Masjed-e-Soleiman Dam instrumentation

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SYNOPSIS. The 187m high Masjed-e-Soleiman clay core rockfill dam forms part of a 1,000MW hydropower scheme. The dam was constructed between 1996 and 2001 and impounding commenced in late 2000. The instrumentation installed at the dam was designed to meet international guidelines for the primary purposes of monitoring construction, impounding and long term performance. The instrumentation comprises earth pressure cells, extensiometers, piezometers, groundwater observation holes, survey monuments and seismic monitors. In the event a significant proportion of the buried instrumentation failed during construction. A study was undertaken to review the performance of the instrumentation, evaluate available monitoring data and develop the criteria for control of the impounding.

INTRODUCTION

Masjed-e-Soleiman is a clay core rock fill dam situated in a narrow gorge on the lower reaches of the Karun River in southwestern Iran. The Karun River rises in the Zagros Mountains in western Iran and flows southward to the Persian Gulf. A number of dams are constructed along the Karun River and more are planned as shown in Figure 1.

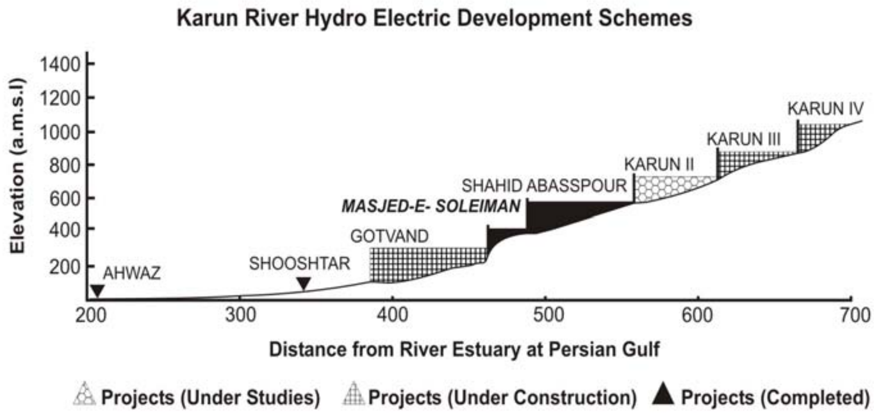

Figure 1: Karun River Cascade Development

Long-term benefits and performance of dams. Thomas Telford, London, 2004

The catchment area of Masjed-e-Soleiman reservoir is 27,550km^2. The reservoir has a total storage of 285 million m^3 and a live storage of 90 million m^3 between elevations 363m and 380m. Peak flood inflows for the 1,000 year and 100 year floods are 9,300 m^3/s and 6,800 m^3/s respectively.

The purpose of the dam is to provide river regulation and storage for hydropower generation. Clay core rockfill construction was favored because of the relative seismicity of the area and the availability of suitable clay and rockfill materials nearby.

The dam comprises a rockfill embankment with clay core and upstream and downstream filters. The upstream slope of the dam is at 1^V to 2^H and incorporates a rockfill cofferdam with upstream clay membrane at its toe. The overall downstream slope is at 1^V to 1.8^H and incorporates an access roadway for construction. The clay core of the dam has a minimum width at the crest of 10m increasing in width by 0.4m for every 1m below crest elevation. Each of the filters has a nominal width of 5m. A typical section through the dam is shown in Figure 2.

