Recovery of Collapse and Restoration of Tunnel Belonging to the Water Supply System

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ABSTRACT: Underground structures require maintenance services and, if necessary, recovery, since they may suffer structural damage both during its construction as during its lifetime, by natural wear and tear or for associated geological-geotechnical risks. This article presents the measures adopted for recovery of collapse and tunnel restoration, executed by NATM method, located in the city of Mogi das Cruzes and belonging to the water supply system of São Paulo City, Brazil. We also present local geological aspects, diagnosis of rupture occurred, inspections and mapping of excerpts to be strengthened, short, medium and long term measures to be adopted and geological-geotechnical risks after recovery of the collapse.

1 INTRODUCTION

The Tietê/Jundiaí interconnection system was conceived in the 1990s to meet the water demand of the Metropolitan Region of São Paulo (RMSP), allowing increased production of the (ETA) Taiaçupeba Water Treatment Plant from 5 m³/s to 10 m³/s. Through this system it is adduced up to 70% of ETA Taiaçupeba production, reinforcing immensely its importance. This system promotes the use of waters from Ponte Nova and Paraitinga dams, which discharge in Tietê river and are impounded in the Biritiba Raw Water Pumping Station (EEAB) with flow of ≈ 9 m³/s. From the EEAB Biritiba water is conducted to the dike of Biritiba dam and forwarded by gravity through Canal 2 and Tunnel 2 to Jundiaí dam, where it follows by gravity to Taiaçupeba dam to then be impounded, treated at Taiaçupeba Water Treatment Plant and distributed to 3.5 million inhabitants of the East Zone of the MRSP.

This article approaches the recovery and restoration of the rupture in the above mentioned Tunnel 2. Tunnel 2 was in operation without incident or significant incident for over 10 years, when its obstruction occurred by breaking effect in the vicinity of progressive 1,400 m, from the upstream portal. Figure 1 shows an overview of rupture. The rupture was observed from the flow reduction to 2m³/s.

Figure 1. Overview of collapsed section.

Figure 2 presents a general plan with the rupture location. Notice that on the left of the tunnel on the treeless area lies a quarry that was inactive during construction and began to be explored just prior to the occurrence of the rupture.

The original excavation section had no coat, and shotcrete was used to cover any flaws. The constant detonations led to formation of micro-cracks that evolved to more severe damage because of repeated blasting.

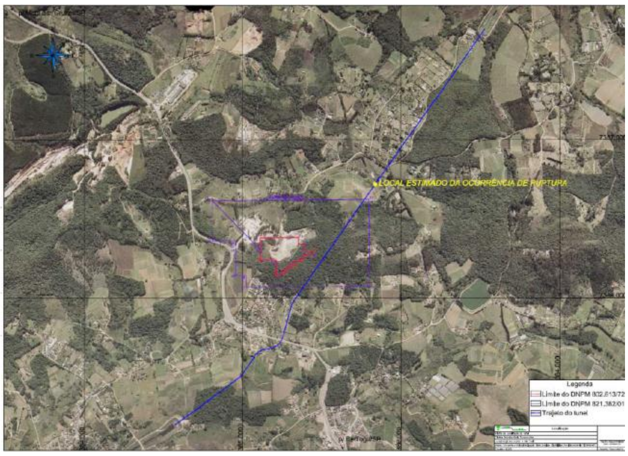

Figure 2. General photo identifying the rupture site.

2 LOCAL GEOLOGY

The area where the tunnel was implemented is part of the Pre-Cambrian-age Embu Complex (CPRM, 1990). The tunnel is sub parallel to the Cubatão Shear Zone, crossing at least two mapping units: gneisses (sheared or not) and granitic rocks with similar grains, average granulation and pink and/or gray color. In the region where the rupture occurred, the tunnel is intercepted by a NW-SE oriented line, mapped as failure / fracture.

Faced with the presence of thin layer of shotcrete on tunnel wall, there have been rare observations of rock block by focusing on the rupture site, and in places where the shotcrete had been eroded. Geological inspection of field was restricted to the northern portion of tunnel 2 (Intake portal until close to pile 1+200m, rupture site).

