Interpretation of exceptional stress levels from back-analysis of tunnelling problems in shallow basalts at the ITA Hydroelectric Power Project in S.E. Brazil

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ABSTRACT: A 1450 MW hydroelectric project was recently completed and commissioned in south-eastern Brazil. Extensive and unexpected rock engineering problems occurred during construction. A narrow basalt ridge separating a long meander of the Uruguai River was the site of the project, which was potentially very favourable due to the relatively short length of the five diversion and five pressure tunnels through this 150 to 200 m high ridge of basalt. Extensive tangential stress related popping and spalling was experienced when driving the auxiliary 15×17m diversion tunnels, both in the arch and invert, even when the depth was as little as 50m. The horizontal stress, trending NE in the region, appeared to have been seriously concentrated in the narrow N-S oriented ridge, and further concentrated in two massive basalt flows having highest Q-value. When the 9m diameter pressure tunnels: mostly as 53° shafts were driven, tangential stress related popping occurred again during excavation, but the greatest problem occurred when contact-grouting the reinforced concrete linings, which caused tensile cracking and extensive needs of repair, due to augmentation of the negative, effective tangential stress. The negative effect of stress fracturing was experienced again when operating the emergency spillway for just a few hours. Extensive N-S and horizontally oriented stress fracturing caused an unexpected high rate of scour in the unlined chute. The paper discusses the likely magnitudes of the major horizontal stress, based on back-analysis and some in situ tests. An interesting phenomenon was the measurement of highest permeability in the most massive basalt flows, due presumably to tensile cracking caused by the exceptional horizontal stress anisotropy which may have exceeded 20:1 or even 25:1.

1 INTRODUCTION

The UHE Ita hydroelectric project is located on the Uruguai river, in southern Brazil. Construction started in March 1996 and the first turbine was commissioned in June 2000. Installed capacity is 1450 MW, provided by five 290 MW Francis turbines fed by 9,0m diameter, 120,0m long power tunnels inclined at 53° (designated TF-1 to TF-5). At the project site the Uruguai river has a sharp bend, designated Volta do Uvá, where the river describes an 11 km long meander. This special geomorphology favoured a very compact layout of the project which is detailed in Figure 1.

River diversion was accomplished through five tunnels: two main tunnels (TD-1 and TD-2) 14,0m wide by 14,0m high, with control gates, permanently in operation, and three auxiliary tunnels (TD-3, TD-4 and TD-5) 15,0m wide by 17,0m high, without control gates, which operate during floods.

An auxiliary spillway VS-2, with 4 gates, is located over the upper-diversion tunnels (TD-3, TD-4 & TD-5), in a way that the stilling basin coincides with the downstream out-flowing portals of these three tunnels.

The Project was built through a "Turn Key Lump Sum" contract by CONITA (Consorcio Ita), a partnership lead by the ODEBRECHT Group and formed by CBPO Engenharia Ltda (civil works), TENENGE (assembling), ABB/ ALSTOM/ VOITH/ COEMSA-ANSALDO/ BARDELLA (eletro-mechanical equipment) and ENGEVIX Engenharia (design). The authors were consultants to the civil contractors of Consorcio Ita, and are grateful for the opportunity to be involved in these challenging and unusual rock mechanics problems.

