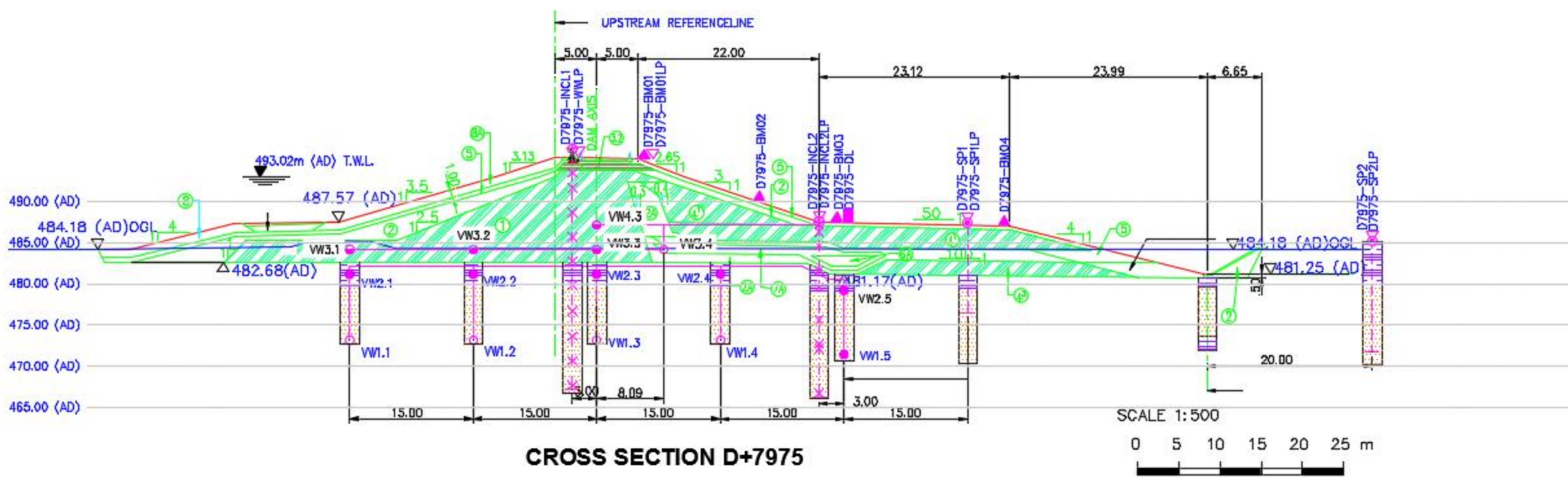


Figure B-6: RDHP – Section D+7975.

