Metro construction at the most unfavourable depth caused a major metro station collapse in Brazil due to a unique sub-surface structure

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ABSTRACT: A metro project that was constructed in the most difficult elevation possible, with constantly changing rock-to-saprolite-to-soil-to-rock conditions, due to São Paulo metro operator requirements, suffered the predicted consequences of severe overbreak and slow progress. On two occasions there was break-through to surface. This paper describes one of these events that involved a set of adverse circumstances that tragically converged in time and location. On January 12th 2007, the following dramatic accident occurred. Nearly the whole of one of the station caverns of 40 m length and 19 m span suddenly collapsed. Despite extensive drilling around and even within the cavern centre, a misleading top-of-rock elevation was indicated, giving an assumed average 3 m of rock cover above the arch of the cavern, beneath about 18 m of saprolite, soil and sand. Heavy lattice girders at 0.85 m centres and steel-fibre reinforced shotcrete of minimum 35 cm thickness were used as primary support. Subsequent excavation through all the collapsed rock and soil during 15 months of investigations revealed a previously undiscovered, 10-11 m high ridge of rock with adversely oriented steep sides, caused by differential weathering of the foliated gneiss and an amphibolite band. A secondary planar joint set, a major bounding discontinuity, and probable elevated pore pressure from a cracked storm drain, constituted an unpredictable set of adverse conditions at an adverse location beneath a road, causing the death of seven people when sudden collapse occurred. Lessons learned the hard way confirmed the prior opinions of several prominent consultants who had called for either shallower, or deeper construction, either options in order to avoid frequently changing mixed-face conditions, which create a range of unnecessary difficulties.

1. INTRODUCTION

There are several possible choices for expansion of metro lines involving the addition of new stations in major cities. The most difficult from the point of existing infrastructure and buildings is of course cut-and-cover. In less developed parts of cities under expansion, this is nevertheless the most viable option, and there are many examples from around the world. We can then consider two remaining basic options: shallow tunnels with stations developed from large diameter shafts or deep tie-back excavations, and the third option of deeper construction, probably entirely in rock, with all major developments from the underground including the station caverns. In this third option there remains the need for an inclined escalator shaft, or in a few cases vertical lift shafts. These of course have to tackle soil, saprolite and rock transitions, but they are of limited dimensions, making for a faster and cheaper project.

From a tunneling viewpoint, the second option is by far the most complicated, as deep but differential weathering may mean frequent mixed-face tunnelling and cavern construction. In the present expansion of the São Paulo Line 4, there is an example of a station with one end entirely in rock, and the other entirely in soil. Photographs from construction of this (Butanta) station are reproduced in Figure 1, to emphasise adverse conditions even in the end in rock. The main topic of this paper is however what happened at the next station. On the afternoon of Friday 12th January 2007, a dramatic accident occurred at the next station (Pinheiros) along Line 4 of the São Paulo metro, about 1 km away on the other side of the Pinheiros River. Nearly the whole of one of the station caverns of 40 m length suddenly collapsed, immediately followed by collapse of nearly half of the adjacent 40 m diameter and 35 m deep station shaft. The reasons are clear after the event.

Figure 1. Top: Severe overbreak in Butanta Station cavern arch, due to proximity to sub-surface saprolite. Bottom: The dramatic consequences for final concrete lining thickness.

Figure 2 shows the situation some months before the collapse, before construction of the first bench in the station cavern to the east of the shaft. The ill-fated Rua Capri is marked by the line of trees behind the gantry crane. The multiple accident shown in Figure 3 occurred so rapidly that there was no time for warning to be given. The seven unfortunate victims died after falling from the surface and becoming deeply buried under the collapsed rock and soil. It took firemen and consortium CVA rescue teams a total of 12 days to recover all the bodies following burial under 15 to 20 m of soil and weathered rock. Five of the victims were in a mini-bus that had been

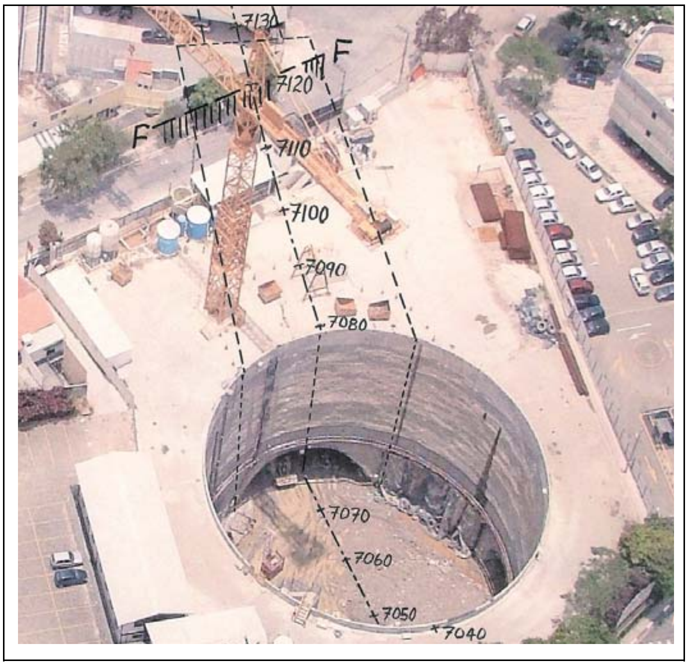


Figure 2. Aerial view of the Pinheiros Station shaft. The black curved arch at 20m depth down the shaft wall is the top-heading of the eastern station cavern. The shaft wall chainage at 7080 m and the rear discontinuity (FF) at chainage 7120 m, mark the approximate limits of the 40 m long cavern collapse.