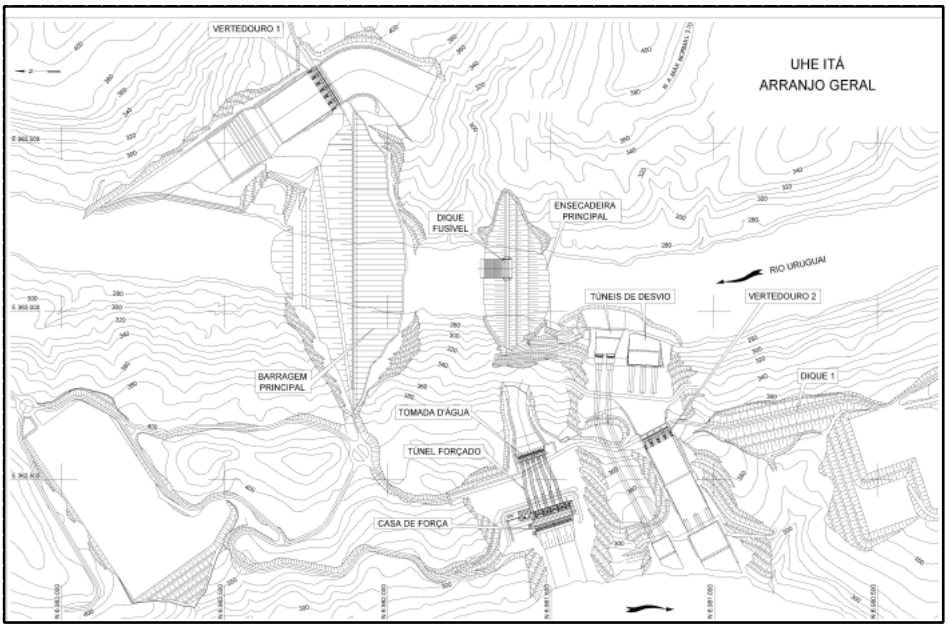

Figure 1. General Lay-out of the UHE Ita hydro electric project.

2 PROJECT GEOLOGY

UHE Ita is the first hydroelectric project built on the Uruguai river. The Basic Design was completed in 1987. The Ita project site is located in the domain of basaltic lava flows of the Serra Geral Formation with a local thickness of about 400m, overlying sedimentary rocks (Botucatu sandstone). At the site nine basaltic flows were identified between El. 400,00m and 200,00m, designated "D" to "L" from top to bottom (Figure 2). The tabular sub-horizontal character of the flows may be observed at valley hillsides, with a sequence of slope scarps, breaking in steps.

The weathering mantle is thick in the dam area and also along the spillway channels, where saprolite overburden varies between 10 to 20m, and may reach 30m sometimes.

3 INITIAL OBSERVATIONS OF STRESS PROBLEMS

The first telltale signs of high horizontal stresses and strong stress anisotropy developed gradually as the project itself progressed, during the four years of construction time. Separate phenomena in different locations in the project eventually built a convincing picture of a highly stressed, narrow rock ridge in which the river meander itself presumably had acted like an 'over-coring' agent. (See the satellite photograph reproduced in Figure 3). The assumed regional stress anisotropy was concentrated in the narrow, pillar-like ridge, and with each new project-

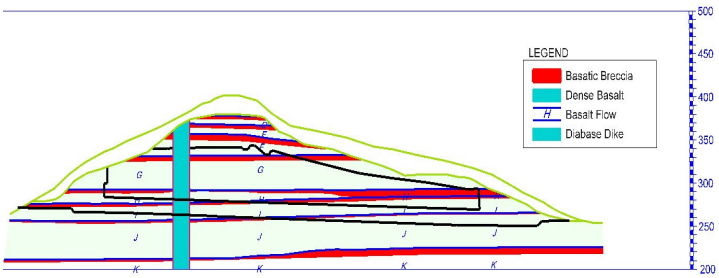

Figure 2: Geological Section through Diversion Tunnels and Spillway 2. The basalt flows exhibited different degrees of jointing and therefore stiffness.

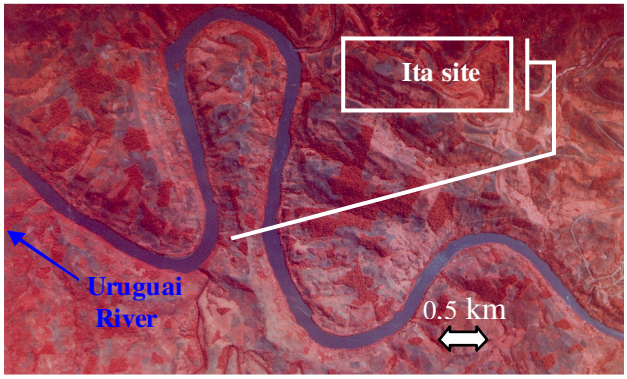

Figure 3: Satellite Image of Project Site, showing the narrow ridge where the tunnels were located.

related excavation, the engineering induced stress concentrations proved to be close to the limit of stress-induced fracturing – and sometimes exceeded the limit, despite the high strength of the basalts.

3.1 *The diversion tunnels*

It soon became clear that stresses were higher than expected. for these supposedly well known S.E. region basalts, when popping noises, some thin slab ejection, and larger than expected deformations were recorded during excavation of the five diversion tunnels, at depths of between 50 and 100m. The second author's involvement in the project began with stress analyses and deformation measurements in these tunnels. These were 15m in span and 17m high, excavated with an 8m top heading and a 9m bench.

