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Tunnel designing and construction process in difficult ground conditions using Controlled Deformations (ADECO) approach; a Case Study

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The 10.6 km long Sabzkuh water conveyance tunnel is currently under construction in the High Zagros structural zone, located in western Iran, through conventional (in section T1) and mechanized (in section T2) methods. The section T1 of the tunnel driven through alluvial geological units with a great complexity in an area of highly potential for sliding and water-bearing fault which has made the excavation operation encounter many challenges. After excavating 35 m of the tunnel using conventional methods, geological problems and the inappropriate method of the tunnel excavation resulted in the tunnel collapse and failure in the tunnel inlet portal. After the incidence of the collapse, the Design and Construction Process (DCP) of tunnels were revised by using an Analysis of Controlled Deformations in Rocks and Soils (ADECO-RS) in conjunction with the Sequential Excavation Method (SEM). The ground improvement in the portal and tunnel excavation continued in five phases including survey, diagnosis, therapy, operation, and monitoring. The application of this approach into construction of Part T1 of the tunnel increased the safety and minimized risks and costs, despite geological complexities and hazards. Moreover, the excavation process with appropriate daily advance rate of 1 to 1.5 meters was achieved according to a predicted schedule.

Keywords : ADECO-RS, Daylight collapse, Difficult geological conditions, Design and Construction Process (DCP), Sabzkuh tunnel

1. Introduction

In recent years, tunnel construction has extensively increased in seismic zones with high landslide potentials [1-3]. The construction of tunnels in such areas have always faced some challenges such as support system deformation, instability of the tunnel face, surface subsidence, and failure in the tunnel portal. [4-6].

However, if an informed management with proper knowledge of the excavation environment constructs a tunnel, they analyze the behavior of the underground structure from the beginning, disregarding geological, topographical, and environmental constraints of the designed project [7-9].

This process is carried out in two parts: design and construction. The Design and Construction Process (DCP) consists of Engineering Geology of the site [10] and improvement of the tunnel face in order to monitor and control the deformation to verify the caused deformations using an instrument [11, 12]. This method, which is now being developed as an Analysis of Controlled Deformations in Rock and Soil (ADECO-RS), is able to plan a reliable project in terms of time, cost and safety in all geotechnical and geo-environmental conditions [13].

Deformations can be controlled in four methods: (1) Preconfinement intervention [14-17]; (2) Confinement intervention [18]; (3) Sequential Excavation Method (SEM) [19], and (4) combination of these methods [20].

Preconfinement intervention includes the mechanical precutting using fiberglass nails at the core of the tunnel [21], the umbrella arch method using traditional injection [22] and horizontal jet-grouting in front of the tunnel work [23,24]. Confinement intervention involves application of rock bolts and shotcrete and closing the invert [25]. SEM is used both in controlling the face stability and in reducing the displacement [26]. The number and stages of excavation process change according to the distribution of stress in the environment.

The Sabzkuh water conveyance tunnel with a length of 10617 meters is currently under construction using conventional (350 m, T1 section) and mechanized (10267 m, T2 section) methods in the High Zagros zone, which is near the Main Zagros fault. After construction of the tunnel inlet portal and before starting the tunnel excavation, certain signs of instability were observed in the portal, including crack formation in surface concrete lining of slopes and surface water collection canals, displacement in slope monitoring pins, and tensile cracks in the trench. However, despite these warnings, the possibility for instability and collapse in this area was disregarded. After excavating 35 m in the tunnel portal using full face conventional methods, a daylight collapse occurred in the tunnel and the trench slope majorly failed due to the presence of highly dipping slope layers, poor geotechnical characteristics of soil layers, landslide prone area, poor drainage in the tunnel and slope, and utilizing an inappropriate tunnel excavation method. The failure resulted in the formation of a 22 m wide crater in the ground surface [27]. Handling the collapsed area lasted about seven months and imposed high costs on the project and consequently on the schedule. Having the tunnel collapse and given the difficult and complex geotechnical and geological conditions, the DCP was revised and accordingly, the approach of Analysis of Controlled Deformations in Rocks and Soils (ADECO-RS) [15, 21, 28] was selected due to its successful application experiences. Consequently, the excavation