The rupture site matches approximately the geological contact between the granite and gneiss. It is a fragmented granitic, altered, oxidized and discolored (Figure 3). The sizes of the blocks that make up the rubble of the rupture indicate spacing between fractures about 20 cm. It was not observed contribution of groundwater at the rupture site.

Figure 3. Fracture system in wall of Tunnel 2 with detail of fragmented granitic quartz.

3 DIAGNOSIS OF RUPTURE

For tunnels and underground works, the effect of vibrations caused by detonations is equivalent to that of earthquakes, but repetitive and with cumulative effect. For actual earthquakes, the North American AASHTO standards show that occur distortions and displacements in grounded structures, whether tunnels or ditches.

In the case of quarry close, repeated blasting lead to vibrations that by reaching the tunnel surrounding, generate micro-cracks and minor damage, leading to more severe damage by cumulative effect of repeated blasting. There were indications that the detonations from the quarry were of medium to high intensity, by the result on the quarry walls, showing intense fracturing of the rock massif, Figure 4.

Figure 4. Intense fracturing of rock ,mass on the quarry wall.

In addition to this observation, it has been found through probing hole performed nearby the rupture that the rock block on site features good geomechanical quality (Figure 5), and that it would not break spontaneously after over 10 years of good operation.

Figure 5.Rock block with good geomechanical quality nearby the rupture.

4 SHORT, MEDIUM AND LONG TERM MEASURES

4.1 *Short Term – Tunnel Recovery*

Tunnel 2 was built 15 years ago by manual excavation, crossing soil and rock sections, covering 62m of which 3m of soil. The recovery of the collapse was scaled to be implemented from 10m prior to the start of the rupture until 10m after the start of the rupture. The reinforcement section has coating in shotcrete with 25cm in thickness, cap framing consists of trellis rib and 1 welded screen Q196 type for inverted arch the frame consists of 2 welded screen, Q396 type. Figure 6 presents the section for reinforcement of the rupture site.

The following activities were provided for recovery of the collapse: 1) excavation of the rubble from the tunnel collapse, 2) Removal of loosen rocks in the chapel formed by collapse, 3) Implementation of trellis ribs every 60 cm (or less) and welded screen, 4) Application of shotcrete lining in tunnel Cap, 5) Implementation of inverted arch in shotcrete or reinforcement of cap base with injected root pile and 6) filling of chapel with mortar injection.

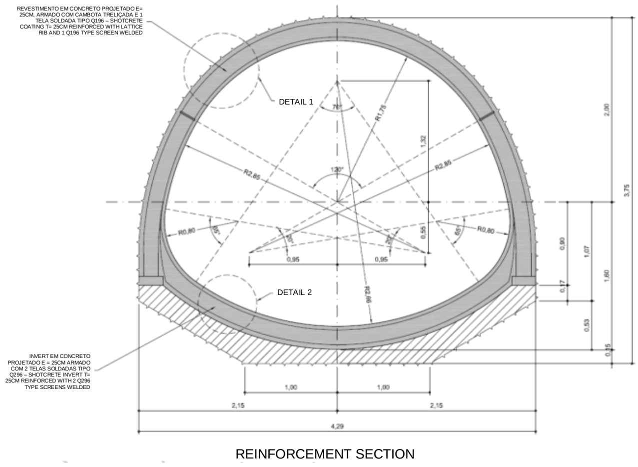

Figure 6. Reinforcement section used

4.2 *Medium Term – Tunnel Restoration*

For tunnel restoration, the following reinforcements were provided for: 1) reinforcement with eventual tiebacks, 2) reinforcement with systematic tiebacks, 3) reinforcement with trellis ribs, 4) shotcrete reinforcement, 5) reinforcement of floor – wall joint and 6) deep drainage. The Figures below show these reinforcements.