Since stress problems and deformations were more notable as the tunnels reached their full height, the previously provided rock bolting in the arch of each top heading proved, in retrospect to be insufficient, as some areas of excessive scouring in the arch and invert were later experienced following river diversion. Several metres thickness of over-stressed rock were lost in places. These lost meters of failed rock will be back-calculated as indicators of stress magnitudes. We will also return soon to the deformations measured in the tunnels, when trying to back-figure the likely levels of stress.

3.2 *Emergency spillway stress concentration*

The second stress related problem occurred at the downstream edge of the emergency spillway, exactly above the downstream portals of the three auxiliary diversion tunnels (TD-3, 4 and 5). The 80m wide spillway had been benched down through the overburden, leaving a 20m thick, sound basalt cover (consisting of flow H) above the diversion tunnel arches. It seems that this was sufficient to concentrate the already high tangential stresses above these large tunnels, causing 50m long, curving sub-vertical cracks around the lip of the spillway, and the potential release of perhaps 30,000m^3 of rock. Bolting had been added to help secure this stress-damaged spillway exit.

3.3 *Differentiating the basalt flows*

The first author's involvement in the project started at this spillway location, with a Q-system, histogram logging of the characteristics of the four basalt flows G, H, I, J that were now well exposed at this downstream location. Most of the diversion tunnels had been excavated in the central, and most massive H and I flows where most of the 'popping' was registered. The Q-logging confirmed the significant difference in the degree of jointing between the basalt flow 'pairs' G and J (above and below) and H and I in the centre. In this case we were dealing with a 'sandwich' with a hard centre, which was perhaps responsible for concentrating horizontal stresses to an even higher level in the N-S oriented ridge (see Figure 3). The relative magnitudes of the Q-parameters in the two pairs of flows were as follows:

Flows G and J: general characteristics

$$Q = \frac{70-90}{6-9} \times \frac{1.5-2}{1-2} \times \frac{.66}{1}$$

Flows H and I: general characteristics

$$Q = \frac{90-100}{3-6} \times \frac{1.5-4}{.75-1} \times \frac{1}{1}$$

Prior to the assumption of a significant stress differentiation between the two pairs of flows, we can give the following preliminary Q-ranges of 5 to 13, and 30 to 100 respectively. If we assume general high stress for all these flows, and a preliminary SRF ranging from 0.5 to 2, the above ranges are extended to 2.5 to 26, and 15 to 200 respectively.

Correlation of such Q-values with rock mass parameters such as deformation modulus and seismic velocity are improved, following Barton 1995, 2002, by normalization with the uniaxial strength σ_c.

The normalized value Q_c is estimated as follows:

$$Q_c = \frac{Q \times \sigma_c}{100} \qquad (1)$$

An estimate of P-wave velocity (for verification with site characterization) is given by the following empirical relation for rock of low porosity, and is also shown in Figure 4:

$$V_p = \log Q_c + 3.5 \quad (\text{km/s}) \qquad (2)$$

The basalt at UHE Ita was unusually hard, with a range of uniaxial strengths of 140 to 280 MPa. If we assume a mean of about 200 MPa, the above Q-value ranges for the two pairs of flows become Q_c estimates of 5 to 52, and 30 to 400 respectively. Ranges of near-surface (nominal 25m depth) V_P are therefore 4.2 to 5.2, and 5.0 to 6.1 km/s respectively. These ranges proved, quite independently, to show reasonable agreement with the 4.2 to 5.6 km/s range for 'sound rock' measured above the future diversion tunnels many years previously.

The previously referred 'sandwich' of massive flows H and I are likely to have attracted higher levels of horizontal stress than their neighbours, and this can be indirectly assessed by the relative magnitudes of deformation modulus that can be estimated from the following equation, again for near-surface (nominal 25m depth) and low poros-

ity. (Generalizations to the above conditions: near-surface, low porosity are also available from Barton 2002, and would tend to exaggerate the different moduli between the flows due to stress effects.)

$$E_{mass} = 10\, Q_c^{1/3} \quad \text{(GPa)} \qquad (3)$$

The estimated contrasts in rock mass deformation moduli were perhaps in the range 17 to 37 GPa for flows G and J, and 31 to 74 GPa for flows H and I, in fact roughly a doubling of moduli due to the more massive nature of the central, and eventually very troublesome basalt flows. With greater horizontal stress in the H and I flows, an anisotropic distribution of moduli would probably have been in operation, but this possibility has been ignored in the simple treatment that follows.