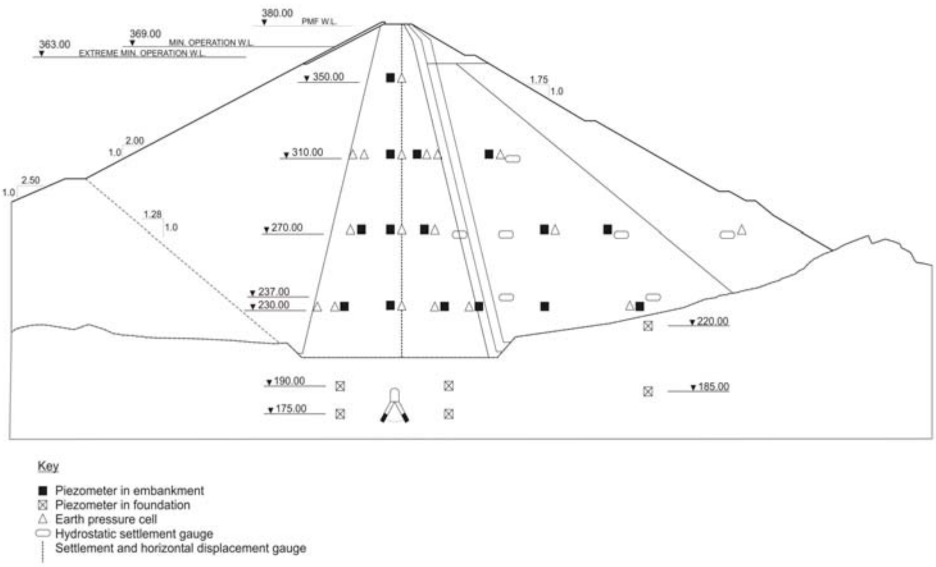

Figure 2: Typical Instrumentation Arrangement at Masjed-e-Soleiman

The clay core was placed at approximately 2% wet of optimum moisture content of 14.2% and compacted densities of 98.5% of maximum were maintained. The core material has a plasticity index of 19.9% and a permeability of between 10^{-8} and 10^{-9} m/s.

The dam foundation comprises alternating layers of permeable sandstone and impermeable claystone with a dip toward the upstream. In general the grout take for the cutoff curtain was low, with the only area of high grout take being on the left abutment between the upper and lower galleries. For this reason the arrangements of the groundwater observation holes in the galleries were amended to give clear indications of seepages in this area.

The site experiences temperature extremes ranging between +55^{o}C and 0^{o}C. The high temperatures through July and August were particularly disruptive to construction and made it difficult to control the moisture content of the clay fill.

INSTRUMENTATION

The primary objectives of the instrumentation and monitoring systems can be summarized as follows:

- To confirm the design assumptions and predictions of performance at the construction phase
- To monitor performance of the embankment during the impounding of the reservoir
- To confirm safe operation through the life of the dam including the provision of early warning of the development of unsafe trends in behaviour
- To verify the safe aging of the structure

The instrumentation installed, at Masjed-e-Soleiman was evaluated against international guidelines for dam instrumentation including the ICOLD Bulletin No 60 and the US Army Corps of Engineers manual EM 1110-2-1908. In general it would be considered a well instrumented dam if compared to other rockfill dams of a similar size worldwide. Both guidelines promote the monitoring of ground water, pore pressure, movements, deformation and fill pressure whilst recognizing that every instrument system is unique and that a significant amount of engineering judgment must be applied.

The instrumentation installed in the Masjed-e-Soleiman embankment comprises the following equipment:

Foundation Peizometers	Standpipe Piezometers
Embankment Piezometers	Casagrande Piezometers
Earth Pressure Cells	Groundwater Observations Holes
Hydrostatic Settlement Gauges	Seepage measuring weir
Settlement Inclinometers	Earthquake Accelerometers
Surface survey monuments	

The instrumentation for the dam was intended to assist in the evaluation of the performance relating to the following areas:

- Seepage and leakage
- Deformation due to
    - Slope instability
    - Settlement due to internal erosion
    - Consolidation of fill
    - Consolidation of foundation strata
    - Secondary consolidation of fill and foundation
    - Volume change in clay
    - Changes in reservoir levels
- Seismic disturbance

INSTALLATION & INSTRUMENT FAILURES

During the construction of the embankment a significant number of the instruments were damaged or became inoperable. The reasons for the malfunctions included incorrect installation, damage by construction plant, use of incorrect equipment and faulty instruments. Table 1 sets out details of the instrumentation installed and their operational condition at impounding.

The instrumentation for the embankment was largely installed as the filling progressed. The foundation piezometers were installed in boreholes prior to starting the dam filling.

