CASES OF TUNNEL FAILURES IN TWO HYDRO-ELECTRIC PROJECTS

CAS DE DEFAILLANCE DE TUNNELS AU COURS DE DEUX PROJETS HYDRO-ELECTRIQUES

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SYNOPSIS : The paper describes the failure of two tunnels associated with hydro-electric projects in the Himalayan region. In the first case the diversion tunnel of Maneri Bhali Project under construction was daylighted when a dam created by a landslide in the upstream reaches of the Project was breached. In the second case, the pressure tunnel of the Kopili Hydro-electric project failed when it was being filled up for the first time for commissioning the project. The paper describes the circumstances leading to the failure, the extent of damage and the remedial measures adopted.

MANERI BHALI PROJECT STAGE I

Maneri Bhali Hydro-Electric Project Stage I harnesses the power potential of water flowing down the river Bhagirathi between Maneri and Uttarkashi. The project works include construction of a 8.63 km. long., 4.75 m dia. concrete lined tunnel, a concrete dam, an intake cum sedimentation chamber and a powerhouse.

For the construction of the main diversion dam, intake structure and the sedimentation chamber etc., a diversion tunnel of 7.0m diameter and 139 m. length was constructed on the right bank of the river Bhagirathi.

Cloudburst and Landslide

A sudden cloudburst took place on 5th August, 1978 at Kanodia Khad on the hill side of river Bhagirathi about 25-30 km. upstream of Maneri. A huge landslide occurred following the rains. Landslide mass included huge Deodar trees along with the debris of soil and rock mass. This created a blockage in the river. The normal flow of the river at that time of the year was substantial, but following the blockage the flow was reduced to a mere trickle. When Mr. D.P. Sharma, Superintending Engineer (Hydel) went for his morning walk on 6th August, 1978, he noticed that the water in the river had reduced considerably and it was flowing at much lower level as seen from the marked water lines on the banks. As he went further, he found that most of the discharge in the river was coming from the Maneri Gad, a small stream which joined the river Bhagirathi at Maneri and there was very little water coming from the upstream. He immediately informed the district authorities, the contractors, the public works department and other such authorities which he thought were concerned.

Due to the general alert sounded by the district authorities, irrigation and hydel departments and the contractors necessary precautionary measures had been taken to save the men and machinery from the flood wave that was likely to come.

Precautions taken

The Hindustan Construction Company which was constructing most of the civil works for the project, took necessary precautions to remove their construction plant and machinery from the vicinity of the river bed. Work was stopped and people staying near the river banks were evacuated.

As in-charge of the construction of the powerhouse at Uttarkashi, Mr. D.P. Sharma took care to remove the heavy plant and machinery. Even the spiral cum speed ring weighing as much as 35 tonnes was removed to a safer place. The diesel powerhouse which was put up to supply the construction power was blocked all around by sand bags to prevent the water from entering.

Flash flood and the damage

Efforts were being made in the meantime to blast the blockage to breach the dam created by the landslide. As a result thereof, the dam breached in the late afternoon of 6th August. What was not anticipated fully before breaching the dam, was the fury with which the water will run down. As the dam broke, the water gushed out creating a monstrous wave front of about 5 to 6 metres height. As the wave front moved downstream, it carried the uprooted trees and debris along with it, smashing everything that came its way. Roads on either side near Gangnani village were washed out. Five to six road bridges were washed out. The diversion tunnel was day-lighted for a length of about 40 m. Large cavities had been created below the sill beam of the gate and also on both sides of the tunnel inlet. The flood waters found its way into the tunnel through these cavities below the intake structure.