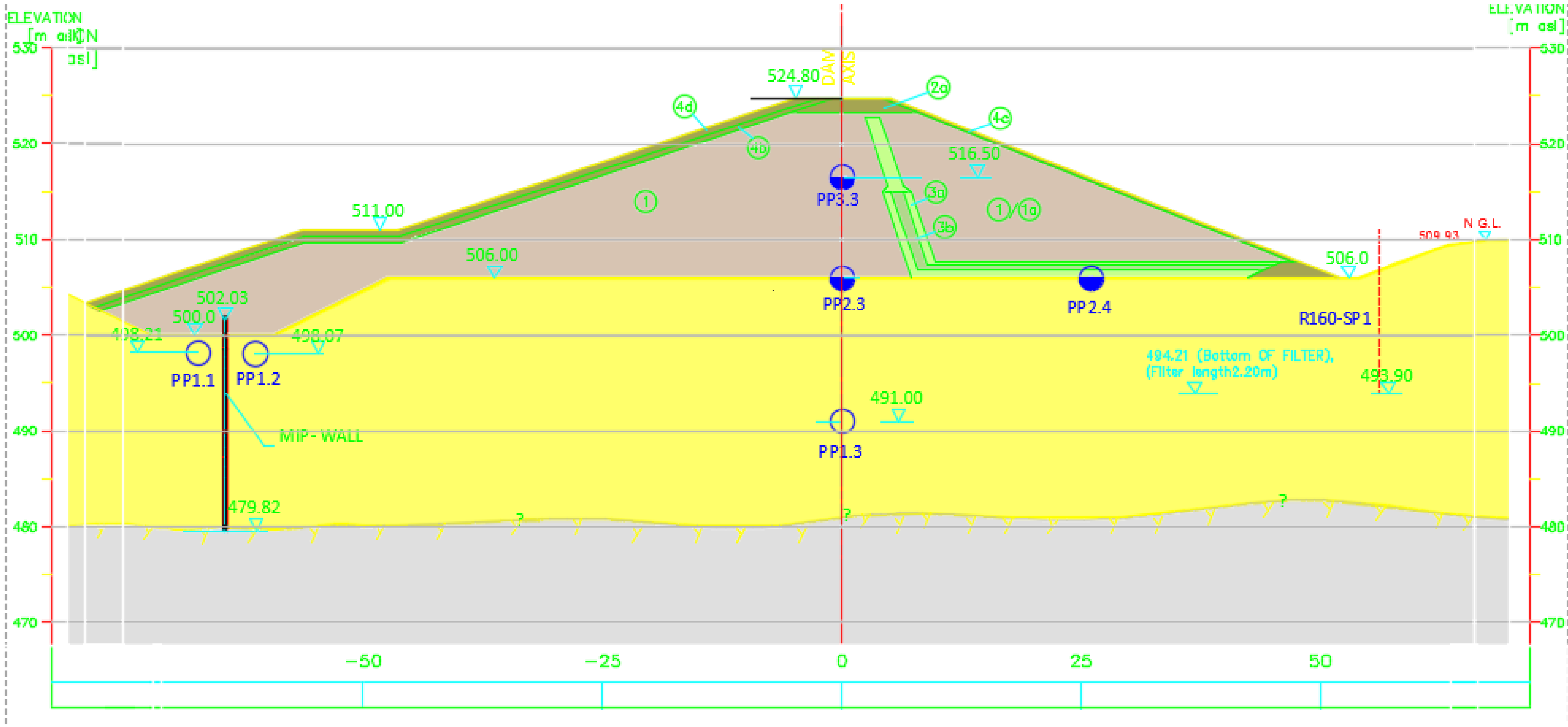


Figure B-7: DCUAP – Section R0+160.

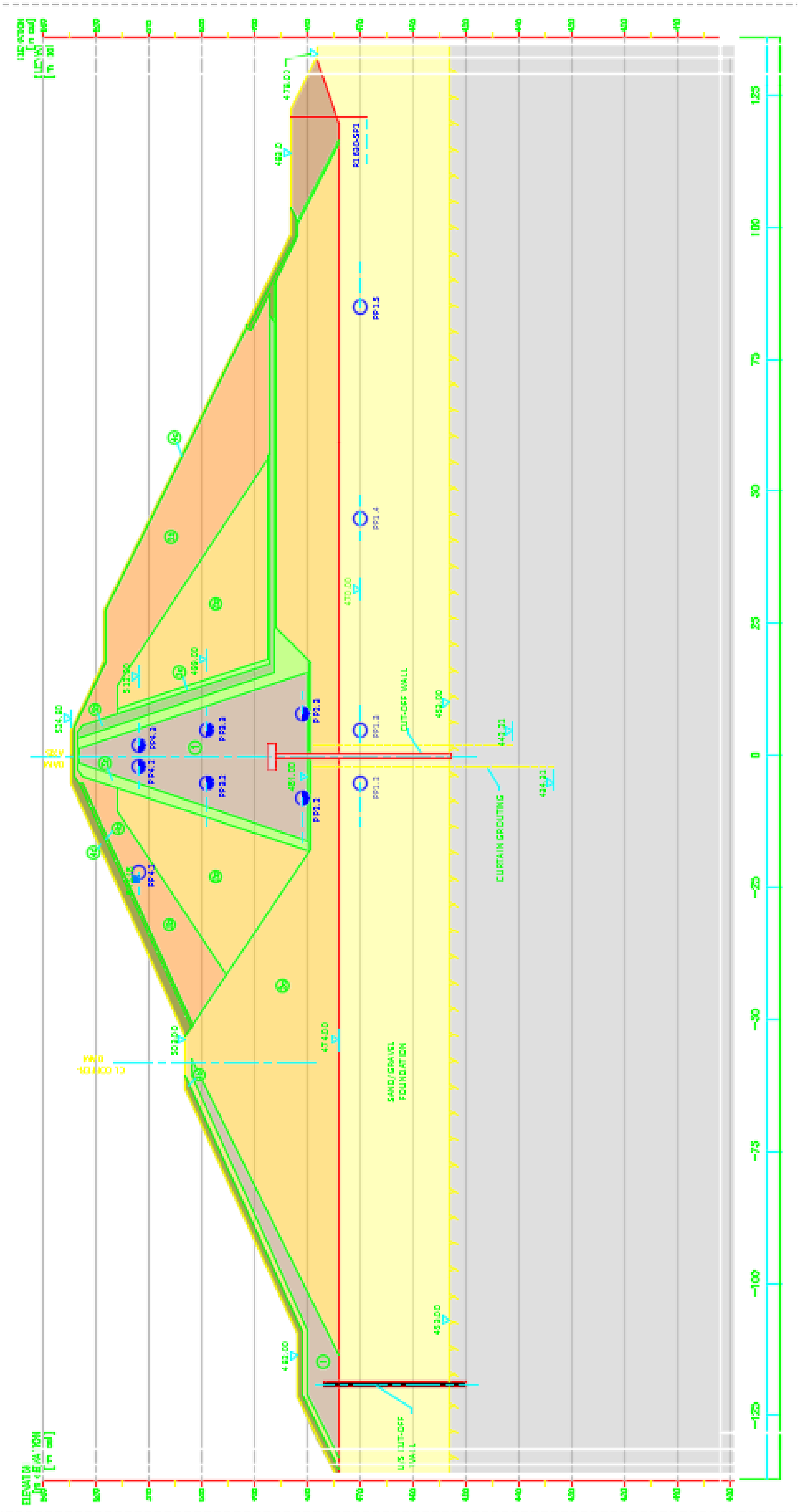


Figure B-8: DCUAP – Section R1+630.