driving along Rua Capri, which crossed the eastern end of the station cavern. Another victim was an elderly pedestrian in the same street. The seventh victim was a lorry driver from CVA.

It is probable that suction, caused by the rapid fall of a huge undetected ridge of jointed, foliated and often deeply weathered rock weighing some 15,000 to 20,000 tons, actually sucked the Rua Capri victims to a lower level in the debris than they would have fallen if materials had been more uniform. The rapidity of the collapse caused an air blast in the running tunnel, sufficient to knock down a distant fleeing tunnel worker. A ‘piston action’ is assumed.

It may be that sudden collapse of a temporary soil/saprolite arch beneath Rua Capri immediately above the falling ridge of rock, was the reason for

Figure 3. 12th January 2007. The station cavern collapsed suddenly beneath Rua Capri (in right foreground). This was followed by the simultaneous collapse of approximately half of the 40 m diameter shaft wall. White arrow is along cavern axis.

not being able to avoid being sucked into the void. Rescue teams worked mostly from the running tunnel adjacent to the cavern, some 30 m beneath the collapsed section of Rua Capri.

2. BOREHOLES FOR SITE INVESTIGATION

Prior to final design and construction of the 19 m span station cavern, numerous boreholes had been drilled through the soil, saprolite and gneiss. The four boreholes located close to the sides of this eastern station cavern, and one in the centre of the cavern (Figures 6a and b), had indicated some zones of deeply weathered rock, especially in the biotite gneiss. This had been expected from investigations elsewhere along Line 4, also ten years previously. These five local holes, around and within the planned excavation, consistently indicated unfavourably low rock cover above the planned 40 m long cavern arch, located on the eastern side of the shaft.

Figure 4. The appearance of the rock core from the borehole 8704 that was drilled near the centre of the (future) station cavern. The eighteen plastic containers contain the (minimal) recovery of 18 m of overlying sand, soil and saprolite. The rock head, encountered first at 18 m depth, at elevation 706m, was weathered gneiss. The sub-vertical foliation is not indicated.

3. SUB-SURFACE RIDGE OF ROCK THAT WENT UNDETECTED DESPITE MANY HOLES

On average the planned cavern arch was at a depth of 21 m below the surface soil, which was at an elevation 724 m. The central borehole (number 8704) drilled near the centre of the future cavern, had correctly indicated a (local) top-of-rock elevation of 706m. This was the same as the *mean* rock elevation found in the four other closest holes. The arch of the planned Pinheiros station was at elevation 703m, giving an assumed 3 m of rock cover over the arch. This was the reason for *not* using bolts in the pre-planned B+S(fr) temporary support. Heavy lattice girders were used instead, with a minimum of 35 cm of good quality S(fr).

Figure 5 shows what was *expected on average* concerning top-of-rock elevations, when a diagram-

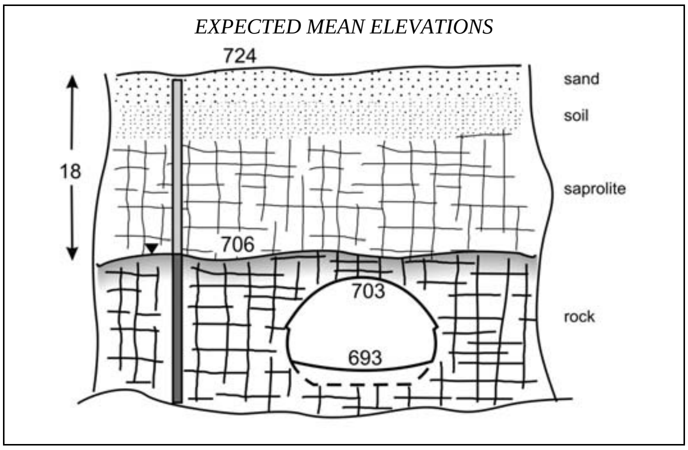


Figure 5. Most of the closest boreholes were drilled from 723-724 m surface elevations, and rock was reached between 706-707 m in the majority of cases.