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process and the improvement of the tunnel portal continued using ADECO-RS combined with the Sequential Excavation Method (SEM). Therefore, ground improvement in the portal area and tunnel excavation continued in five phases: (1) survey, (2) diagnosis, (3) therapy, (4) operation, and (5) monitoring. This technique principally works by controlling the deformations of ground improvement operations or the reinforcement of excavation core-face using cement grouting and drainage techniques surrounding the tunnel profile area and the invert closure. Based on the obtained results and despite the frequent challenges and risks, the excavation process continued safely with minimum risks and costs according to the predicted schedule with an acceptable daily advance rate of 1 to 1.5 meters.

2. Sabzkuh tunnel and site conditions

The Sabzkuh water conveyance tunnel in the Chaharmahal and Bakhtiari province (southwestern Iran) has a length of 10617 m, a slope of 0.001%, an internal diameter of 3.60 m, and an excavation diameter of 4.54 m.

The tunnel construction method is based on mechanized excavation. According to the data obtained from field observations, geotechnical and geophysical studies in the elementary part (T1), it was identified that up to 350m of the tunnel basement is alluvial sediments. Therefore, since the tunnel excavation machine is of double universal shield type and it is not capable of excavating in such areas, the primary tunnel section (T1) was excavated in the conventional method.

The Sabzkuh tunnel route from Sabzkuh to Choghakhor is situated in the Zagros fold-thrust belt, western Iran. The Zagros thrust encompasses the highest Zagros Mountains (i.e. Zardkuh, Oshtoran-kuh and Dena with elevation of 4400, 4170 and 4000 meters, respectively), and thus, it is called High Zagros. In addition, since the lithotypes have been severely fractured by frequent faults, it is called the Crushed Zone, as well (Fig. 1).



Fig. 1. Main fault in Zagros belt of Iran [29].

Along the first 350 m long section of the tunnel, the Solaghan thrust fault with a direction of N130 and a dip angle of 40° – 65° toward northeast disturbs the geological layers. Along its route, the fault has caused Paleozoic and older units as well as clastic-rock units to cover the recent sediments. There are many exposing springs, such as Saki Abad, along this fault. This fault has a number of branches called STF developed deep in the ground toward the surface (Fig. 2). The fault has caused landslides, disturbances, and complexities in the geological layers of the region

The local evidence and derived data from the drilled boreholes at the portal and the conventional excavation part (T1) demonstrated that the geology of the area consists of alluviums and debris units with a mixture of sand, clay, and silt. Therefore, the data from boreholes and surface site investigations showed that the groundwater level is high.



Fig. 2. Geological profile (above) and Plan view (study area) of the Sabzkuh tunnel (Below) modified after [30].

3. Application of ADECO on the Sabzkuh Tunnel

The design and construction of an underground structure is performed based on informed up-to-date management and using the data from site investigations conducted before and during the construction. This method is to control the behavior of the underground structure from the beginning of the excavation operation in which geotechnical and geological conditions, as well as environmental and architectural limitations do not matter anymore. According to Lunardi (2008), Analysis of Controlled Deformations in Rocks and Soils (ADECO-RS) is the only approach that can be successfully applied in all types of grounds under various conditions [13].

ADECO-RS is the only reference parameter which is based on predicting, controlling and interpreting the soil and rock mass displacement. First, this parameter is predicted and adjusted using analytical techniques and is then measured and interpreted through empirical techniques. The logical procedure of designing and constructing a tunnel using ADECO is illustrated in Fig. 3. Because the deformations occurring during the construction process and the stability of the tunnel depend on the behavior of ground core face (advance core). In addition, the core-face stability is expressed in terms of three basic behavior categories A, B, and C.