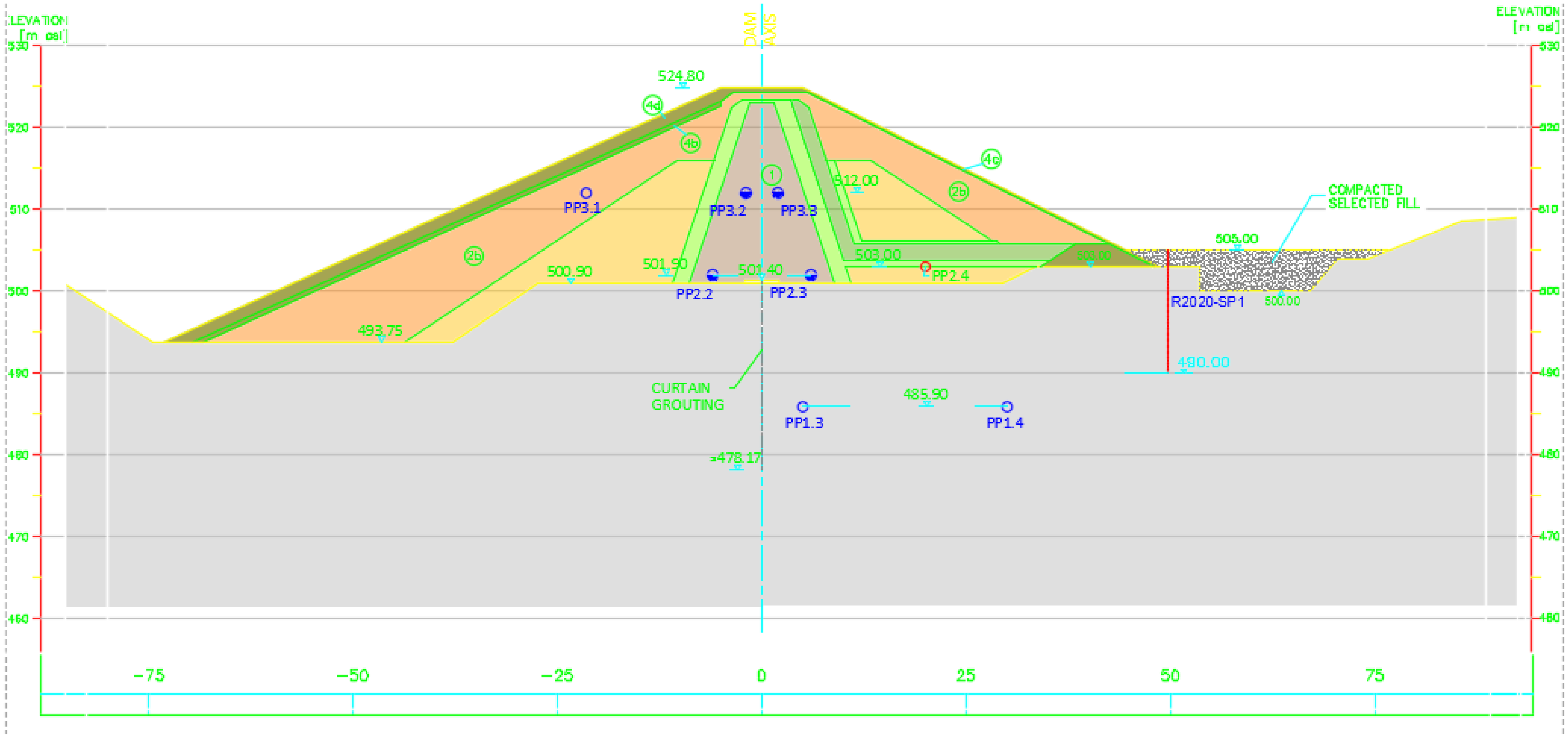


Figure B-9: DCUAP – Section R2+020.