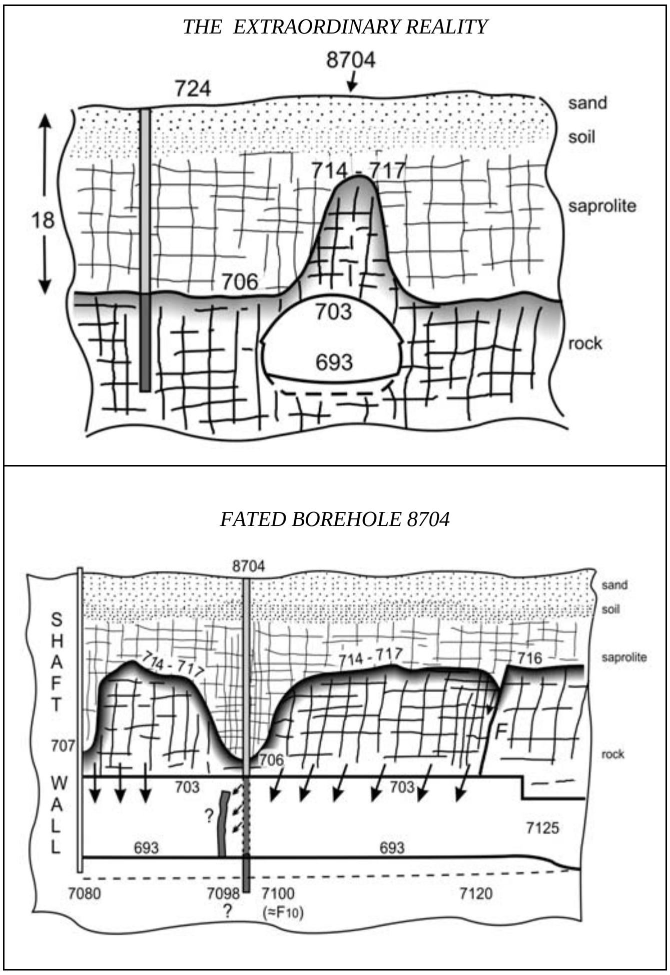


Figure 6 a and b. Most of the collapsed rock in the centre of the cavern fell 10 m, to a top elevation of 704-707 m, i.e. remaining 1 to 4m above the (original) cavern arch. The ridge-of-rock was missed due to the fated location of borehole 8704. Drilled at ch. 7100, it reached rock at 706 m. This was the same elevation as the mean top-of-rock elevation (706 m) of the five nearest boreholes. Low cover was 'confirmed'. B+S(fr) was therefore rejected as temporary support as too little rock cover.

matic vertical cross-section is stripped of all geological detail. A uniquely adverse sub-surface ridge of rock with steep sloping sides was eventually discovered along the axis of the cavern *after* the collapse. The ridge proved to be an astonishing 10 to 11 m in height, in relation to the surrounding top-of-rock levels, running for many tens of meters along the cavern and even along the running tunnel. It appears to have been divided into two unequal ‘halves’ close to the location of the fated 8704 borehole, as sketched in Figure 7.

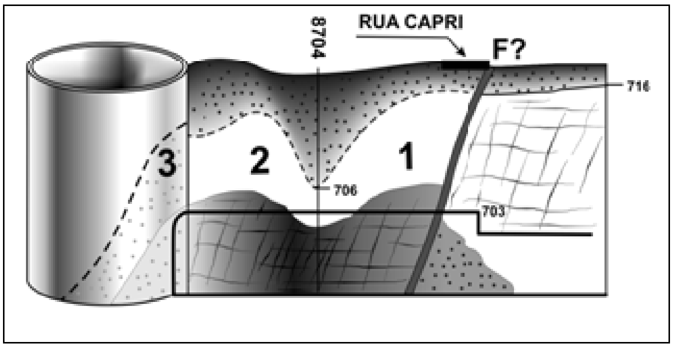


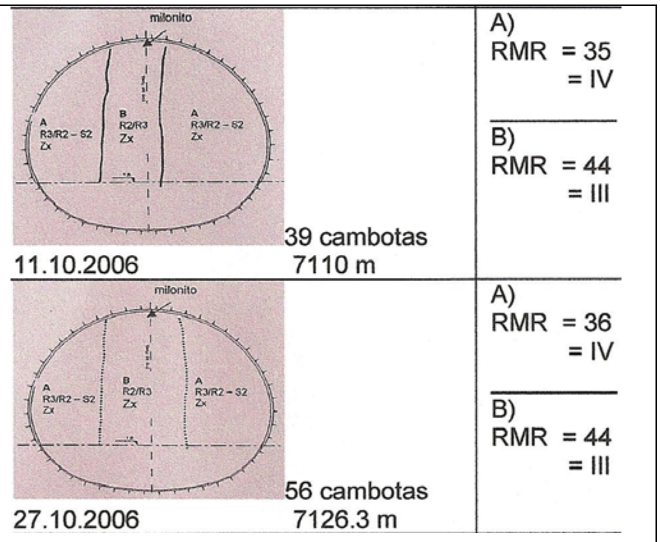
Figure 7. A simplified interpretation of the fallen rock, in the form of two large ridges. A bounding discontinuity, marked F, marked the eastern limit of the collapse, beneath Rua Capri.