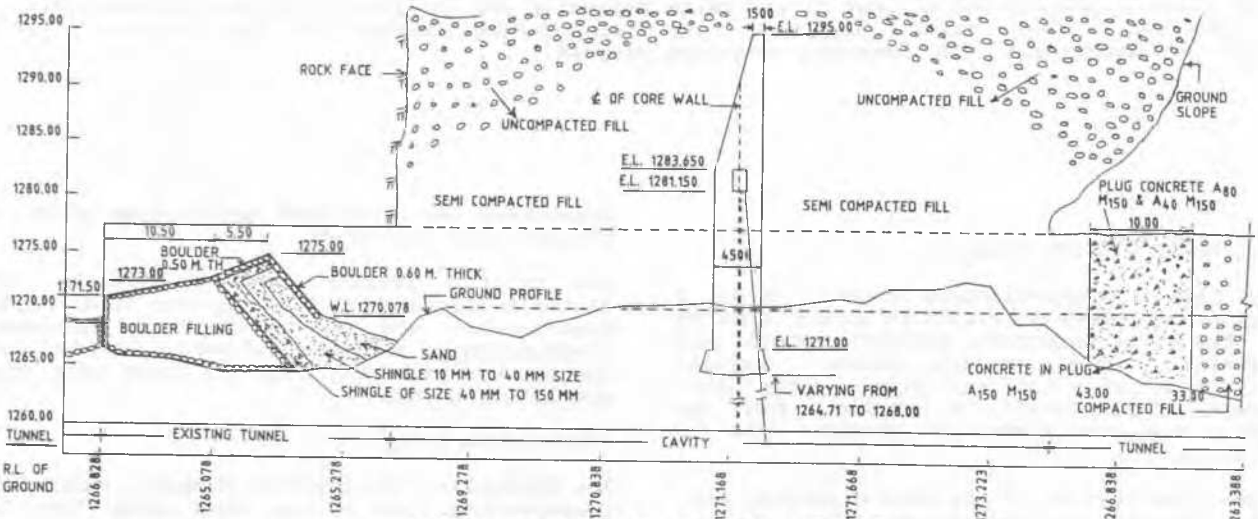
Since prior warning had been given and most of the people were prepared, no loss of life took place, even though almost the entire left bank was washed out. The spillway glacis had already been constructed. Its face on upstream as well as downstream was damaged due to the boulders brought by the river hitting the surface. Boulders as big as 1.5m x 1.5m to 2.5m x 2.5m were seen rolling over the glacis and passing through the diversion tunnel. There was not much damage to the intake structure of the sedimentation chamber as the trees blocked the passage of boulders etc. At the powerhouse site the penstock pieces were shifted downstream. Slushy material filled all the block-outs left for construction creating a mess.

Second Wave

However, the breach of the dam was not total and quite unexpectedly a second flood wave came during the night of 8th August, 1978. Since most of the people in the downstream area had seen the flood wave on 6th August and the damage caused by it, they had become complacent thinking that the worst was over. This flood came as a surprise and even though the physical damage caused by it was not that severe, it caused heavy loss of life.

Diverting the river over spillway and treatment of cavities in Diversion Tunnel

Following the damage to the diversion tunnel due to the flash flood, it was decided to create a barrier in front of the diversion tunnel by dumping crates filled with boulders, stones and sand bags. This had to be done before the arrival of the next monsoon flood. It was however, not found very effective and as this operation was going on in the month of May and June, 1979, it was seen that a new cavity was formed on the right side of the inlet portal of the diversion tunnel, cutting off the road. The main cavity formed in 1978 also started to widen after the monsoon set in and further damage to the roads took place. It was then decided to divert the river water over the spillway section. A diversion channel and a coffer dam was accordingly constructed. The bulk head gate from the tunnel was removed by cutting piece by piece, and the diversion tunnel was plugged by concrete portions of the diversion tunnel which had daylighted were converted into a compacted rockfill section with a concrete core wall.



L-SECTION ALONG ℄ OF DIVERSION TUNNEL

Rockfill material of the order of 54,500 million cubic metres was required for this purpose.

KOPILI HYDRO-ELECTRIC PROJECT

Kopili Hydro-electric Project constructed in the North-Eastern part of the country comprised

The tunnel was designed to withstand an internal water pressure of 1.4 MPa. Part of the length of the tunnel between the surge shaft and the penstock was provided with the steel liner and part with reinforced concrete. The lengths of the steel lined and RCC portions were 60m and 70m respectively. The remaining length of the tunnel was lined with plain concrete. The slope of the tunnel in the reach was 1 in 50. When the tunnel was being pressure tested for commissioning the project, the reinforced concrete portion of the tunnel cracked open and a stream of water gushed out of the sloping hill mass; bringing down soil debris along the hill. It was reported later that the maximum pressure inside the tunnel at the time of failure was recorded as 0.7 MPa at the valve house gauge located close to the tunnel exit portal.

Causes of failure

A committee of experts was constituted to investigate the causes leading to failure. During their inspection, it was found that the RCC lining had cracked at the crown extensively along its entire length of 70m and about 30m in the adjoining plain concrete lining. A subvertical joint in the rock could be seen through the crack in the concrete lining. The joint had been opened up by the water which escaped from the tunnel flushing the joint of the infilling materials. Though the reinforcements themselves were found to be intact, they were separated from the concrete at many places. The lapping joints of the reinforcements were all along in the crown, and these joints had failed. Behind the cracked lining, the rock mass was observed to be extensively jointed.