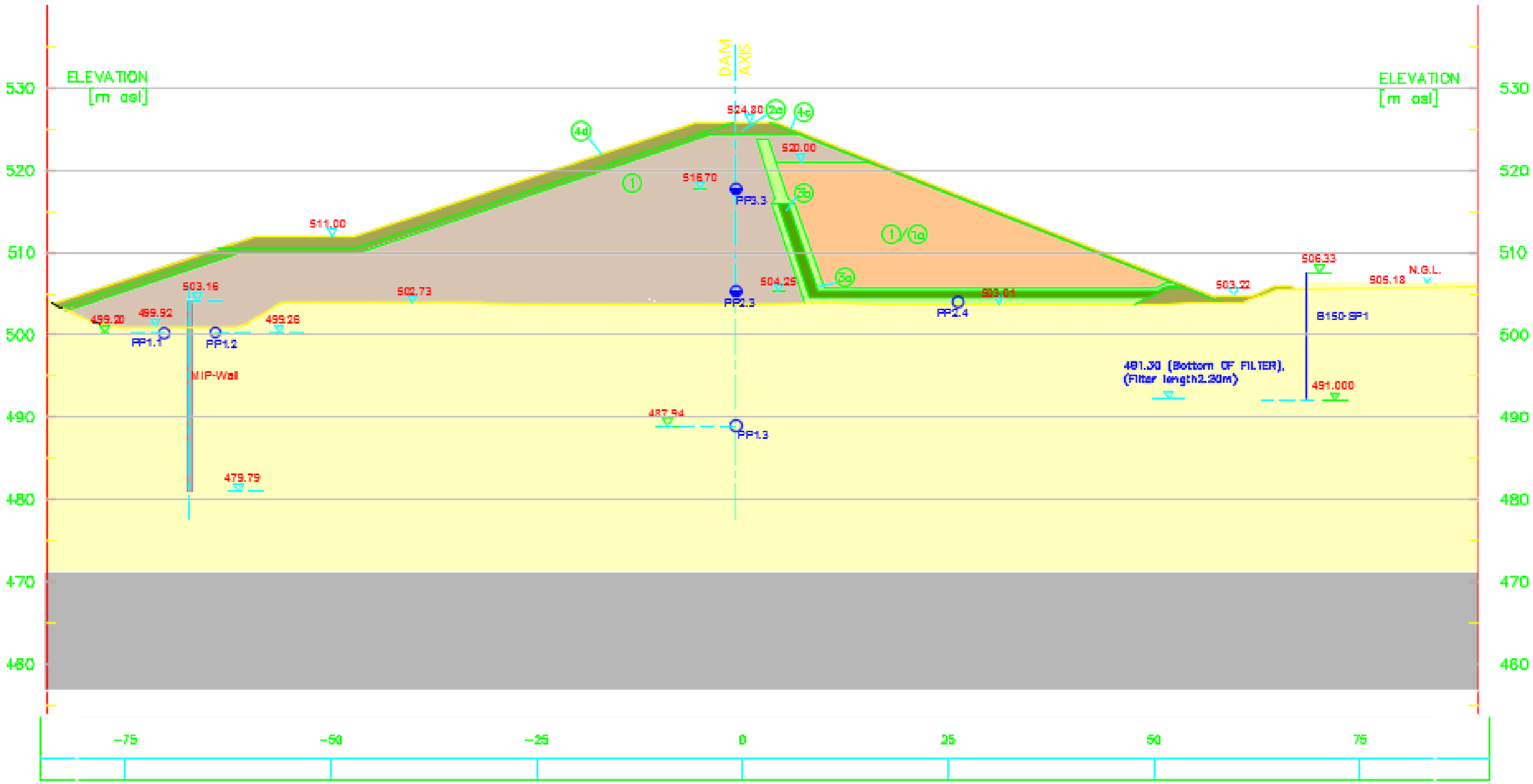


Figure B-10: DCUAP – Section B0+150.

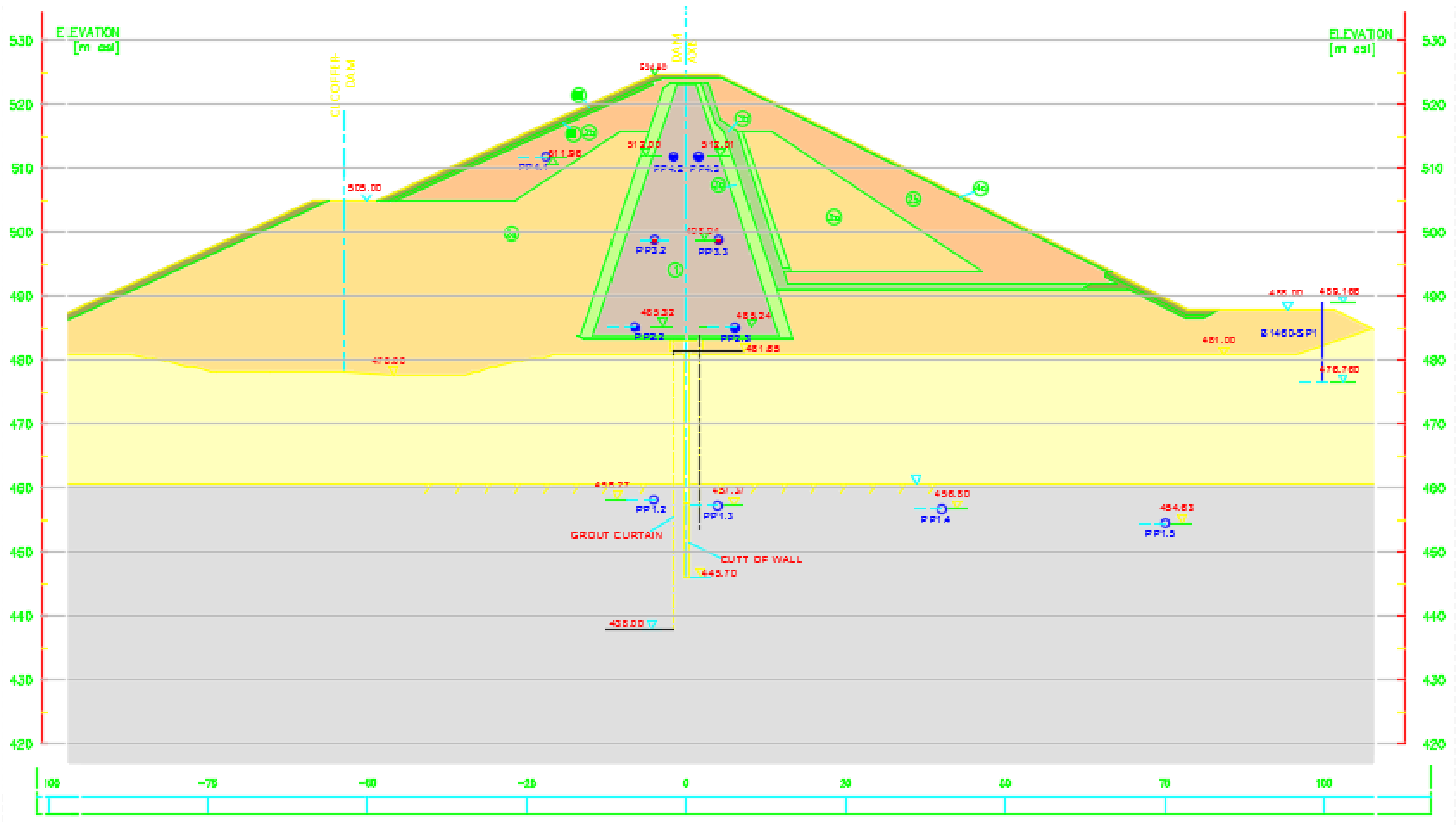


Figure B-11: DCUAP – Section B1+460.