3.1. Collapse mechanism

According to the conducted numerical analyses, the preliminary design of Part T1 of the Sabzkuh tunnel was performed based on a fullface excavation with a daily advance rate of 1.5 m. The support system consisted steel sets (2IPE) with two layers of steel meshes and 25 cm of shotcrete. The invert was closed at a distance of three times greater than the diameter of the tunnel without taking any reinforcement and protection measures. After completing the excavation of the tunnel portal and before starting the tunnel excavation, some signs of slope instability were observed. During the tunnel excavation, deformations were negligible but some local instability was found in the tunnel face. In addition, the results of monitoring and visual inspection of slopes showed the intensified instability of the tunnel portal after the tunnel excavation started. After excavating 35 meters of the tunnel, the high slope of layers, soil layers with poor geotechnical characteristics, old landslide activation, alternation of layers, as well as the non-compliance with the correct advance rate without taking proper reinforcement and protective measures resulted in the high instability of the forefront ground. Thus, the tunnel cross-section appeared in a non-arched form. The core-face behavior started with the delamination of soil layers from the excavation face that resulted in the total failure of the face and the tunnel collapsed in less than 8 hours. This procedure created a chimneylike cavity (22 m in diameter) at the top of the tunnel entrance [30] (Fig. 4).



3.2. Development of ADECO-RS approach

After the tunnel collapsed and the excavation halted, given the complex geological conditions of the region, it was necessary to conduct an appropriate analysis on the slope stability and the tunnel collapse. Keeping this in mind and based on successful experiences of using the ADECO-RS technique for excavation of tunnels under critical conditions, the DCP approach was implemented in the following five phases (Fig. 5):

Survey phase: Determining geological and geotechnical characteristics based on soil mechanics, revising and updating the geological profile.

Diagnosis phase: This phase was performed in two sections of the portal and the tunnel itself and included predicting the slope



deformation, studying the type and the size of common instabilities using three-dimensional numerical analysis and the findings from the previous phase.

Therapy phase: Making decisions about slope and tunnel prereinforcement and reinforcement measures in the framework of behavioral categories (A, B, & C).

Operational phase: Performing the stabilizing operation based on the

slope and tunnel pre-reinforcement and reinforcement.

Monitoring phase: Performing accurate monitoring actions based on predictions made in the fault detection and therapy phases with the interpretation of deformations as the response from the environment to the tunnel and the slope reinforcement progress.



Fig. 5. Flowchart of the DCP in T1 part of the Sabzkuh tunnel modified after [27].

3.3. Survey phase

Analysis of the surrounding environment with respect to the tunnel excavation operation was based on the results from the exploration and survey phase. It is, therefore, essential that the results of this phase be reviewed and discussed based on "*a sense of mutual cooperation among the design engineers and geologists*". Taking the advantage of this approach, the design engineer would have an accurate picture of geotechnical and geological conditions of the region. Thus, the fault detection and therapy phases would be followed more accurately. However, this was not taken into account during the initial stages of constructing the Sabzkuh tunnel before the tunnel collapse and finally the tunnel collapsed followed by the disruption of the construction process. Accordingly, detailed information needed to be collected from the geotechnical data before resuming the excavation process.

Surface studies

In the first stage, some samples were taken from inside the surface cavity on the tunnel portal. The results showed the existence of alternating layers of alluvial sediments with a slope of 75 to 80 degrees (Fig. 6).



Fig. 6. Samples taken from the hole in the cavity [27]

As a complementary measure of site investigations for identification purpose of the soil profile, two channels were excavated in both sides of the portal (Fig. 7).

The diversity of stratification inside the cavity, thesampling conducted in the soil layers, and the geological profile of the area of interest were reviewed and corrected. (Fig. 8).



Fig. 7. Stratification of the excavated channel in both sides of the portal [27]



Fig. 8. The modified geological profile of the slope [29] *Subsurface studies*

Subsurface studies were performed along the surface studies using geophysical techniques (Geoelectrical in Part T1 and low-depth reflective seismography in the first three kilometers of the tunnel) to identify crushed zones, potential faults and their vertical and lateral development, to identify the boundary between the formations and slope conditions as well as the ruptures from tectonic movements. The results of these studies were then combined .