When the tunnel was being designed, the practice followed was to locate the tunnel below the ground as far as possible at a depth equal to the total static head of water over the centre line of tunnel. Wherever sufficient cover was not available, steel liner or RCC liner was provided. These liners were designed depending upon the competency of rock mass in sharing the hydraulic pressure. If leakage of water from the tunnel was expected to endanger the rock mass, steel liner was preferred to RCC liner. Based on the above concept, it was decided to provide a steel liner in the last 300 m length of the tunnel. However, due to certain practical difficulties of construction, a review of the design was made to reduce the length of steel liner. Though in-situ stress measurements were made using flat jack tests at several locations in this tunnel, the tests could not be carried out at certain critical locations. Based on the general observation of the rock mass, the length of steel lining was reduced to 60m and an RCC lining of 70m length was provided.

The failure of this tunnel has demonstrated the need for precise measurement of in-situ stresses and its impact on the design. The criteria of sufficient rock cover may be suitable for preliminary designs but for the final design, the detailed geological features, rock mass deformability, in-situ stresses, rock permeability and ground water conditions should be given appropriate considerations. Where the minimum principal stress is not the vertical stress, and where deformable rock or shear zones exist, positioning a tunnel to meet only vertical cover criteria may not be adequate. Knowledge of complete stress field and of the rock modulus is necessary.

Finally, in the design of the lining for a pressure tunnel, the following factors should be considered.

- Head loss
- Leakage of water
- Long-term stability

Head loss through a conduit is principally a function of wall roughness, the diameter and the velocity. As a result, hydraulic equivalence can be obtained between large diameter unlined tunnels versus smaller lined tunnels of greater hydraulic efficiency. With small tunnels, 2 to 3 m in diameter, there is a greater need for a smooth lining to maintain acceptable head losses. However, as the diameter is increased, the wall roughness has less effect on head loss, and equivalence is achieved with small diametral changes.

Excessive leakage of water can occur from pressure tunnels in two ways. Firstly, by hydraulic fracturing, and secondly if the rock is pervious and the internal pressure exceeds the external ground water pressure. Different types of linings can be considered as impervious and RCC lining or reinforced shotcrete lining can be considered to be pervious as they have local pervious zones due to placement imperfections, or shrinkage cracks during curing. It is a common misconception that concrete or shotcrete is impervious. They are also easily cracked under internal pressure where deformable rock zones exist. If the permeability of the rock mass is high, grouting will be necessary. Controlled pressure grouting in different stages will be useful where grout loss has to be controlled and where high pressures are necessary.

To ensure the long term stability of the tunnel, the lining must be designed for the following :

- Erosion of rock or joint filling by water under pressure.
- Rock support, temporary and final.
- Hydraulic pressure during watering, operation and dewatering.

The designed thickness of RCC lining was 30 cm. Good amount of skill is called for concreting the lining, as it is to be carried out in the restricted space behind the shutter. The workability 5 of the concrete could be increased without reducing strength by adding plasticisers. Concrete may have to be pumped using concrete pumps. After shrinkage of concrete, the gap between concrete and rock surface should be grouted. In the failed portion of the tunnel, it was learnt from the records that the grout holes at the crown had taken substantial quantities of grout whereas the side holes had taken very little grout. This raised a doubt about the integrity of the concrete at the crown. However, a series of non-destructive tests carried out on the concrete lining did not indicate any apparent bad quality.

Remedial Measures

Based on the study of detailed topographical survey carried out after the mishap, the steel lining was extended to a length of 240m beyond and in continuation of the existing steel lining. The diameter of the steel lining was decided to be 4m so that it can be constructed inside the existing concrete lining with additional concrete backing. It was stipulated that any damaged and loose concrete should be removed and filled with fresh concrete before the new lining is placed. Grouting as well as monitoring of hill slopes were also suggested for the overall stability of the rock mass.

Lessons learnt

The lessons learnt from the failure are :

- Adequate and rational investigations should be carried out to precisely identify the geological features of rock mass, deformability and permeability characteristics.
- The conventional 'rock cover criteria' may not be adequate for the design of pressure tunnel especially when adverse geological features are encountered and in-situ stresses are not compatible with the physical ground profile.
- Precise measurement of in-situ stresses using suitable methods should be made.
- Selection of final lining for pressure tunnel is a process which begins at the design stage and continues until the construction is complete and the geological conditions are known in detail. This requires well structured and flexible specifications and contract documents, which allow design modifications.
- Free and fair flow of information between gelogist, design and construction engineers should be ensured.
- Appropriate quality control measures should be adopted.