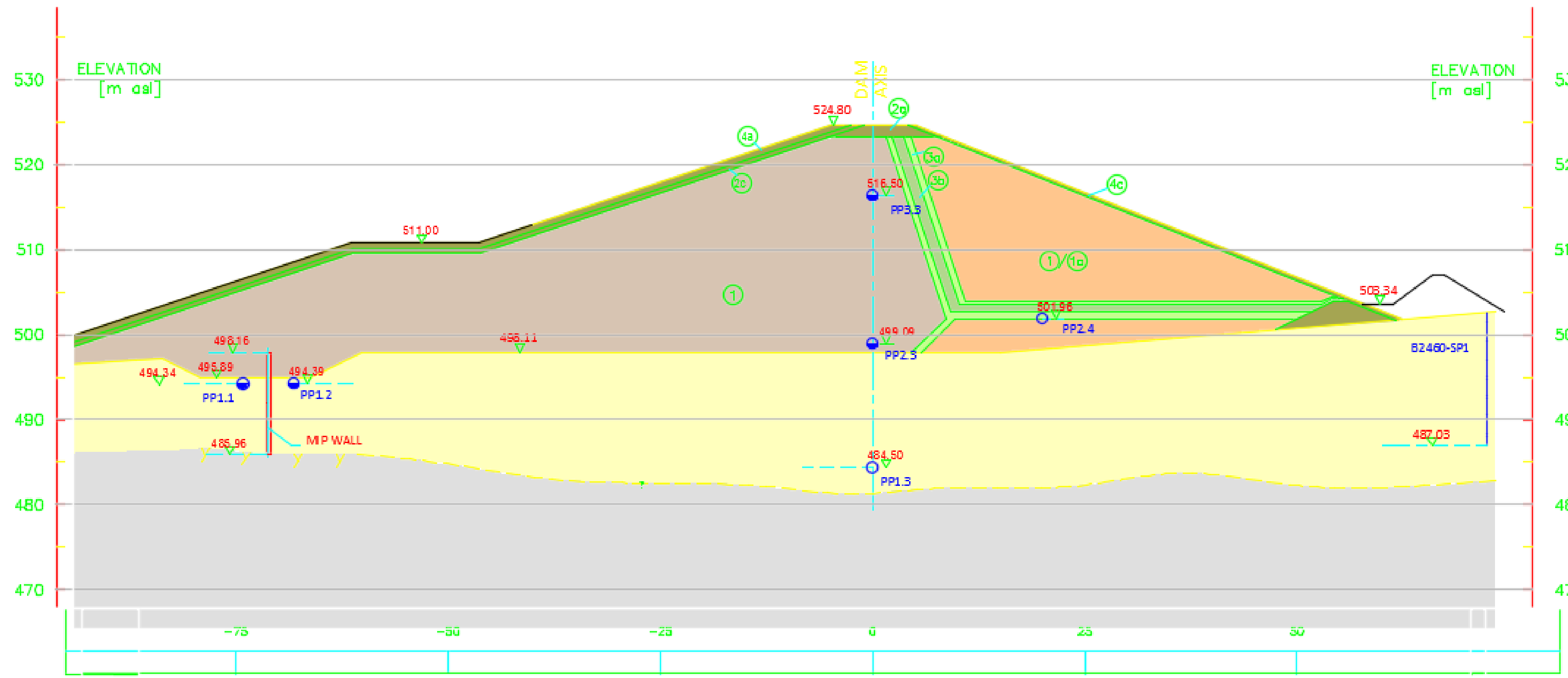


Figure B-12: DCUAP – Section B2+460.

Appendix C : Distribution of instrumentation in RDHP and DCUAP.

Table C-1: Distribution of instrumentation in RDHP embankments.

	Section	VW Pressure Cells	VW Piezometers	Inclinometers	Standpipe Piezometers
Right Bank Embankment	Interface	5	-	-	-
	CH+12.5	-	16	2	-
	CH+217	-	19	2	2
	CH+585	-	17	2	2
	CH+1060	-	17	1	2
	CH+1438	-	17	2	2
	CH+2303	-	14	2	2
	CH+4710	-	10	2	1
Left Bank Embankment	Interface	4	-	-	-
	D+65	-	14	2	1
	D+177	-	21	2	2
	D+752	-	18	2	2
	D+977	-	18	2	2
	D+1475	-	15	1	2
	D+3275	-	14	2	2
	D+7975	-	15	2	2

Table C-2 : Distribution of instrumentation in DCUAP – Rumela Dam embankments.