Geophysical resistivity surveys were conducted in part T1 through complex resistivity profiling (CRP) and Schlumberger vertical electrical sounding (VES). Seismic studies were also conducted using refractive and reflective techniques .

Besides, seismographic and Geoelectrical profiles showed some evidence of Fault F3 at a distance of 70 to 90 m from the tunnel. As indicated by Geoelectrical profiles of the tunnel path, there were two sandy aqueous layers at a distance of 80 to 190 m and 250 to 310 m across the tunnel path. Furthermore, there was a fault zone at a distance of 150 to 200 m \cdot

There is a fault zone at a distance of 70 to 220 meter in the Geoelectrical profile of path B. There was some evidence in surface studies showing this fault, which is probably the intersection of the tunnel path and Solaghan fault. Moreover, in the seismic and Geoelectrical profiles, there were Fault F2 and a sliding surface at a distance of 310 m (Figures 9&10).

At a distance of 70 to 220 m across the tunnel, a few samples were taken in the cross-sections of the tunnel path perpendicular to profiles A, B, and C. The results of samplings in profiles X11 and X12 showed the tunnel intersection with fault F3 at a distance of 70 to 85. In addition, cross-sections X1 and X2 (at a distance of 200 to 220 m) confirmed the existence of the Solaghan Fault at a small distance from the tunnel (Fig. 10).



Fig. 9. Longitudinal view of low-depth reflective seismographic surveying



Plan-view of geoelectrical surveying in the study area





Cross section X12

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Fig. 10. Longitudinaland cross sections of geoelectrical surveying

4. Diagnosis and therapy phases

According to the predicted geological model and also the results of the diagnosis phase, the tunnel face show a short-term stable behavior and a long-term unstable behavior throughout the tunnel route as it collides the fault zones and sliding surfaces under the groundwater table. Accordingly, the following decisions were made:

In order to deal with possible instabilities in the core - face, it was decided that a central core be placed for the safety purposes. However, the rapid decline in the confining stress (σ 3) leads to conditions that are beyond the inherent strength of the masses around the tunnel even ahead of the tunnel face. Nevertheless, due to the complexities, uncertainties and possible changes in the ground conditions, a sufficient confining pressure must be induced in front of the face to prevent any failure and collapse and before choosing any costly and time-consuming solutions. To summarize, fiberglass nails were used in the tunnel face for unstable areas.

Ground improvement and reinforcement measures for the core must be implemented at the time when the core is still in a triaxial stress state with limited deformations. Tunnel excavation and loosening the advancing core before the face arrival drastically decreased the minimum principal stress. In some sections of the tunnel with an unstable face, application of conventional cement grouting creates an artificial resisting arch that deviates the resulting stresses in the face and simultaneously reduces the decline rate of the minimum confining stress. Also, the cement grouting improves the mechanical properties of the ground by decreasing the ground permeability.

In faulted zones and landslide surfaces, the stress state in the ground and surrounding the face exceeds considerably the strength properties of the host soil. Therefore, the arch impacts do not naturally occur in the tunnel face and the failure potential would increase. Deformations in these areas are considerably high and if stabilizing measures are not taken, the core and the face would become highly unstable resulting in serious instabilities such as the face collapse and the tunnel destruction.

Implementation of a drainage system by creating a truncated cone umbrella outside the core-face reduces the hydraulic gradient and increases the shear strength.

Invert closure has a significant structural functioning and is used in cases where a high stabilizing pressure in needed. This can completely control the creep inside the tunnel by forming a load-carrying ring. In addition, it makes the support system more rigid and increases the tunnel capacity for imposed loads and pressures. Based on the calculations, the invert closure must be executed at a distance 1 to 1.5 times greater than the tunnel diameter, so that it becomes statically active under the arch effect before the minimum principal stress disappears completely. In faulted and earthquake prone zones and in the portals, the tunnel must possess the required strength and rigidity.

Fig. 11 shows the geological model profile along with the information from survey, diagnosis and therapy phases in Part T1 of the tunnel.