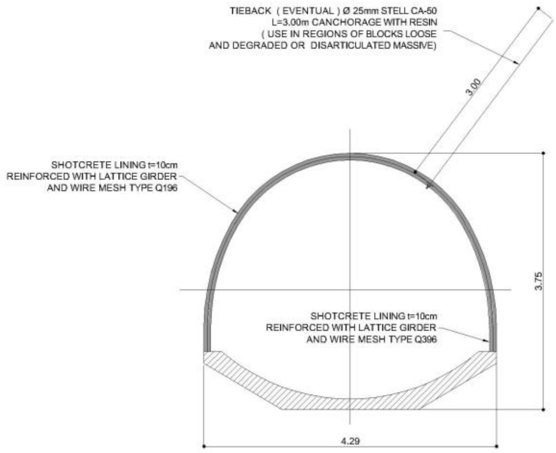

Figure 6. Reinforcement with eventual tiebacks.

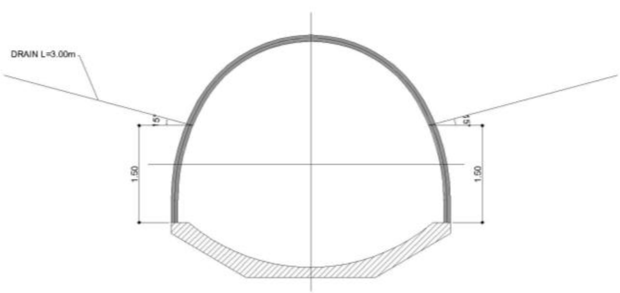

Figure 7. Deep Drainage.

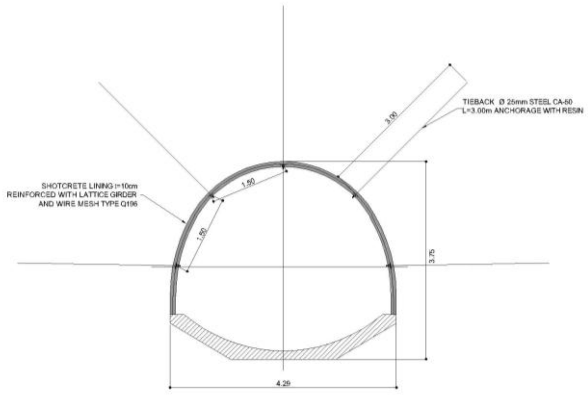

Figure 8. Reinforcement of systematic tiebacks.

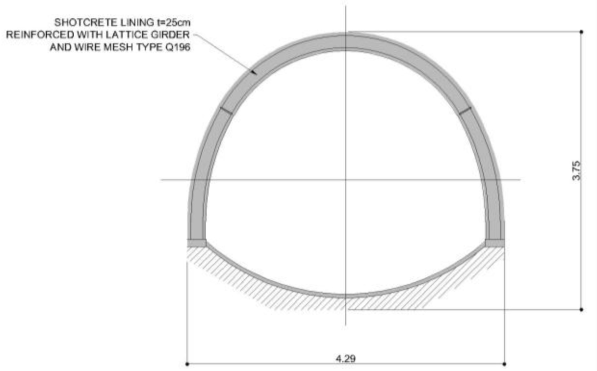

Figure 9. Reinforcement with trellis ribs.

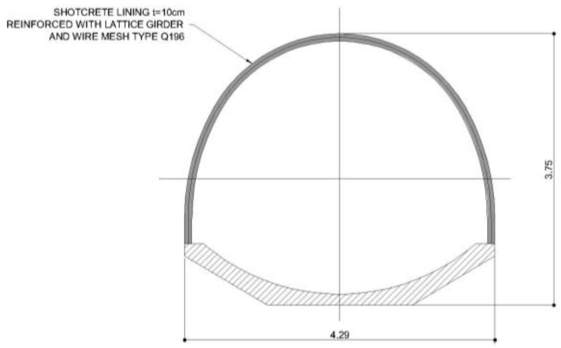

Figure 10. Shotcrete reinforcement.