Embankment Monitoring Section		Pore pressure cell	Magnetic Settlement System	Standpipe Piezometers
Right Bank	R0+160.00	6		1
	R0+500.00	6		1
	R0+850.00	6		1
	R1+117.00	10	1	2
	R1+249.00	10		1
River Section	R1+391.81	7		
	R1+510.00	7		2
	R1+630.00	11	1	1
	R1+710.00	10		
	R1+767.72	10		1
Left Bank	R1+870.00	12		
	R1+892.97	12	1	2
	R2+020.00	8		2
	R2+125.00	9		2
	R2+300.00	6		1

Table C-3 : Distribution of instrumentation in DCUAP – Burdana Dam embankments.

Embankment Monitoring Section		Pore pressure cell	Magnetic Settlement System	Standpipe Piezometers
Left Bank	CH B0+150	6		1
	CH B0+500	6		1
	CH B0+850	6		1
	CH B1+000	7	1	2
	CH B1+120	12	1	2
	CH B1+131	12		2
Dam River Section	CH B1+292	10		1
	CH B1+376	10		1
	CH B1+460	11	1	1
	CH B1+560	7		2
	CH B1+650	7		
	CH B1+870	7		1
Right Bank	CH B2+050	6		1
	CH B2+350	6	1	1
	CH B2+462.03	6	1	1
	CH B2+600	6	1	1
	CH B2+900	6		1
	CH B3+300	6		1

Appendix D : Locations of Pressure Cells in RDHP.

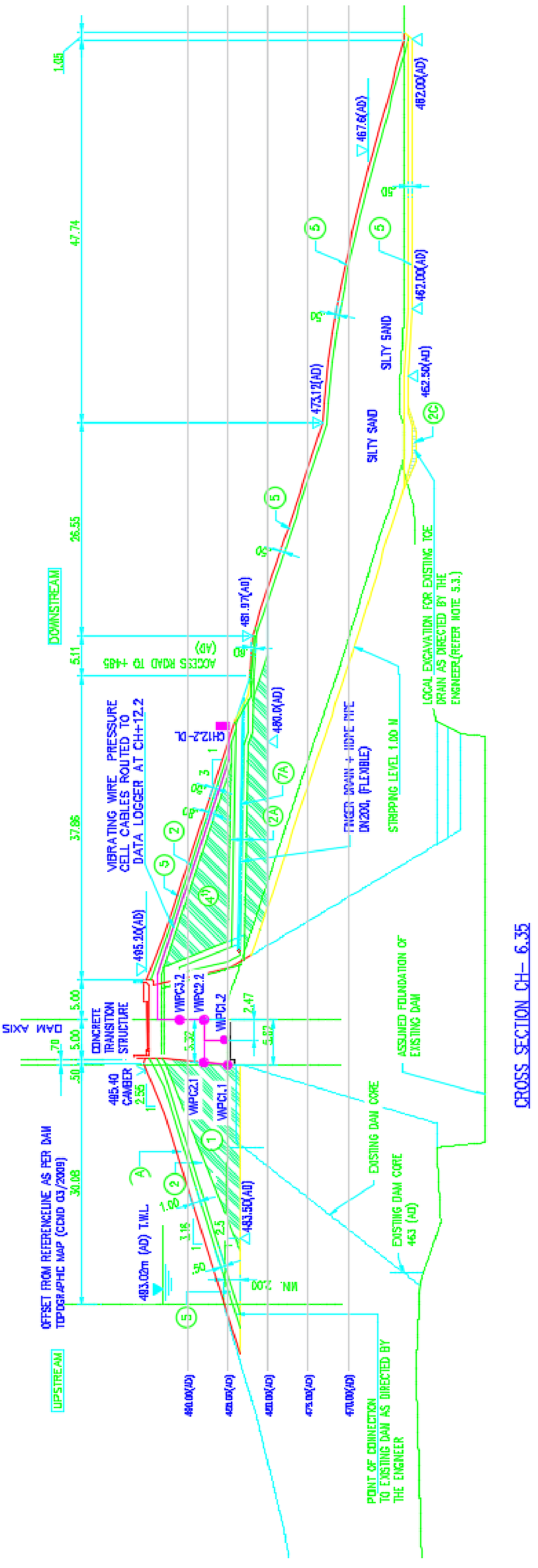


Figure D-1: RDHP - Locations of Pressure Cells in Section CH-6.35 in the Right Bank.