Hydraulic type peizometers were installed in the downstream shoulder together with hydrostatic type settlement gauges. These instruments could not be used to monitor the construction of the dam because the instrument houses could not be constructed until construction of the embankment was complete. This meant that potentially beneficial monitoring data could not be used to analyse the performance of the dam until after impounding.

There were three types of instrument, which showed significant rates of failure. These were the vibrating wire foundation piezometers, the earth pressure gauges and the settlement inclinometers, each with failure rates well outside of the range that would be expected from equipment malfunction, given careful installation.

Table 1 – Operational Instrumentation at the end of construction

Type	Location	Installed No.	Operational No.	Defective
Peizometers – foundation - vibrating wire	Core D/S fill	14 5	6 4	57% 20%
Piezometers -dam – vibrating wire	Core	25	18	28%
Pore pressure –dam – hydraulic type	D/S fill	10	Unable to monitor	
Standpipe piezometers	Abutments / galleries	22	22	
Casagrande piezometers	D/S toe	3	3	
Groundwater observation holes	Abutments	15	15	
Earth pressure gauges	Core D/S fill	39 9	30 8	23% 11%
Hydrostatic settlement gauge	D/S fill	13	Unable to monitor	
Settlement inclinometers	Core D/S fill	4 (519m) 4 (288m)	4 (340m) 4 (263m)	35% 9%
Earthquake accelerometer	Crest/Core/ Gallery/face	1/2 1/1	Installed later	

Foundation piezometers

The vibrating wire foundation piezometers monitoring data is of importance at construction stage to evaluate the stability of the foundation under the loading imposed by the dam. This is particularly important when construction is rapid and pore pressures do not have time to dissipate. With the majority of the piezomters upstream of the grout curtain inoperable, the ability to evaluate pore pressure distribution across the grout curtain at impounding was compromised. It was not considered advisable to proceed to impounding without establishing a method of monitoring foundation pore pressures upstream and downstream of the grout curtain.

A series of vibrating wire piezometers were retrofitted from the foundation grouting gallery to replace those piezometers, which were inoperable. These piezometers were installed by drilling inclined upward holes from the gallery to position a piezometer tip close to the position of the inoperable piezometers. The installation of these piezometers went well and presented no problems. The proposed piezometers were installed before impounding and gave plausible readings.

Earth pressure gauges

The earth pressure gauges provided important information relating to the build-up of earth stresses as the fill progressed. This information was used to monitor embankment stability during the construction period when pore pressures were particularly high and effective stresses low. Fortunately, a sufficient number of the instruments remained operational to allow the stability of the dam to be assessed. The importance of the monitoring data from the earth pressure gauges reduces as the fill continues to settle and construction pore pressures dissipate thereby increasing the effective stress.

Although there are earth pressure gauges available that could have been installed in a borehole from the surface, it was considered that, these were difficult to install and because they were relatively small their effectiveness would be limited. It was decided that it was not cost effective nor technically beneficial to install additional earth pressure gauges.

Settlement inclinometers

The settlement inclinometers would normally provide useful monitoring data on the distribution of settlement within the body of the embankment. The settlement inclinometers consist of inclinometer tubes installed with magnetic ring plates fixed to the outside of the tube at regular intervals. The ring plates are arranged to ensure that they settle along with the fill thus compressing the tube system. A probe is then lowered down the tube to record the relative positions of the magnetic rings. The tubes can also be used as traditional inclinometers to monitor horizontal displacements in any direction. In the event all inclinometers below EL 290m became inoperable and therefore it was not possible to assess the consolidation of the fill. Although there were hydrostatic settlement gauges below this level in the downstream shoulder these instruments could not be read until after impounding when the permanent instrument houses were completed.

It would have been possible to drill boreholes and to install settlement inclinometers in a borehole but this is generally only done for shallow holes. To reinstate the inclinometers in the dam core below EL 290m would have necessitated drilling holes to a depth of 170m through the clay. Such drilling would be very expensive and particularly disruptive to the core itself.