Even after falling at least 10m during the collapse, and crushing the heavy station cavern support, and flattening an excavator down in the cavern, the two fallen ridges of weathered, foliated and jointed gneiss and amphibolite came to rest with a top elevation as high as 704 m to 707 m. The sketches in the longitudinal direction given in Figures 6 and 7, show why the ridge-of-rock or ‘seismic tower’ was missed by borehole 8704. Remarkably, the investigating institute (IPT, 2008) working for the prosecuting authorities, did not notice, or ignored these elevation discrepancies in their official report, which ran to 46 volumes and 3000 pages. The geology was clearly not “more or less as predicted” as they have claimed.

4. ROCK QUALITY RMR LOGGING

During construction of the eastern station cavern, geologists had registered an increasing volume of medium quality class III rock (Bieniawski’s rock mass rating RMR = 44-48) in the centre of the cavern in the direction of Rua Capri. This ‘core’ (B) of improved rock is indicated by simplified cavern face maps, two of which are shown in relation to cavern progress in Figure 8. Each logging sheet also showed all RMR ratings and all joint orientations.

The Class III ‘core’ mapped along almost the whole length of the cavern centre was surrounded by poorer quality Class IV rock (RMR= 34-36) on either side (as classes A/B/A). That this better quality rock ‘core’ could be a threat to cavern stability was not of course imagined, since an average 3 m thickness of rock cover had been ‘confirmed’ by all boreholes. The closest holes resembled Figure 4. With the benefit of hind-sight (following the collapse), the possibility of differential weathering was considered, and during 15 months of painstaking digging



(“cambotas” are Portugese for lattice girders)

Figure 8. Examples of the the RMR rock class values logged by geologists at the cavern face, with differentiation of the ‘core’ (B) and the surrounding rock (A). Note increasing width.

and excavating through the fallen materials, a (previously) high ridge of rock that had fallen 10 m was indeed indicated, in contradiction to earlier borehole evidence. A previously high ridge of rock means that the upper parts reaching nearer to the surface, would have been deeply weathered, especially if of lower quality. This concept, and its possible development over a sub-surface *geomorphological* time scale, is illustrated in Figure 9 (from Barton, 2008).

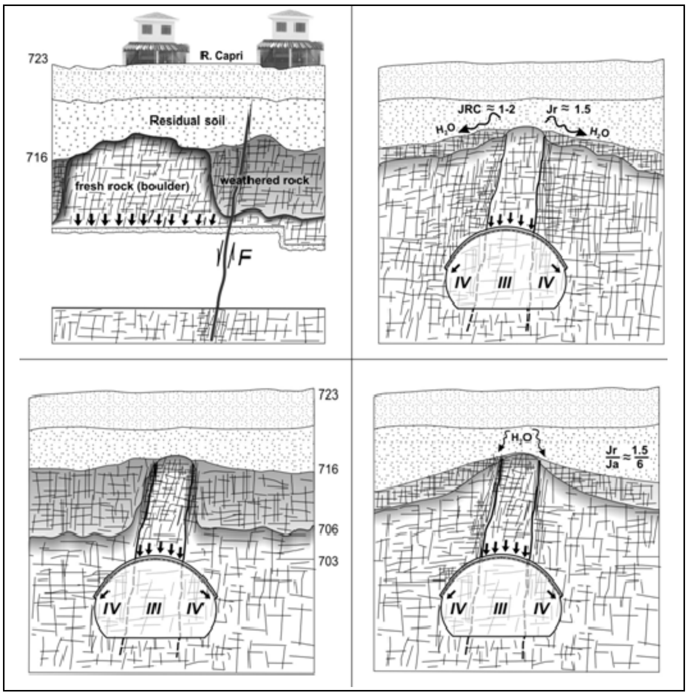


Figure 9. Conceptual progressive weathering models that were developed as possible explanations of the gradual development of *differential weathering*, eventually leaving a ridge (and wedge) of rock that would threaten stability due to prevention of arching i.e. little of the usual stabilizing, curving, tangential stress above the arch. The first sketch (with added discontinuity and houses) was adapted from a 300 m distant seismic profile that was achieved in a quieter street. No velocities were reported in any of the IPT investigations of 10 years previously.

5. HEAVY PRIMARY SUPPORT FOR THE STATION ARCH

Because of some weaker rock drilled through at the sides of the 19 m span cavern, conservative assumptions were made for the foundation strength and stiffness of this rock beneath the footings of the lattice girders. The so-called 'elephant feet' supporting the structural arch, assist in sharing the load from the over-lying rock mass, and are placed in large excavated recesses in the rock, on concrete plinths at either side of the cavern, as indicated in Figure 10. Figures 11 a and b show typical execution.

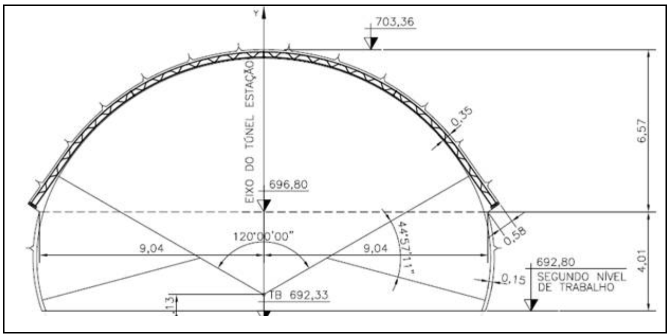


Figure 10. Lattice girders were at 0.85 m c/c, and embedded in more than 35cm of S(fr). Note 'elephant-feet' recesses.