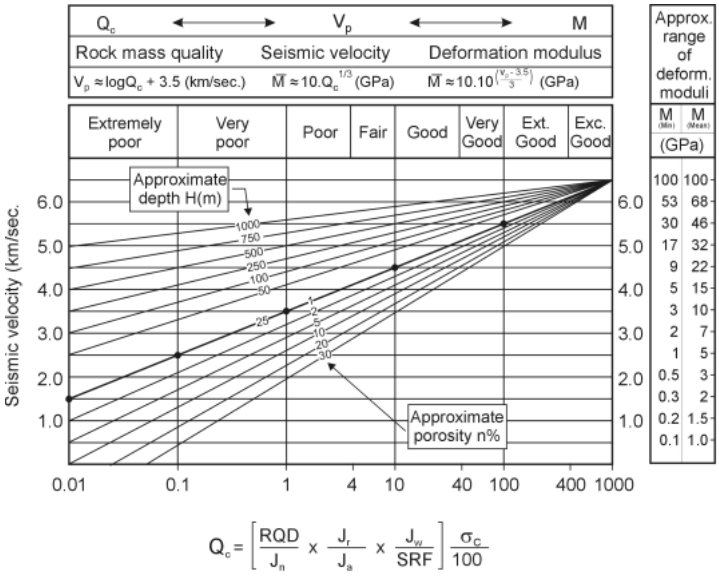

Figure 4. Inter-correlations of Q or Q_c and V_p and E_{mass} which were used for differentiating basalt flow properties. (Barton, 2002).

4 BACK-CALCULATION OF POSSIBLE STRESS LEVELS

We can first address the magnitude of the deformations actually recorded at up to twenty measurement locations along each of the five diversion tunnels. The convergences were plotted by Infanti in the $\log_{10}$ Q/ (span or height) versus $\log_{10}$ (convergence) format of Barton et al. 1994. Even at the top heading stage, the deformations, which ranged from 0.5 to 13mm, were mostly higher than expected from the central empirical trend of numerous data:

$$\Delta(\text{mm}) \approx \frac{\text{SPAN or HEIGHT}}{Q} \qquad (4)$$

In the case of TD-5, ten of the twenty instrument locations that were monitored again after benching down to the full 17m height, showed magnitudes of convergence at the triangular monitoring stations that ranged from 13 to 50mm, with a median value of 22mm, and a mean of 25mm.

Back-calculation according to equation 4 suggested much lower 'stressed' Q-values, 20mm deformation implying Q ≈ 0.8, and 50mm implying Q ≈ 0.3. So *characterization* prior to tunnel excavation, was suggesting Q-values for the massive H and I flows of the order of 15 to 200, while *classification* for tunnel design was, through back-calculation from deformations, suggesting Q-values in the approximate range of 0.3 to 1.5. We were clearly mostly within the 'stress-slabbing' SRF class of 5-50 (Grimstad and Barton, 1993) which implies a σ_c/σ_1 ratio of 5 to 3, or an 'elastic behaviour' tangential stress ratio assumption (σ_θ/σ_c) of 0.5 to 0.66, i.e. a tangential stress high enough to cause failure with rock strength scale effects considered.

In the case considered here, the major principal stress is of course σ_H and the above ratios are suggesting that its value might be in the approximate range 47 to 56 MPa, when using the 140 and 280 MPa uniaxial strengths in the logical way in relation to the above strength/stress ratios of approximately 5 to 3.

An alternative way of back-calculating the possible horizontal stress level is to use the set of empirical 'depth-of-failure' data assembled in Figure 6. With depths of failure as seen in Figure 5 in the range 2 to 3m for an average tunnel 'radius' of about 8m, we see in Figure 6 that ratios of σ max/σc of about 0.6 to 0.7 are implied when Df/a is in the range of (8 + 2 or 3m)/8 = 1.25 to 1.38. Taking σc as an average 200 MPa, the above implies that the maximum tangential stress may have been as high as 120 to 140 MPa. If we further assume relevant vertical stress ranges from about 1 to 3 MPa from 50 to 100m overburden depths, and an elastic isotropic theoretical $\sigma_{\varphi(max)} = 3\sigma_H - \sigma_v$, we obtain estimates of σ_H of about 39 to 46 MPa. The implication is therefore that the ratio of principal stresses (σ_H/σ_v) may be as high as 20 or even 25, which of course is exceptional.