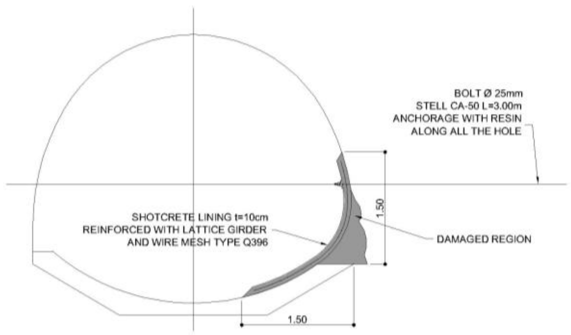

Figure 11. Reinforcement of floor – wall joint.

4.3 *INSPECTION AND MAPPING OF REINFORCED SECTIONS*

4.3.1 *Intake Portal*

The tunnel intake portal section features horseshoe-shaped section with H and W ≈ 3.8m. It was verified the existence of support system with ribs until the initial 370 m of the tunnel, This section showed overall good conservation condition, presenting only punctual degradations, possibly by effect of blasts after the construction. Figure 13 shows the view of the tunnel portal.

Figure 13.View of Tunnel 2 portal

In massif outcrop locations, with the exception of the rupture site, it is consistent and little to medium fractured, with visible drilling marks.

We identified 15 sections requi9ring reinforcement in the support system, according to Table 1. The problems of these sites are related to supply of groundwater, and/or section loss, and/or fracture observed during qualitative visual inspections.

Table 1. Section that needs reinforcement from the portal to the rupture.

Distance of Portal (m)	Geological description	Support recommendation	Section Geometry
0 to 370	-	-	Ribs
230 to 250	-	A	Ribs
280 to 300	-	A	Ribs
315 to 330	-	A	Ribs
350 to 370	-	B	Ribs

355	-	B+C	Ribs
395 to 410	-	B+A	Erosion on base of section
430 to 460	Fractured rock	B+D	-
460 to 480	Fractured rock	A	Erosion on base of section
510 to 850	Fractured rock	B+D+A	Formation of small chapels, structural blocks, and section loss
870 to 890	Fractures parallel to ceiling	B+D	Formation of small chapels
890 to 915	Fractured rock	A	Erosion on base of section
925 to 950	Fractures parallel to ceiling	B+D	Loss of section
1000	Rock slightly fractured	B+A	Loss of section
1019	-	D+B+A	Erosion on base of section
1052	-	F+A	-

Where A=reinforcement of floor – wall joint, B=Reinforcement of shotcrete, C=Injection, D=Eventual Tieback, E=Systemic Tieback and F=Drain.

4.3.2 *Tunnel Outflow*

The tunnel features horseshoe-shaped section, com H and L ≈ 3.8m. There are some sections with ribs, which are: 0-150 (tunnel portal), 890-940 and 1030-1100m. Figure 14 shows a view of the tunnel outflow.

Figure 14.View of Tunnel 2 Outflow.

We identified seven sites requiring reinforcement in the support system, according to Table 2. The problems of these sites are related to underground water supply, and/or the section loss of the excavation. In the massif outcrop locations, with the exception of the rupture site, it is consistent and little to medium fractured, with visible drilling marks.

In the region of pile 0+721m (bypass) there is significant volume of groundwater. This site has low coverage and, in surface, it coincides with the urban area.

Table 2. Sections that require reinforcement from the portal to the rupture.

Distance from outfall portal (m)	Geological Description	Support Recommendation	Section Geometry
0 a 150	Section with Ribs		
150	-	F+B	
160	-	F+B	-
500	Geological structures parallel to excavation Gneiss of low consistency	F+B	Formation of Chapel
721	Coherent massif, little to medium fractured	B	Intersection of bypass tunnels
750	-	B	Loss of section
880	-	F+B+D	-
890 to 940	Section with ribs with contribution of groundwater		
1030 to 1100			
1490 to 1500	Coherent and little to medium fractured granite	B	Enlargement of section
1795	Site of rupture		

4.4 *Long Term*

The basic components for maintenance and management of underground structures are inspection and diagnosis. Inspection means an examination of tunnel support conditions, identifying, mapping and recording existing anomalies. The diagnosis means an assessment of research, observation and of results obtained by identifying possible causes responsible for the occurrence of anomalies, its connection with the deterioration of the concrete and the recommendation of corrective measures to ensure the integrity and durability of tunnel support. The anomaly is defined as any deterioration that occurs in concrete support, resulting from chemical reactions and mechanical actions.