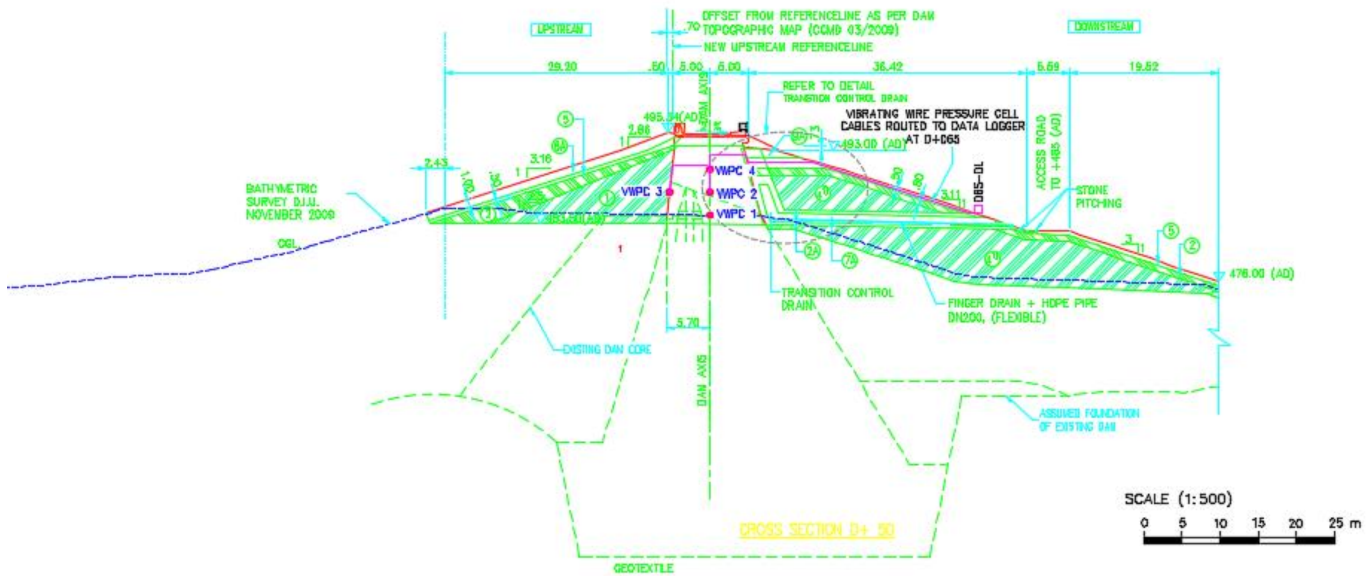


Figure D-2: RDHP - Locations of Pressure Cells in Section D+50 in the Left Bank.

Appendix E : Layouts of seepage locations in DCUAP.

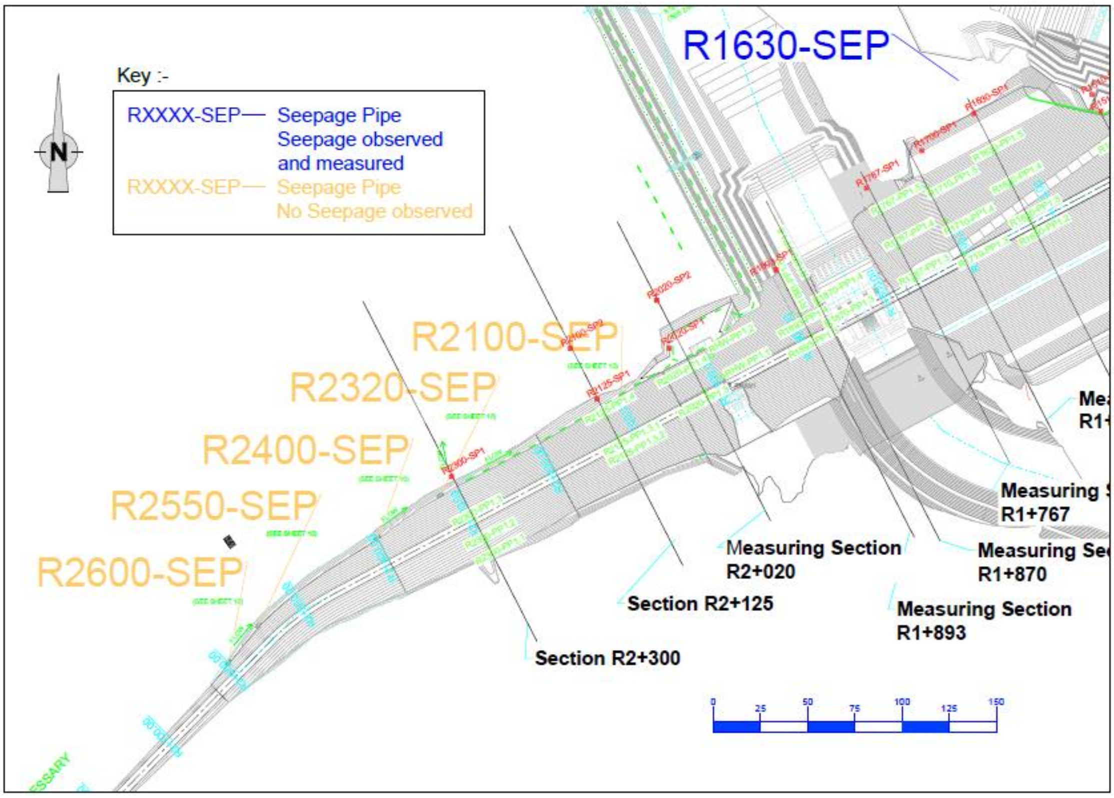


Figure E-1: Layout of seepage locations in Rumela Left Bank and River Section.