Failures

The reasons for the failure of the foundation and core piezometers and earth pressure cells could not be established from an analysis of the records. Given that in general it was the instruments in the upstream shoulder and in the core which had failed it was considered likely that the reason for loss of readings was due to damage of the connections from the instruments to the monitoring points on the downstream face. The cables and tubing had been laid in sand filled trench across the filter and shoulders and at the transitions between core, filters and shoulder the cable had been ‘snaked’ in the trench to allow for drawing out due to differential settlement. It was concluded that the friction on the cabling or tubing as the fill continued was too great to allow any movement. This in turn led to high stresses in the cable or tubing and failure at the interfaces where differential settlement occurred.

The reason for the failure of the settlement inclinometers was easier to determine when photographic records were studied and a failed section uncovered. The inclinometer tubes had been joined by use of an outer sleeve as per manufacturers instructions but the installer had failed to leave a gap between the tube sections to allow for telescoping of the tubes under consolidation of the fill. Subsequent settlement caused buckling of the tubes at the joints, which meant that it was not possible to pass the probe down the tube.

EVALUATION OF EMBANKMENT

The instrumentation suite was designed with particular interest in monitoring pore pressures, stress and deformation in the clay core. This was to enable an assessment to be made of the transfer of total stress to effective stress as pore pressures dissipated allowing consolidation to occur in the form of vertical displacement.

Before impounding could proceed it was necessary to demonstrate that the embankment section was stable and that there would be no safety problems associated with the excess pore water pressures that were being experienced. On completion of the embankment construction it was found that pore water pressures in the core were particularly high and that even at the lower elevations there had been very little dissipation of pressure. This meant that with low effective stresses the shear strength of the clay core was also reduced. Figure 3 shows the recorded pore pressure ratio r_u values on the highest section of the dam at the end of construction in 2000, despite the fact that construction at the foundation level had commenced in 1997. It was estimated that it would take a further 1-2 years for the pore pressures to dissipate significantly.

For this reason it was necessary to evaluate whether the reservoir could be impounded immediately after completion of construction and if so at what rate the water level could be raised.