Long-term measures to be taken are periodic inspections, and during this inspection if an abnormality is observed, a diagnosis must be done, taking the necessary measures to repair. Inspections shall be carried out each year or at most every two years.

The visual inspection consists of the quantitative survey and qualitative assessment of the existing anomalies in the structural elements of the tunnel. The site to be inspected must be free of debris, corrosion or other foreign substance on the surface. During the visual inspection in Tunnel 2 anomalies must be registered, such as: concrete erosion; cracking; detachment; outcropping; efflorescence and carbonation; hollow area (no adhesion); segregated concrete; reinforcement and/or exposed trellis rib; humidity and water emergence and infiltration.

From the register of anomalies in the tunnel and based on analysis of these anomalies is possible to take decisions in the most relevant cases and the decision may be to perform just the monitoring over time. Among the decisions to be made in the cases considered relevant are: define the types of repairs and specify them by type of anomaly observed in its alert level, characteristics and causes, establishing standardization. When you choose to monitor certain anomalies means that they are not at a level to immediate intervention, but will cause future problems if evolve.

To classify the type of intervention to be used in a particular anomaly, important factors should be considered as its location, if compromises its functionality, If it causes structural problems, the importance of a certain structural piece in the structural support as a whole.

In addition to the punctual classification of anomalies and emergency measures directed to each one of them, it is important to perform an inspection report on the General conditions of the tunnel, analyzing the structure globally. It is necessary to assess the conditions of studied results, and in the tunnel as a whole, indicating their degree of deterioration, what is to say, in addition to the micro-level diagnosis (anomalies), it is necessary to also have another one in macro level (tunnel as a whole).

When the structure management is done, in terms of the gradual degradation process, we obtain a domain of their evolutionary behavior and it is possible to act on structures in preventive maintenance-level, or even predictive in some cases. As a consequence of this management there is increased cost / benefit ratio of maintenance substantially greater considering that, the sooner we intervene in a degradation process, lower the cost of this intervention will be.

Maintenance of Tunnel 2 can be carried out as follows, according to the diagnosis of anomalies:

- Without the presence of anomalies – there is no measure to take, from the point of view of physical interventions, but it is necessary to schedule periodic inspections, to ensure the proper functioning of the structure;
- Presence of anomalies, but without need of physical interventions - these anomalies must be monitored;
- Presence of anomalies, however they do not cause serious damage to the durability and safety of the structure – need to perform minor repairs;
- Presence of anomalies that compromise the system's functionality – repairs must be performed immediately

5 GEOLOGICAL-GEOTECHNICAL RISKS AFTER RECOVERY OF THE COLLAPSE

Taking into consideration the quarry next to the site of rupture of the tunnel, vibratory shock waves caused by blasting behave like analogous seismic vibration. In case of blasting in rock around the tunnel, this vibrating wave could hit the rock massif of tunnel section eventually with rupture and with consequences to the integrity and stability of Tunnel 2. As well as cement grout to fill the Chapel, formed by the rupture, the shotcrete used in tunnel wall reaches its design strength 28 days after its application. Before this period we avoided detonations in rock surrounding the quarry since the vibrating wave could reach the shotcrete reinforcement in rupture section. Therefore the quarry had to be paralyzed in its detonation activities during those days.

After applying all the reinforcement structures described in design, set out in tables, and reaching the final resistance of building materials, the tunnel safety and integrity were reestablished, and the water supply returned to its normal volume.

6 CONCLUSIONS

This article presented the case of rupture of Tunnel 2. A water tunnel responsible for water supply in the MRSP. The tunnel recovery and restoration was performed successfully, not requiring the interruption of water supply in São Paulo City.

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