Minimum tangential stresses as low as (-)29 to (-) 42 MPa, could also be calculated by the same reasoning. These would easily be enough to exceed the tensile strength of the basalts, and to cause tensile fractures on NS sides (or 3 o'clock and 9 o'clock positions) around the pressure tunnels, which were yet to be be completed. The proviso here was whether the five pressure shafts passed through sufficiently massive flows for the above Kirsch isotropic solutions to still be utilized.

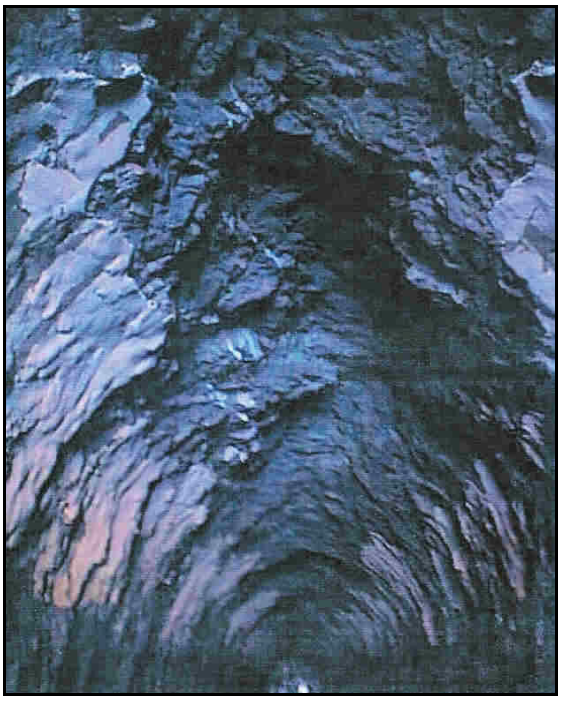

Figure 5. Example of depth of stress fracturing in one of the diversion tunnels.

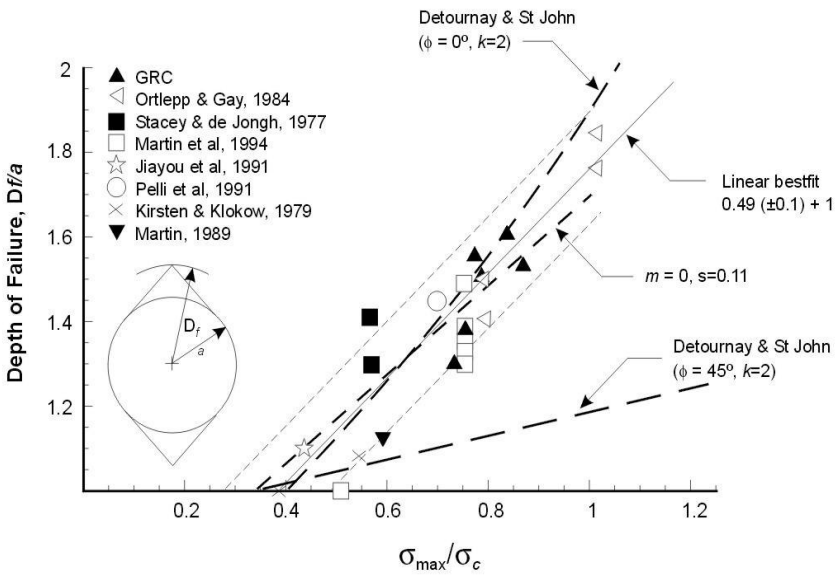

Figure 6. Empirical data for stress-induced depths of failure Martin et al.2002.

