FIELD MONITORING AS A TOOL FOR GROUND IMPROVEMENT PERFORMANCE CONTROL

Mahdi Moosavi, Mining Engineering Department, The University of Tehran, Iran

Mostafa Pasha Nejati, Senior Engineer, Payab Zamzam Consulting Engineers, Iran

Abstract:

Careful monitoring program at one underground cavern and one road tunnel enabled determination of hazardous areas and performed as a reliable controlling tool to check the adequacy of the additional support program. At Masjed Soleiman powerhouse cavern, preliminary support design at centerline and downstream side proved to be not enough to maintain the integrity and stability of the rock mass. Adding more ground anchors confirmed to be enough for retaining the rock mass stability by stopping the increasing trend of rock mass displacements. Increasing trend of tunnel wall displacements was also alarming at Pardis traffic tunnels when additional support installation stopped the rock mass movement and maintained stability.

Key words: Instrumentation, Monitoring, Caverns, Powerhouse, Reinforcement

1. Introduction

Most of underground structures are built in grounds where without some means of improvement can not maintain their stability. This is also true for large span tunnels built in soft grounds. These reinforcement operations range widely from compaction, drainage and grouting of soft soils to installation of fully grouted bolts and applying shotcrete in jointed and blocky rock masses. The amount of ground improvement measures which is really adequate for each job is a challenging question in front of the engineers. A heavy reinforcing system can assure safety but is often not economical. On the other hand, insufficient measures will not guarantee the safety of the structures therefore having an accurate ground improvement program is always a controversial question to be answered by engineers.

In recent decades with advances in other branches of science and technology, specially electronics and mechanics, very accurate instruments are developed which are frequently used in geomechnical projects. These instruments have helped a lot to determine the exact performance of underground excavations as well as the reinforcement and improvement aids. With continuous monitoring of the whole system, one can pin point the problematic locations and quantitatively determine the sufficient improvement procedures to provide the required safety factor.

This paper has focused on two examples in Iran which have benefited from careful monitoring programs. This will show how a continuous interaction between obtained data from the monitoring team and prompt response by the engineer and contractor teams has helped to prohibit instability problems by employing an economical improvement program.

2. Case study 1 - Masjed-Soleiman Hydroelectric Power Plant

Masjed-Soleiman Hydroelectric Power Plant (HEPP) is built in two phases with total capacity of 2000MW on Karun River in Iran. The rock mass consists of compacted Conglomerates, Siltstone, Snadstone and Claystone layers crossed by widely spaced rock joints. The dimensions of the powerhouse cavern are 266m in length, 30m in width and 50 m in height. Transformer cavern is located to the right side of the powerhouse cavern at a higher elevation as shown in figure 1. The layers dip at about 25 degrees towards the upstream side. Day lighting Claystone (with low frictional and mechanical properties) in the roof and the walls of the caverns has resulted in some instability problems. The focus of this paper is on the centerline and downstream side of the cavern roof where Claystone outcrops.