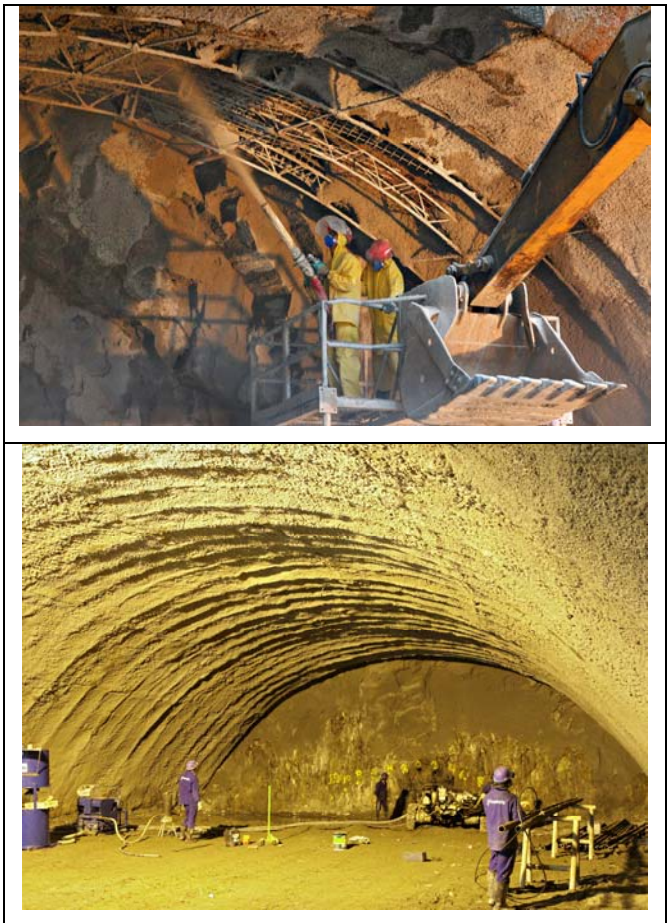


Figure 11. A view of the heavy primary support in the top heading of the eastern station cavern. Lattice girders were at 0.85 m c/c spacing, embedded in at least 35 cm thickness of steel fibre-reinforced shotcrete. The pre-injection process at the cavern face (ch.7126 m) was for the pending intersection with the running tunnel to the east, just beyond Rua Capri.

The consortium CVA consisting of principal Brazilian contractors, excavated with small drill-and-blast advances and applied the successive structural support elements up to the face, followed by shotcreting. An earlier Basic Design lattice girder spacing of 1.25 m c/c was rejected because of the loads resulting from the *assumed inadequate rock cover*, as the usual and desirable arching in the rock above the cavern was expected to be much reduced.

A lighter and cheaper primary support alternative for the cavern, consisting of rock bolt reinforcement of the rock arch, and significantly less thickness of fibre-reinforced sprayed concrete was also rejected, since the five closest boreholes had indicated a mean top-of-rock elevation of 706m, only 3m above the cavern arch roof, considered insufficient for conventional support with rock bolts, since this rock was also deeply weathered.

Final support of this large station cavern was to have consisted of steel-reinforced concrete cast against a membrane. However this stage of construction had not been reached at the time of collapse, neither in the eastern nor western station caverns, nor in the central station shaft.

6. POST-COLLAPSE EXCAVATION REVEALS RIDGE OF ROCK ALONG CAVERN AXIS

During most of 2007 and the first 3 months of 2008, the fallen soil, saprolite and finally rock, sketched earlier in Figure 7 was carefully excavated and recorded, under the supervision of the investigating institute IPT, working on behalf of the police. This excavation occurred from the base of an increasingly deep open excavation, supported eventually by hundreds of tie-backs. (Figure 12a). This has now been completed as a cut-and-cover station platform, with several floors of concrete above platform level.

Figure 12 a,b. Appearance of the open excavation towards the end of 2007. A (fallen) ridge of rock had now become visible.

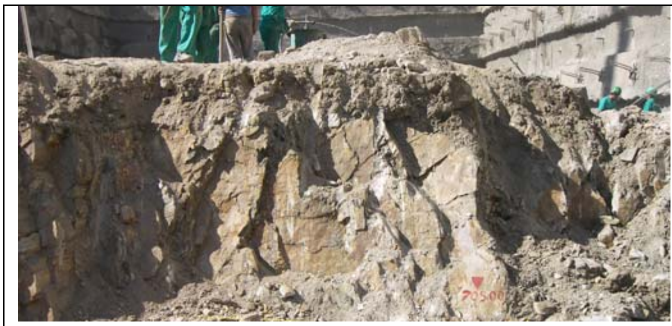


Figure 13. Despite falling as much as 10 m this ridge of rock still rests at a top elevation of 707 m (see red 705 m elevation mark). This suggests that it was previously at elevation 717, or 11 m above *assumed* mean top-of-rock levels.