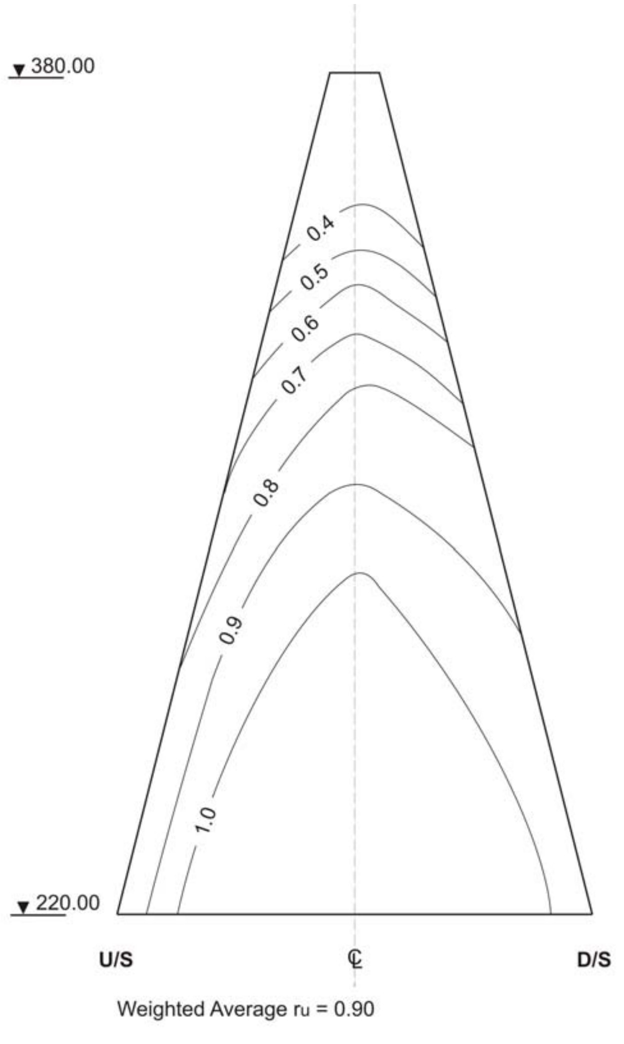

Figure 3: r_u values for the dam core at completion of construction

Of particular concern was the stability of the upstream face of the embankment for the condition of embankment at full height and also for initial impoundment. The slope stability issue was exacerbated by a layer of alluvium under the cofferdam forming the upstream toe, which had a potentially lower angle of shearing resistance. This layer had a thickness of up to 8m and had not been stripped from the foundation when the cofferdam was constructed. The uncompacted toe layer when coupled with the high r_u values in the core meant that it was possible to generate a non-circular slip failure surface through the core and upstream toe foundation, which was barely above unity for the condition of unregulated impounding. A separate study showed that provided the impounding rate was controlled to allow pore pressure dissipation in the core then the short-term factor of safety for impounding could be maintained above 1.3.

The need to make a controlled impounding became more pronounced shortly after embankment construction was completed as the single operable diversion tunnel suffered major damage to the concrete lining when the other diversion tunnel was taken out of service for conversion to a bottom outlet. The situation that developed meant that unless the remaining diversion tunnel could be closed to allow repairs the tunnel would erode further and lead to collapse. It was against this scenario that a balanced impounding procedure was developed. The impounding procedure allowed for initial impounding to EL 303m whilst the bottom outlet was brought into service and then for the water level to be held at this level using the bottom outlet, with its inlet sill at EL 300m, for a period of 3 months to allow the embankment pore pressures to continue to dissipate before filling continued to spillway sill level at EL 350m at a target rate of 0.5m/day.

The early impoundment was complicated by the fact that the upstream shoulder had a clay upstream face to elevation EL 300m. Therefore water ingress into the upstream shoulder was limited to the seepage through the clay membrane until such time as it was overtopped at EL 300m. For this reason the rate of rise immediately above EL 300m was very gradual to allow slow filling of the shoulder and stabilization.

The operational earth pressure cells show a pronounced variation in earth pressure across the core, filters and downstream fill at a number of sections. A typical effective stress distribution has been presented in Figure 4 and represents a series of six working cells. Figure 4 compares the actual minimum effective stresses against the theoretical total stress, calculated as the overburden pressure, and a lower limit of 70% theoretical, which was set as the stable limit for impounding by the designers. This represents a section towards the highest part of the dam at EL 310m some 70m below crest level and is at the time of completion of the embankment filling. Figure 4 clearly shows that the minimum effective stress in the core was considerably less than that in the filters and the shoulders. It was apparent that with effective stresses in the core of only 60% of theoretical overburden stress the core was effectively hanging up on the shoulders which themselves were being crushed with stresses up to 130% of theoretical.