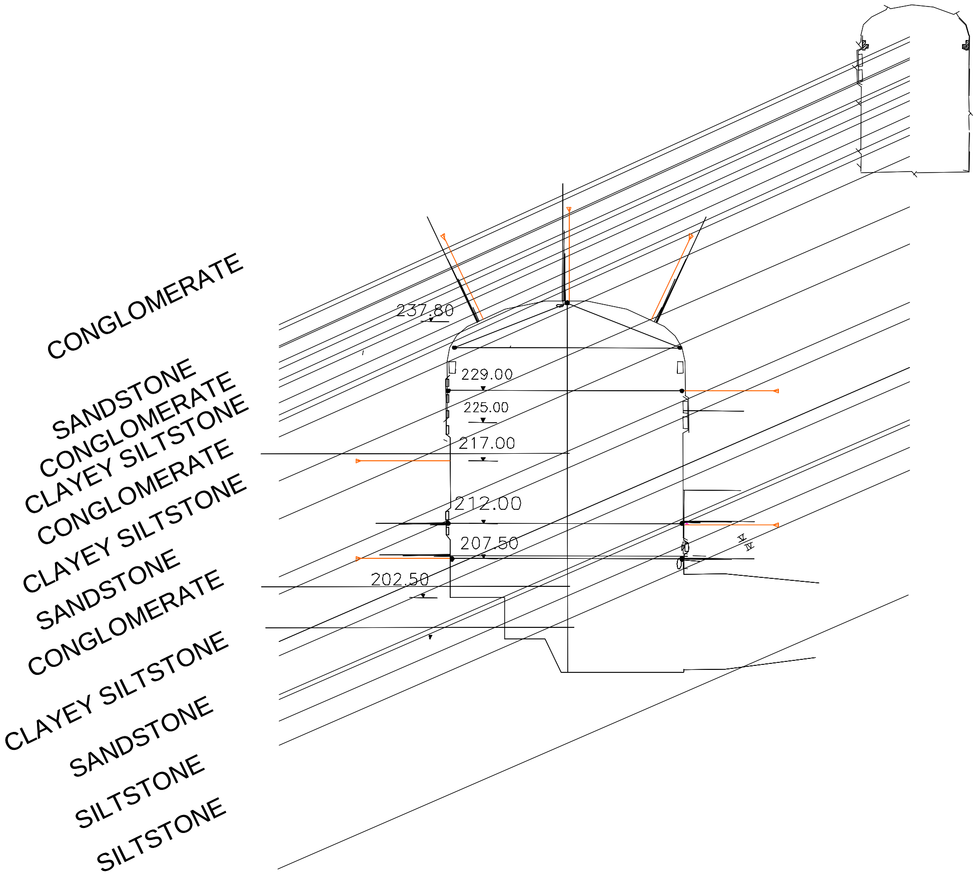


Figure 1: Underground power station scheme and rock mass geology.

2.1. Geomechanical properties of the rock mass

Based on performed laboratory and field tests the following rock mass parameters are reported by Stabel [1]. Low mechanical properties of the Claystone rock, especially when absorbing water, results in further drop in the values and results in roof instability.

Table 1: Rock type and their mechanical properties at site.

Rock type	Young's Modulus (MPa)	Poisson's ratio	Cohesion (MPa)	Friction angle (Degree)
Conglomerate	15	0.2	2.28	43
Sandstone	7	0.2	1.67	38
Claystone	6	0.25	0.73	30

Excessive rock mass displacement which causes shotcrete cracking and bolt failure can be seen in some parts of the roof especially in downstream side of the powerhouse cavern.

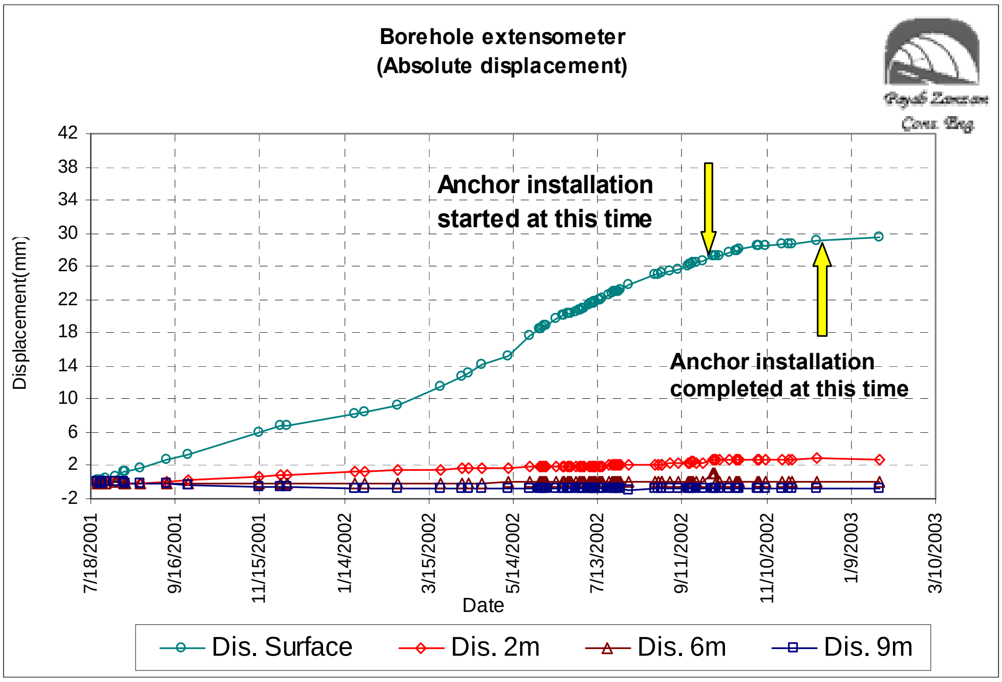
2.2. Monitoring program in the project

To control the stability of the cavern during excavation stages, a comprehensive instrumentation program was proposed and implemented consisting of 200 instruments including borehole extensometers, load cells and pressure cells distributed in the caverns according to table 2.