Figure 12b shows the side of a 1-1.5 m high central ridge or 'core' that had a loosened appearance due to its fall of also about 10 m. It exhibits a curved, smoothed appearance due to deep weathering. This is assumed to be 'remnant Class III', with reference to the 'core' of better quality rock logged along the cavern centre, with increasing width towards the eastern end of the cavern (e.g. Figure 8). In Figure 14 a cross-section of the fallen ridge of rock is shown, indicating its previously high elevation.

7. DIFFERENTIAL WEATHERING THE KEY CAUSE OF A COLLAPSE MECHANISM

Differential weathering along the sides of the 10 to 11 m high ridge of rock was identified during this post-collapse excavation. At some distance above the cavern arch, this unidentified wedge-shaped ridge had developed into a threat to stability, due to its adversely sloping soil and clay-filled boundaries, which must have had insufficient shear strength. These adverse boundaries hindered arching, and instead stood ready to supply a huge load onto the lattice girder and steel-fibre reinforced shotcrete support. The loading of the lattice girders from a variously weathered and jointed 'ridge-of-rock' is illustrated in Figure 14. There is saprolite to each side.

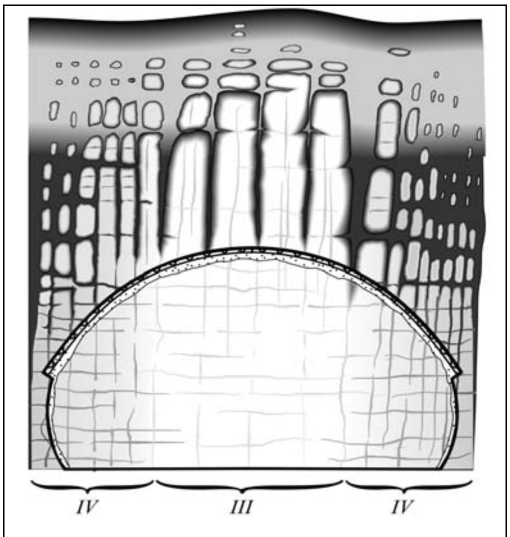


Figure 14. A sub-surface weathering sketch, based on a Linton, 1955 explanation of the development of 'tors'.in SW England.

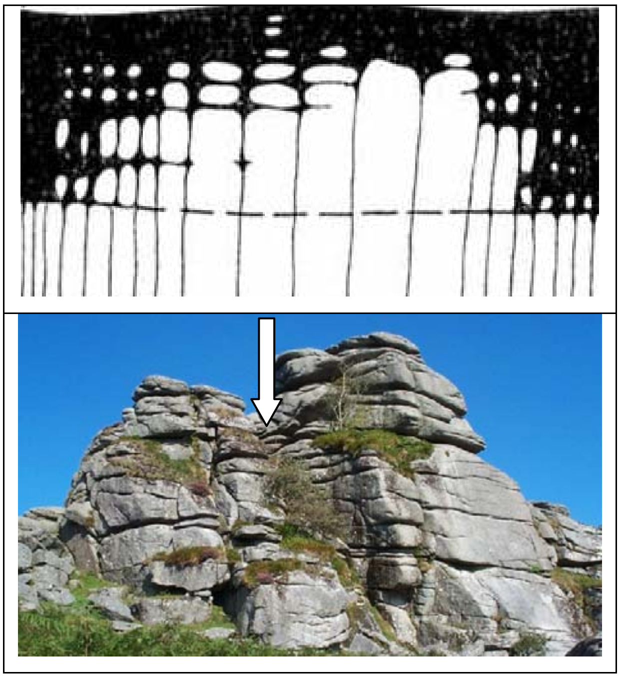


Figure 15. Core-stone phenomena in massive granites. The black-and-white sketch is from Linton, 1955. ('The problem of tors'). The arrow shows (conceptually) hole 8704 location, between two towers (in this case remnant towers or 'tors' of granite from SW England). Despite the *much less massive* nature of gneiss as found at the Pinheiros cavern collapse, the remnants of more jointed, and differentially weathered structures were clearly evident during the stage-by-stage excavation.

Figure 16. The running tunnel to the east of Rua Capri (looking back in the direction of Pinheiros station) proved, in retrospect, to also display signs of a 'core' of better quality rock, at approximate photographed chainages of 7170 m and 7150 m (this photo). This suggests a unique sub-surface ridge of higher quality rock, stretching some 80 to 90 m, *miraculously also following the gentle curvature of the tunnel.*

8. THE COLLAPSE MECHANISMS SEEN IN THE COLLAPSED SUPPORT

The collapsed parts of the cavern's structural support were eventually reached at post-collapse excavation elevations of 693 to 695 m, immediately above the original cavern floor level of 693 m. The cavern had been excavated to a height of 10 m when the collapse occurred. A final bench excavation remained to be excavated below this level, in mostly quite

Figure 17. Evidence of elephant-footing failure due to presumed stress-fracturing of the gneiss beneath the footings (left wall). See later numerical modelling of this possibility.

sound gneiss. A cleaned cavern floor will be shown later. Evidence of extreme over-loading of the structural support, causing its final collapse, was eventually exposed near the base of the excavations. In part of the cavern there was evidence of footing failure, meaning fracturing of the rock, followed by folding and inwards displacement of wall shotcrete and mesh. An example is shown in Figure 17.