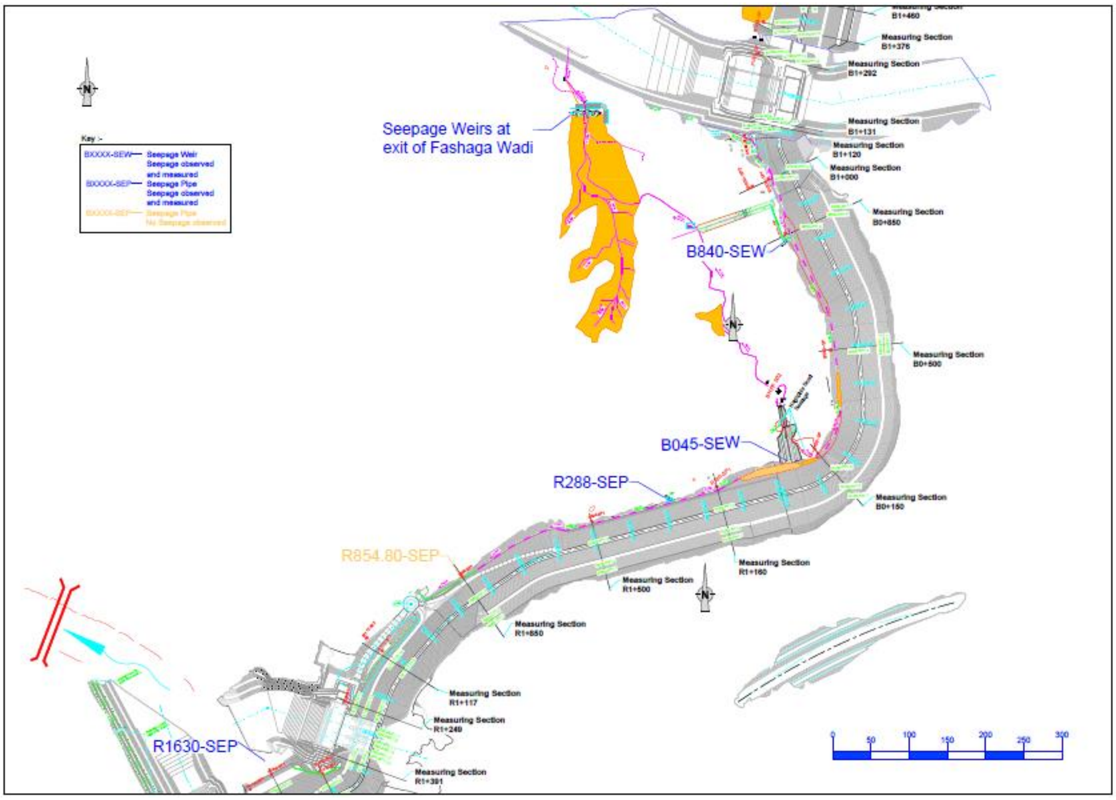


Figure E-2: Layout of seepage locations in Fashaga area.

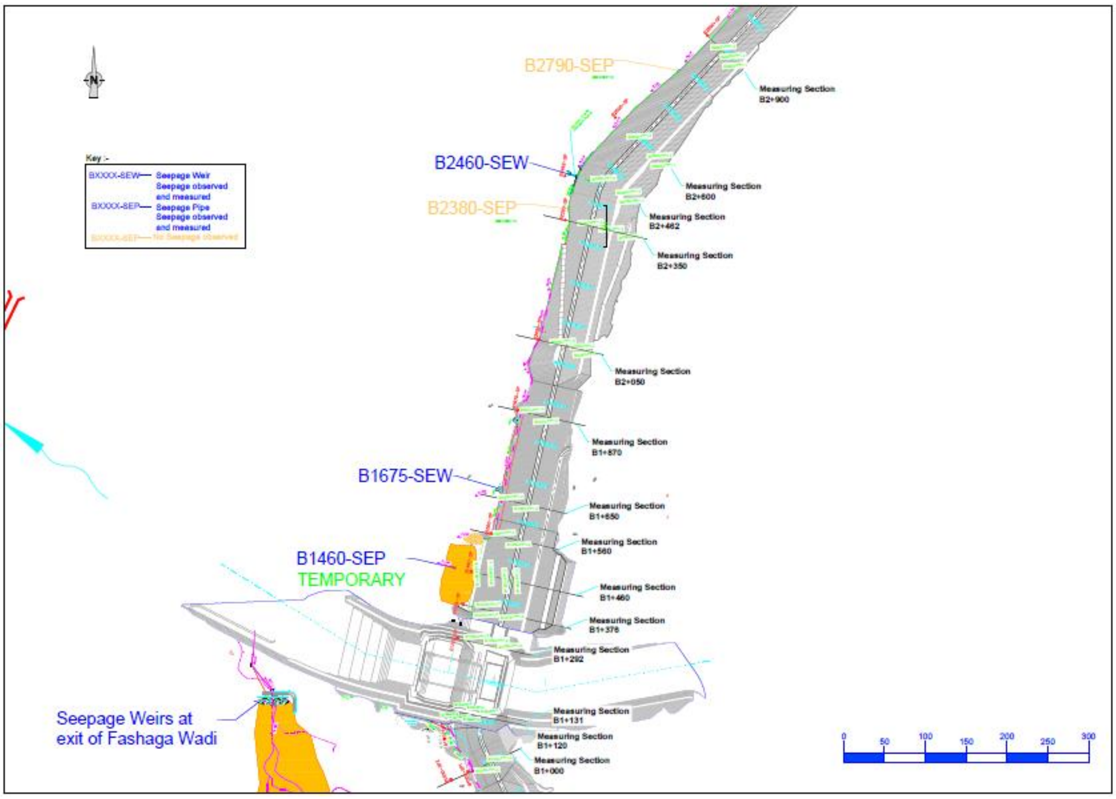


Figure E-3: Layout of seepage locations in Burdana River Section and Right Bank.