Table 2: Monitoring stations and type of instruments in the powerhouse cavern.

Chainage of the monitoring stations (measured from the beginning of the cavern)							Instrument type
Powerhouse cavern	8	25	43	71	93	107	4 point, 30m/15m long borehole extensometers, 50 t and 200 t load cells and 2 MPa pressure cells
Transformer cavern	0	28	58	78	100	10	4 point, 15m long borehole extensometers, 50 t load cells and 2 MPa pressure cells

An example of the monitoring results at chainage 71m in centerline is depicted in figure 2. The upper graph is rock mass displacement recorded by borehole extensometer and the lower graph is the adjacent load cell result. As shown in figure 2 the rock mass has kept moving although the original reinforcement program had fully been applied according to the design. As noted in figure 1, this extra amount of rock movement in centerline and downstream is due to the presence of soft Claystone layer with potential of swelling when absorbing water. However, at the upstream side, the rock mass movement is less and in many cases the rock has stopped moving after a while.



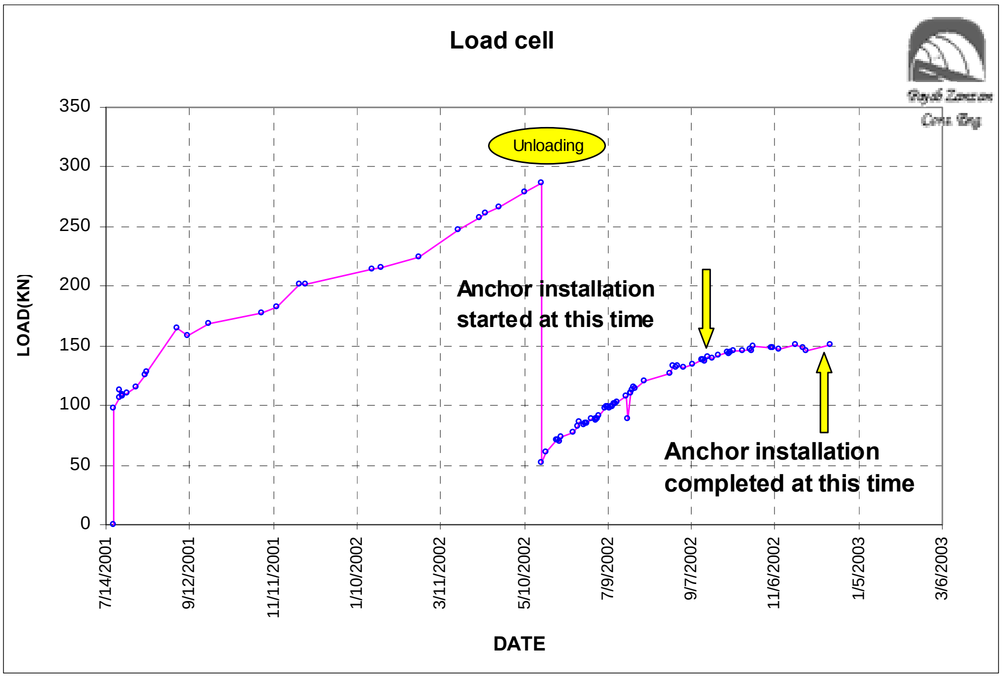


Figure 2: Displacement (upper) and load increase (lower) at chainage 71m, centerline.

Increasing trend of the rock movement has resulted in an increasing trend of load in the load cells. At a stage, the load in the bolts had increased to a level very close to the yield capacity of the bolts. The bolt containing this load cell was unloaded and the load cell was installed again to be able to detect further increase in bolt load. This increasing trend of displacement and load in centerline and downstream side together with a local rockfall, alarmed the engineer that more support pressure than what was anticipated earlier is required to assure long term stability of the cavern.

The additional support program consisted of 15 and 20 meters long tendons (Double Corrosion Protected, DCP) with 60 ton working capacity. This system was gradually applied to the whole roof of the powerhouse cavern as shown in figure 3. Start and the end of ground anchor installation in the vicinity of the extensometer at chainage 71m is shown by arrows on the previous figures. This remedial work stopped the increasing trend of rock movement gradually according to this figure. What happened at the roof in terms of rock pressure and reinforcement pressure is summarized in table 4. To calculate the roof and support pressure, the suggestions of Barton [2] and Hoek [3] are used respectively.