5. Operational and monitoring phases

In the operational phase, the stresses were deviated in front of the tunnel. In this phase, anticipation is necessary in the monitoring and construction stages. It is clear that success in this phase depends on the accuracy of the predictions made in the diagnosis phase and the precision of design and geological models.

Construction activities restarted in the portal and the tunnel. First, the ground improvement and remedial measures were taken in the portal and the collapse zone. Next, the tunnel excavation was resumed based on the findings from the previous phase. The monitoring process was followed concurrently with each of construction stages.

- Drainage

Given the high groundwater level and the annual precipitation as well as the water infiltration through nearby agricultural lands, long drains were used in the portal to reduce the groundwater level. The continuous monitoring and inspections of the drained water in different seasons and preservation are of high significance in management of tunnels and the slopes. Moreover, the surficial water collection channels around the portal were restored and repaired and the irrigation of gardens and farming lands around the trench and the tunnel route was stopped to reduce the amount of leaking water.

Crossing the tunnel collapse zone

In order to cross the collapse zone, mechanisms of similar collapses were studied and the IPE Arc Support Technique (IAST) method, which has taken into account by many designers, considered the economic and technical matters.

In order to cross the collapsing area in part T1 of the Sabzkuh tunnel, and considering the design requirements, implementing the IAST and using the results of finite element analysis, IPE180 was used with an spacing and a length of 15 cm and 9 meters, respectively (Fig. 12). More details on the analysis and implementation of this method are found in Eftekhari et al. (2015) [31].



Fig. 11. Geological profile after investigation and survey phase





Fig. 12. The cross-section and the schematic design of longitudinal profile for crossing the collapsing area [31]

Filling the collapsing cavity

To fill the cavity in the slope by meeting the stability requirements and the minimum applied stress on the support system of the tunnel, three alternatives were proposed and modeled (Table 1). In order to model the filling requirements of the collapsing cavity, the collapsing area was divided into two parts of 5 and 11.8 m high (Fig. 13). In the next step, the properties of filling materials were applied and the maximum displacement and stress at the tunnel roof were modeled (Fig. 14). Having performed a technical and an economic analysis, the first alternative was rejected as it imposes a high stress on the support system. Out of the other alternatives, the third one was selected as the best options for its low induced displacement and stress in the crown and for its cost-effectiveness.



Table 1: Properties of proposed alternatives

Concept	Filling materials	Speci (K The first	al weight N/m³) The second	Maximum displacement in the crown	Maximum stress in the crown	
		stage	stage	(mm)	(MPa)	
First	Soil		18	45	0.065	
Second	Soil and lightweight aggregate concrete (LWAC)	13	18	20	0.03	
Third	Soil and concrete with controlled low strength materials (CLSM)	10	18	15	0.027	

In order to validate the results of numerical analysis, a convergence meter station was installed in the center of the collapsing cavity and the displacement of the support system was measured and recorded during the filling process (Fig. 15).



Fig. 13. Modeling the filling process of the collapsing cavity.



Fig. 14. Displacement variations after filling the collapsing cavity using third



Slope monitoring

To evaluate the performance and control the optimal progress of the

project based on the selected improvement and reinforcement method to ensure that no instability would develop in the long term, the slope monitoring was continued through visual inspections and geodetic surveys. As it is obviously seen in Fig. 16, the displacements showed an increasing trend before starting the reinforcement operations. The displacement in axis Y was the highest than those in other axes due to its higher sliding potentials. In addition, the maximum displacement was seen in Pin No 1 (R1). The reason was the high pressure of leaking water in this part of the aquifer that occurred when excavating the slope. The displacement rate decreased after implementing each step of the ground improvement operation. Upon the completion of the reinforcement operation and starting the tunnel excavation, the displacement showed almost a constant rate (Fig. 17).



Fig. 16. The monitoring benchmark position in the trench of tunnel inlet

Resuming the tunnel excavation

The tunnel excavation was resumed in the area of the behavior category B. The reinforcement system in this area included a 25 cm thick reinforced shotcrete along with 2IPE140 steel frames at intervals of 75 cm. The invert closure operation was executed at a distance of 1.5 tunnel diameter using the reinforced concrete with a thickness of 40 cm and buried 2IPE140 steel sets at intervals of 75 cm.