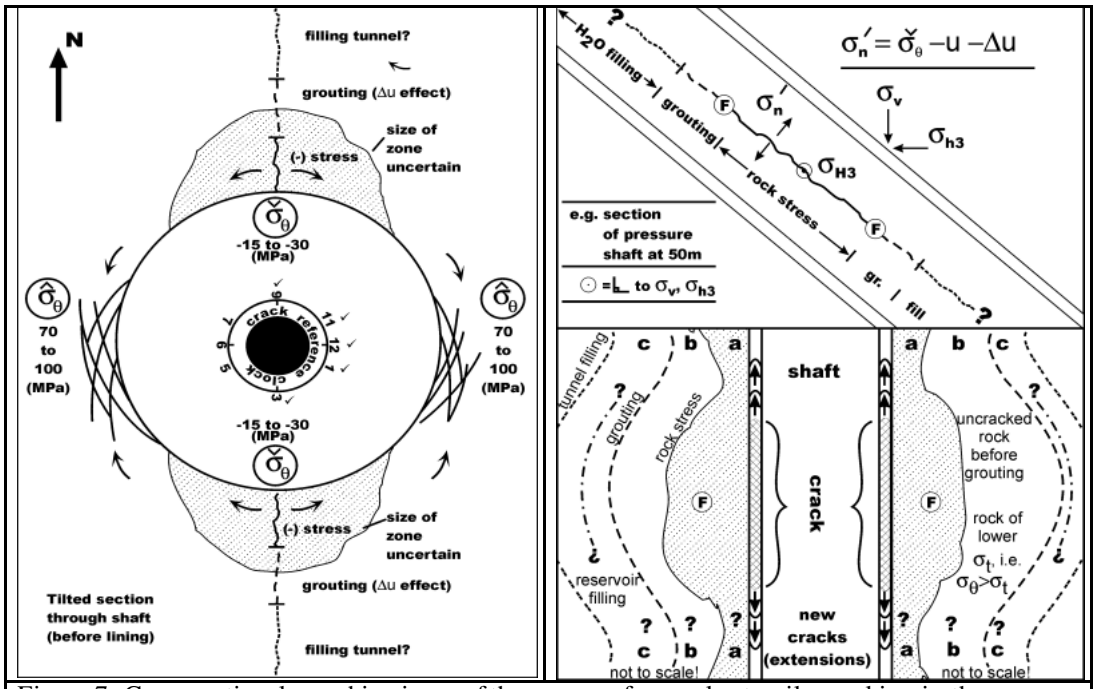

Figure 7. Cross-sectional, graphic views of the reasons for regular tensile cracking in the pressure shaft linings *prior* to their operation under internal water pressure. The mechanical behaviour becomes a coupled-process, when the grout fluid pressure is added.

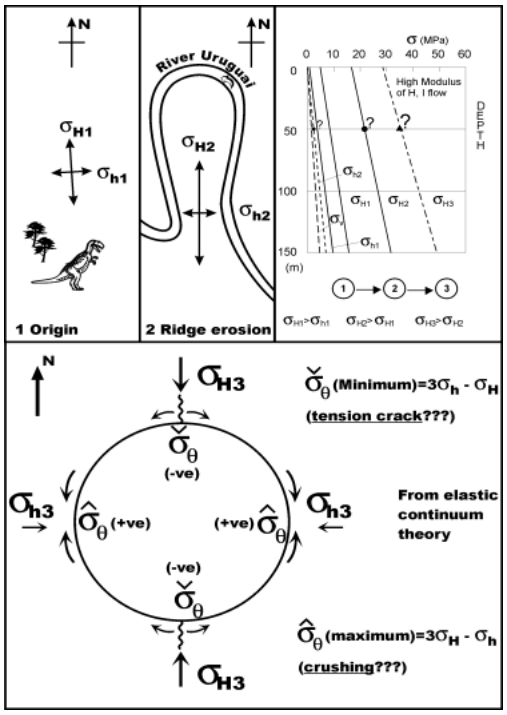

Figure 8. Summary of the probable origin of an elevated horizontal stress and stress anisotropy at UHE Ita, and the reasons for two potential types of rock failure.

CONCLUSIONS

1. Extensive and unexpected rock engineering problems occurred during construction of the 1450 MW hydroelectric project. A narrow basalt ridge separating a long meander of the Uruguai River was the site of the project.
2. Extensive tangential stress related popping and spalling was experienced when driving the auxiliary 15×17m diversion tunnels.
3. When the 9m diameter pressure tunnels: mostly as 53° shafts were driven, tangential stress related popping occurred again during excavation, but the greatest problem occurred when contact-grouting the reinforced concrete linings, which caused tensile cracking and extensive needs of repair, due to augmentation of the negative, effective tangential stress.

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