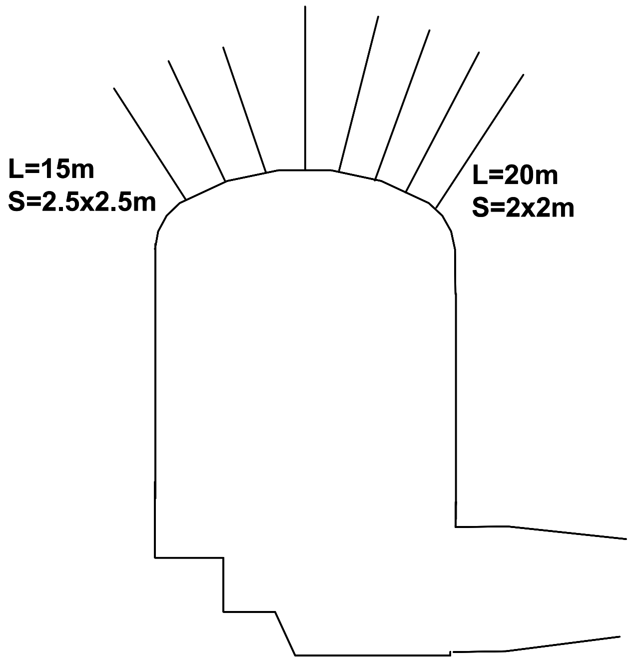


Figure 3: Schematic of additional support measures in powerhouse cavern.

Table 4: Summary of the roof and support pressure before and after additional reinforcement.

Reinforcement system in the powerhouse cavern		**Roof pressure (by Q)**	**Reinforcement pressure (MPa)**	
			Before additional support	**After additional support**
Powerhouse cavern, **D/S** and **CL**	ϕ 28mm, L=6 &12 m, spacing 1.75x1.75, Tendon ϕ 40mm, L=20m and 2x2m spacing	0.225	0.192	0.288

As can be seen from table 4, the support pressure from reinforcements before adding complementary reinforcement (0.192 MPa) is less than what is applied by the roof at centreline and downstream side (0.225 MPa) which explains increasing trend of the recorded rock mass displacements by the instruments. After applying extra pressure by additional support, the reinforcement pressure becomes more than the roof pressure (0.288 MPa) and results in roof stability. The initial support pressure in the upstream side was originally more than the roof pressure which explains the decreasing trend of rock movements in these parts. The applied additional support was not necessary for upstream, but to increase the safety factor.

3. Case study 2 – Pardis traffic tunnels

Pardis highway is designed and executed to relief the traffic for the Eastern part of Tehran city. It consists of twin tunnels 15 meters in width each, locates 20 meters apart. The rock mass includes uncompacted Conglomerates. The depth of overburden ranges from 20 to 50 meters which shows the possibility of surface water ingress into the tunnel through weak and faulty zones. The tunnel is excavated in two stages. The top heading is excavated first (figure 4) which will follow with benching operations at later stages. Excavations are performed using roadheader continuous machines which are quite suitable for the mentioned ground conditions.

Figure 4: Left tunnel exit (top heading excavation stage)..

3.1. Geomechanical properties of the rock mass

The formation consists of uncompacted young Conglomerates known as Hezar Dareh Formation. This is a soft poorly graded Conglomerate with clay bearing lenses which when exposed to water can be easily unstable. The Young's modulus of the ground is about 1500 MPa which is very deformable under external loads. Being very close to Alborz Chain Mountains, the site is classified as active and many small size and moderately long faults cross the project line. The crossing of faults, which are usually filled with soft clay materials, proved to be problematic zones while tunnelling.

3.2. Support system of the project

Support system at Pardis tunnels include a combination of lattice girder support and shotcrete plus mesh. There are two types of lattice girder which differ in size and shape. Triangular ones are used for lower capacity requirements and square shape ones for more heavy duty situations. The shotcrete is also applied with 5 centimetres thickness in some cases and increase to 10 centimetres wherever is suggested by the engineer. The shotcrete is reinforced by welded mesh. Due to high content of clay in the formation and low integrity of the ground, application of rock bolt was not recognized effective so the main support system includes lattice girder plus mesh and shotcrete.