The numerical analysis results stated that the connection between the heel and the tunnel invert must be regarded seriously to transfer the induced stresses of this point. Furthermore, given the possibility of the high and rapid convergence of the support system, steel sets were installed separately after each advance step before concrete casting in the floor (Fig. 18). The results of monitoring operations carried out in this area showed the adequacy of support system and the implementing procedure.

At a 75 m distance of the tunnel inlet, the non-compliance of the correct spacing between the invert components and the advance core ruptured the walls and destructed the support. This problem immediately was solved through stopping the excavation and closing the invert. This challenge revealed again the importance of invert for the completion of the load bearing ring and the safety of the excavation.

At a distance of 85 m along the tunnel alignment, some problems such as an unexpected aquifer and inefficient implementation of the drainage system, sand lenses, and fault F3 resulted in the tunnel collapse. The presence of water under hydrodynamic conditions reduced the ground shear strength and increased the plastic zone and the instability potential of the tunnel. To pass through the collapsing zone, water ingress into the tunnel was prevented using the dewatering system ahead of the face. In the next step, cement grouting was performed from the surface and around the tunnel perimeter. The tunnel face was also plugged with plastic concrete (Fig. 19: A and C).





Fig. 18. The steps for invert closure

Crossing this collapsed area was successfully accomplished in a shorter period compared to the previous collapse. Excavation of the tunnel up to a distance of 150 m from the inlet was followed using the same process at a proper advance rate (i.e. daily advance of 1.5 meters in average). Given the results of the survey and design phases, encountering a water-bearing fault zone increased the stress significantly beyond the strength properties of the materials. The presence of water also reduced the shear strength properties of the ground. In addition, the fault zone caused a drastic decrease in the geotechnical properties of the ground surrounding the tunnel face. Finally, the interaction among these factors prevented the formation of the arch effect in the face, and thus, the resistant ground and enough time were not available for reinforcing the face. This resulted in a serious instability of the face and the re-collapse of the tunnel. Development of the plastic zone made the collapse reach the surface and created a cavity with a diameter of 10 meters on the surface. In addition, the heavy rainfall and the infiltration of huge amounts of water into the tunnels led to rapid development of the cavity and the formation of a flowing ground inside the tunnel.

To move through the collapsing zone, the tunnel face was plugged again and a drainage system was implemented in 6 to 9 m intervals with a spacing of 30 to 50 cm. Due to a high overburden thickness, cement grouting from the surface was not possible so it was performed inside the tunnel. The excavation and the crossing through the collapsing zone were performed in short length sections and the lining of high rigidity was used to control the pressure of the convergence (Fig. 19: B and C).

Having crossed the collapse and the observed severe instabilities, the requirements of the behavior category C came into force. First, drainage pipes and drains were implemented surrounding the tunnel profile before performing any reinforcement measures across the reinforced zone. The length of these pipes varied depending on the permeability of the adjacent layers. The length of the pipes was estimated regarding the type of materials pull out from the drill holes. In addition, installation of the drainage pipes in the face prevented the infiltration of water from the surrounding environment into the core-face and thus reduced the hydraulic pressure in this area. These reinforcement measures improved the inherent strength characteristics and plasticity of the core and contributed significantly to applying the Preconfinement pressure. The face ground was improved forming 4 to 6 m long reinforced arches overlapping in 2 meters. Therefore, the tunnel excavation continued under the support of cement-grouted arches. As the horizontal elements rose up, the distances between steel sets and their rigidity increased. The advance rate reduced to 55 cm and the invert element spacing reduced to tunnel's diameter. In order to monitor and validate the efficiency of



the reinforcement operation, regular inspections were carried out during the excavation process.