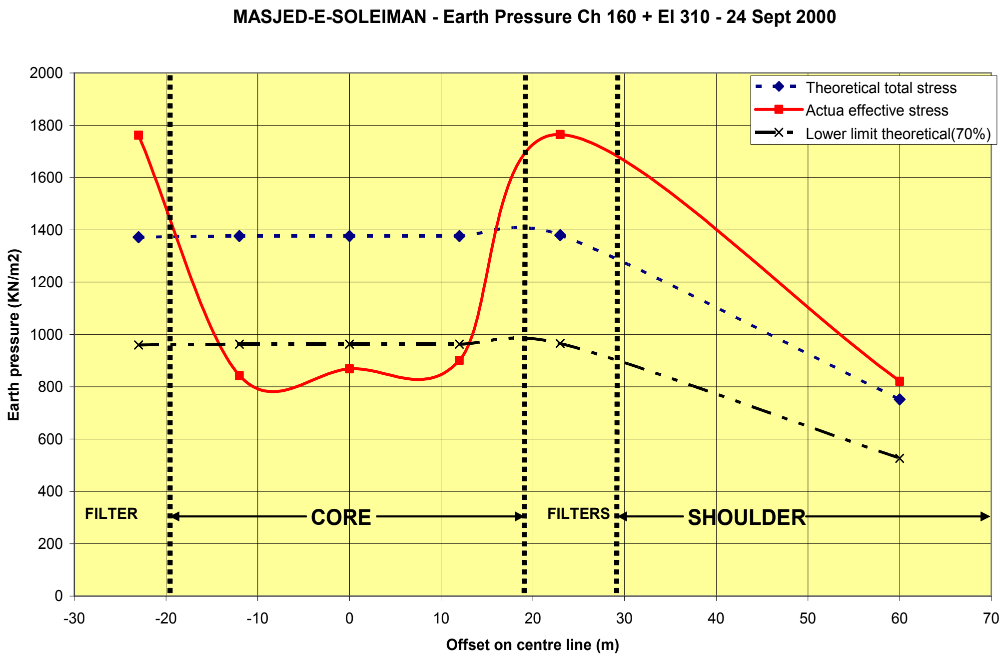

Figure 4: Distribution of effective stress across core and filters

The principal concern with the clay core was that on impounding the hydrostatic pressure of the water in the upstream shoulder would exceed the effective stress in the core material leading to potential fracture. For this reason it was decided to regulate the impounding to ensure that the hydrostatic pressures from the rising water level were not allowed to exceed 80% of the actual recorded minimum effective stress at that level in the core. To achieve this it was necessary to monitor increases in water level and the actual earth pressures as consolidation and pore water pressure dissipation in the core continued. Although it was recognized that the filters may have crushed in the zone against the core it was considered that they would continue to function as an effective filter in the event of leakage through the core.

There was a risk that if there was a significant flood event then it would not be possible to control the rate of impounding with the 330 m^3/s capacity bottom outlet and that this would result in uncontrolled reservoir rising to gated spillway sill at EL 350m. With Shahid Abasspour dam upstream the risk of uncontrolled flooding was mitigated somewhat as it was possible to create nearly 400 million m^3 of storage in the reservoir, under the operational rule curve to attenuate a flood. This meant that the risk of uncontrolled flooding through the winter/spring impounding was reduced to a 1 in 10 year event.

CONCLUSIONS

It was concluded that whilst the instrumentation at Masjed-e-Soleiman was consistent with current international practice the number of instrument failures were significantly higher than would be normally expected. In particular the failure of the vibrating wire piezometers, earth pressure gauges and settlement inclinometers make thorough analysis of the embankment difficult. This demonstrated the importance of adopting good installation procedures.

The analysis showed that the pore pressures within the core were just within acceptable limits but were dissipating far slower than had originally been envisaged. Because of the possible risk of arching and hydro-fracture of the clay core it was concluded that the impounding of the reservoir should only be made under strictly controlled conditions to prevent excess hydrostatic pressures in the upstream shoulder. The impounding was staged to allow dissipation of pore pressures in the core to ensure that at no stage would the hydrostatic pressure in the upstream shoulder exceed the minimum effective stress in the core.

There remained a slight risk that uncontrolled impounding would occur under a flood event but this was mitigated against by using the upstream reservoir to provide storage.

The instrumentation provided sufficient data to determine the behavior in terms of pore pressure and soil pressure but there was no effective measurement of settlement / consolidation of the fill due to the failure of the settlement inclinometers and the inability to use the hydrostatic settlement cells until after construction was complete. This was because the hydrostatic instruments require that the instrument houses are constructed at close the elevation of the instrument.

In the event the impounding went well and in accordance to the criteria set out. By early 2003 pore pressures were continuing to dissipate slowly and seepages remained negligible.

ACKNOWLEDGEMENTS

The author would like to acknowledge the valuable advice and assistance given by Iran Water and Power Resources Development Corporation (IWPC) site staff, Nippon Koei Ltd design staff and instrumentation monitoring staff with Daelim Industrial Co Ltd.

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