Figure 18. Evidence of 'plastic-hinge' development in the lattice girders. Note the remnants of pre-grouting tubes just above the collapsed shotcrete (and lattice girder) support.

There was more extensive evidence of extraordinary 'punch-loading' of the heavy arch support, with multiply folded layers of structural support, and even of 25 mm and 32 mm lattice girder steel failed in tension. This is evidence of extremely unusual, and probably high-velocity loading levels.

The 'folding' of the lattice girders shown in Figure 18 stands in stark contrast to the apparently robust support shown during construction, in Figure 9. It also emphasizes all tunnel designers *taken-for-granted reliance on arching*, with compressive tangential stresses combined with sufficient shear strength, generally taking most of the assumed load.

Figure 19. An example of three superimposed ('folded-sandwich') sections of support (arch-wall-arch) now all squeezed within a 1.5 m thick collapsed layer, just above the muck of the original floor of the uncompleted cavern.

Drawings made by IPT field engineers reproduced in Figure 20 show this 'sandwich' failure phenomenon, that suggests the development of 'plastic hinges' where the arch lattice girders-and-shotcrete were overloaded, sometimes with these failed elements getting below the failed mesh reinforced wall S(mr), shown dark in Figure 20 (top). The development of 'plastic hinges' is suggested.

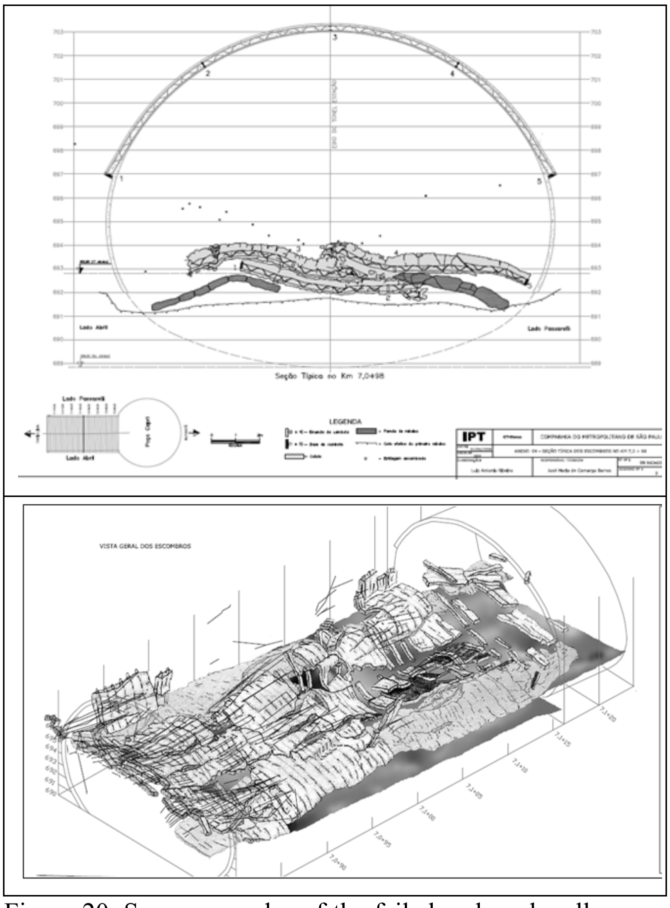


Figure 20. Some examples of the failed arch and wall support, as revealed at the bottom of the 20 x 20 x 40 m approx. of excavation through fallen soil, saprolite and jointed rock. Despite all this careful recording, the ridge-of-rock elevation discrepancy in relation to borehole evidence was ignored. IPT, 2008.

9. COMPUTER MODELLING OF COLLAPSE MECHANISMS WITH FRACOD AND UDEC

The mechanisms of such structural failure could be partially demonstrated in post-collapse discontinuum (jointed rock mass) modelling, and in stress fracture modelling of the over-loaded 'elephant footings'. These models were performed by world experts in their field, and showed multiply over-loaded structural support, and extremely cracked foundations of the footings, when realistic levels of rock strength, fracture toughness, and exceptional rock ridge loadings of up to 20,000 tons were modelled. Two examples of the footing failure models are shown in Figure 21.