The tunnel was excavated for 200 m without facing any risk through implementing these requirements and achieving the advance rate of 1 meter per day. As shown in Fig. 11, a number of faults and sliding zones were observed at a distance of 200 to 350 m that were previously inspected in subsurface investigations. Therefore, the excavation process was performed according to the behavior category C. However, given the experiences gained through the operation and the optimization of the excavation process, the excavation operation was performed from a distance of 200 m to 350 m across the tunnel route applying the requirements of the behavior category B, which had a significant impact on the project schedule.

Despite the existing geological complexities and problems, based on the final data reported in Table 2, the predictions enabled application of ADECO-SEM in the Sabzkuh tunnel that were in a good agreement with reality.



concrete

C: Reinforcement measures B: The third collapse A: The second collapse

Fig. 19. Mechanism of tunnel collapse and reinforcement measures in collapsing zones.

Table 2: DCP values of Part T1 of the Sabzkuh Tunnel.											
Section	Geology	Category		Collapse causes	Remedial solution	Advance	Advance rate (m/day)		Time		
		Predicted	Actual			(Ш)	Max.	Ave.	(uay)		
0~37 m	ML - CL, with gravel and sand lens	А	А			37	2.25	1.5	25		
Collapse 1		С	С	Landslide, Groundwater	IAST	7			140		
42~85 m	CL - ML with gravel and sand lens	В	В			41	2	1.5	22		
Collapse 2		В	С	Fault, Groundwater	Drainage, Surface grouting, Cement grouting	5			22		
90~150 m	CL - ML gravel and sand lens	В	В			90	2	1.5	57		
Collapse 3		В	С	Fault Zone, Groundwater	Drainage, Cement grouting	5			14		
155~200 m	GP – GC, Ground water, Fault zone	С	С			45	1.00	1.00	45		
200~350 m	GP – GC, CL-ML, Ground water, Fault zone with sand lens	B&C	В			150	1.75	1.5	90		

6. Summary and conclusion

Three daylight collapses occurred during the excavation of the conventional excavation part (T1) of the Sabzkuh tunnel. Using the ADECO technique made it possible to cross through collapsing zones in the tunnel excavation process in areas with faults and high landslide potentials. After the occurrence of the collapse at a distance of 35 m across the tunnel and the instability in the tunnel inlet, the excavation operation was stopped and the process of tunneling was revised using the ADECO approach. Base on the derived data from comprehensive surface and subsurface site investigations and predictions on the stability of the tunnel core-face, the tunnel was classified into two basic behavior categories, B and C. by Using of these categories, the design engineer determined the type of the stabilizing operation to be implemented and to control deformations. As a result, the type of the stabilizing method and the best cross and longitudinal sections were

chosen for each location of the tunnel. These activities included the conventional cement grouting, installing drainage systems around the tunnel perimeter, invert closing at a distance of 1 to 1.5 tunnel diameter, and the use of advance rates of 85 and 55 cm for the categories B and C, respectively.

The Sabzkuh tunnel is located in one of the most complex and difficult geological areas of the High Zagros. The results obtained during the excavation of 350 meters of this tunnel indicated that selection and application of the most appropriate excavation and support methods provided by the ADECO-RS technique could give an acceptable advance rate through complex geological conditions. This was evident in the excavation of the Sabzkuh tunnel at a distance of 155 to 200 meters from the inlet portal, as the most challenging part of the excavation process. The average daily advance rate of one meter in this area showed a promising progress compared with other parts. The appropriate advance rate with sufficient safety in ground challenging areas showed a high degree of development in the conventional

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tunneling approach.

Fig. 20 shows the timeline pie chart for different construction stages of Part T1 of the Sabzkuh Tunnel. As seen, almost 42% of the construction time was related to delays from the collapse. However, most of this time (34%) was due to the delayed operation for crossing through the first collapse. The delay from the first collapse was due to the unawareness of failure mechanisms and the techniques used to cross the collapsing areas as well as the lack of the required facilities and equipment because of contractual issues which wasted a considerable amount of time. However, the advance rate and the speed of crossing through the collapsing areas increased significantly after employing the ADECO-RS approach.



Fig. 20. Timelines for different excavation stages of Part T1 of the Sabzkuh Tunnel

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