3.3. Monitoring program in the project

The monitoring program consists of convergence measurement, extensometers and load cells at some points. The monitoring results at chainage 495m in the left tunnel showed an increasing trend of tunnel convergence as shown in Figure 5. The report showed the necessity of increasing the amount of support measures to maintain the tunnel stability. The increasing trend of displacement believed to be due to the increasing percentage of clay materials in the formation and ingress of water through a small valley on the surface. The amount of support was quickly increased by the engineer and concrete base plates were installed under the lattice girder arches to increase the bearing capacity of the floor so that it can resist against the arch penetration. The amount of shotcrete was also increased at this station. As a result of these measures, further monitoring records showed decreasing rate of the displacements and sustaining the stability condition.

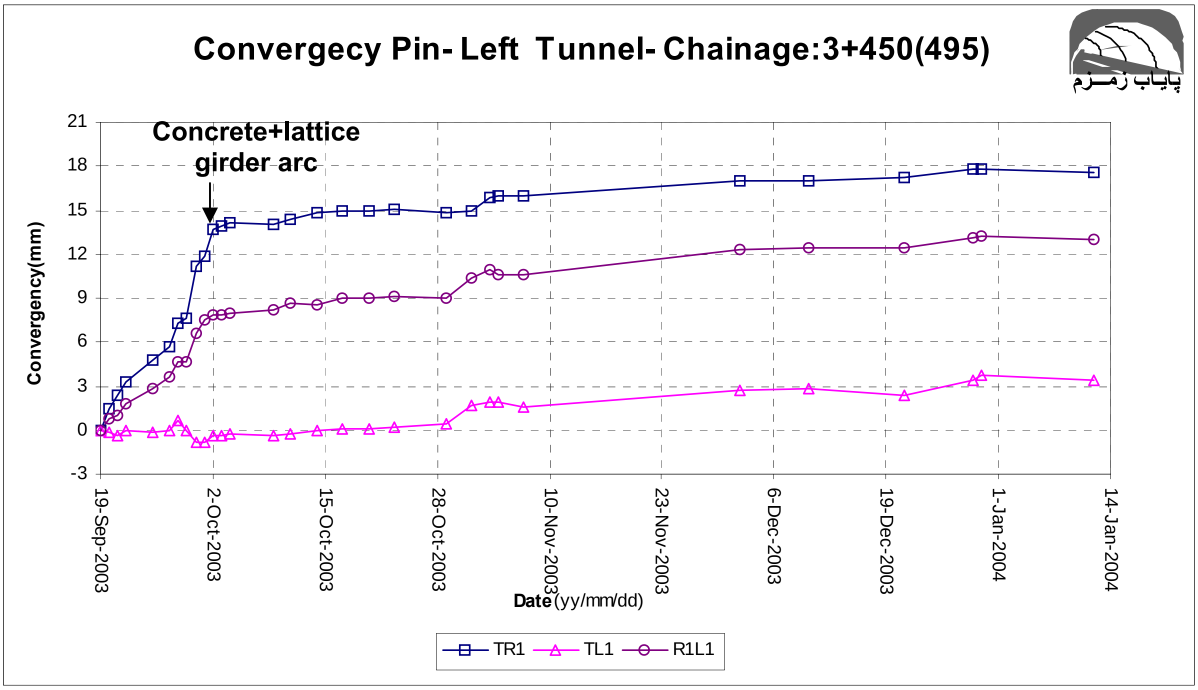


Figure 5: Convergence results for three diagonals at chainage 495, in the left tunnel before and after additional support measures.

4. Conclusions

Two examples are presented, one in a large rock cavern and the other in a shallow twin tunnel in which reliable monitoring programs have played an important role as the controlling tool for monitoring the performance of ground improvement aids to assure stability requirements. The increasing trend of rock mass movements recorded by monitoring instruments showed the necessity of installing additional support. Further monitoring and observing rock movement decrease, proved the adequacy of the additional support measures. Information obtained and experiences gained from a monitoring station can be used as a guide to change or improve the support scheme at the other locations in order to reach an optimum design.

Acknowledgments

The authors express their appreciation to Iranian Water and Power Company (IWPC) and The Ministry of Roads and Transportation, the clients of the two mentioned projects respectively, for their collaboration to execute the monitoring program and presenting the results.

References

[1] Stabel B. Bauckage, “Report on Godar-E-Landar HEPP, Extension Cavern Support”, (2002).

[2] Barton, N., Lien, R. and Lunde, J., (1974), “Engineering classification of rock masses for the design of tunnel support”, Rock Mechanics, 6 (4): 189-236.

[3] Hoek, E., “Support for very weak rock associated with faults and shear zones”, *Proceedings of the Rock Support and Reinforcement Practice in Mining Conference, Rotterdam. Balkema, (1999).*