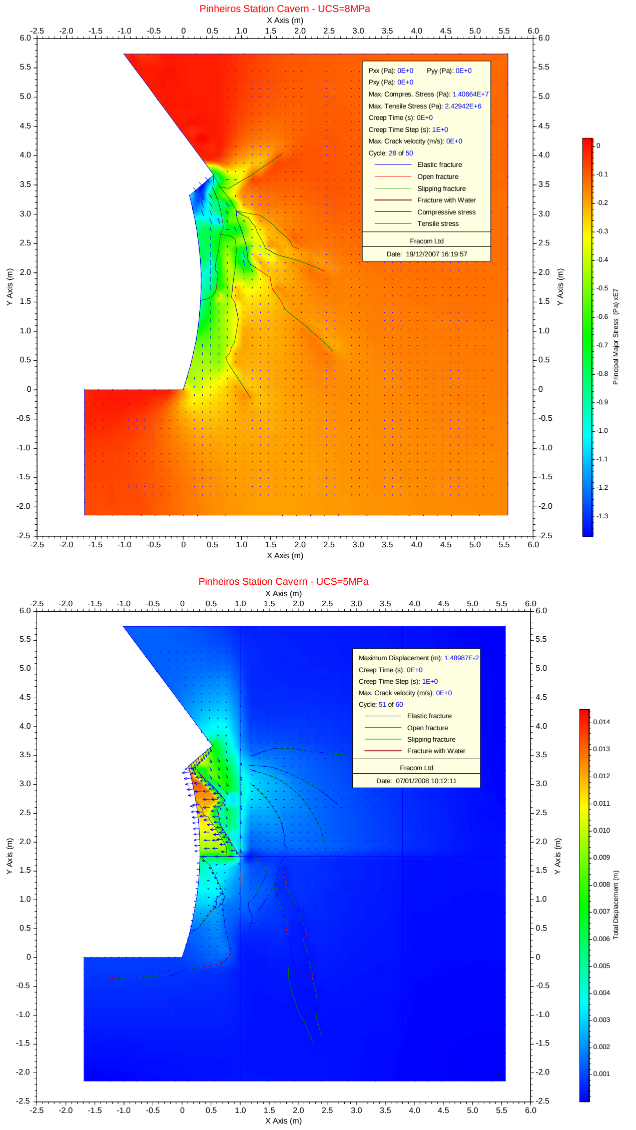


Figure 21. Examples of FRACOD modelling of rock fracturing beneath the 'elephant-footings', caused by over-stressing of the lattice girders in the cavern arch, caused by the unknown elevated ridge of rock. Dr. Baotang Shen, 2008.

Dr. Baotang Shen, the developer of the BEM fracture mechanics code performed this FRACOD modeling for NB&A. There was no cracking in any of the three cases (UCS = 5, 10 or 15 MPa with low moduli) when *low* load levels were applied (three of nine cases), as reasonably expected in the design. Cracking increased as load levels approached those applied by the ridge of rock, assuming limited arching.

The larger-scale modelling of possible over-all cavern failure, due to over-loaded support, was performed with the UDEC distinct element computer model. Since a quite high level of information about the structure of the rock mass: the jointing, faulting, strength and stiffness is required, this code was not used in CVA design studies, due to the limits imposed by investigation via small-diameter drillcore.

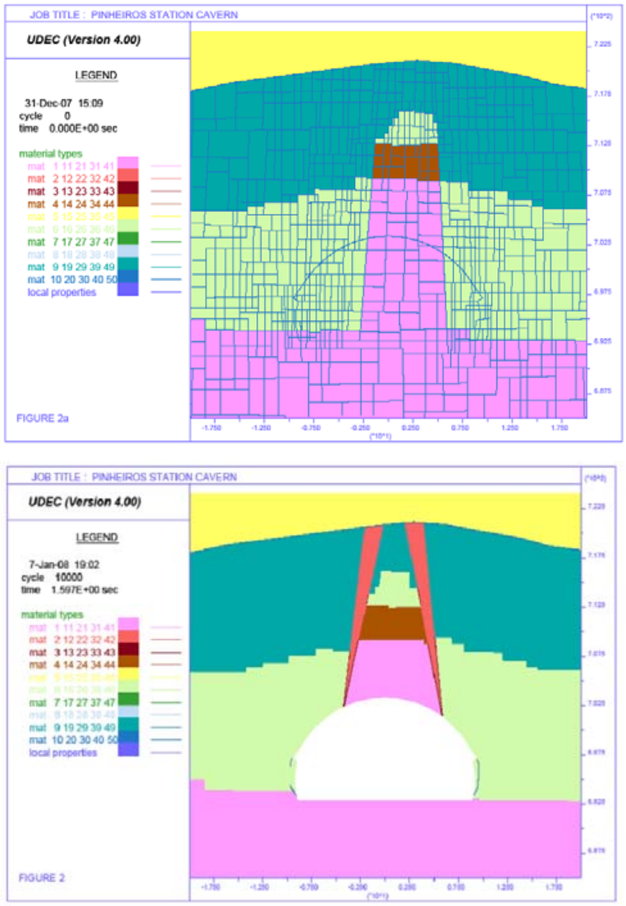


Figure 22. Division of the rock mass into idealized blocks by modelling the jointing and foliation. The colours represent different strength and stiffness assumptions. (Dark blue is saprolite). The upper model did not cause collapse: the increasingly thick wedge of weathered material (red colour) seen in the lower model was required, together with $K_0 = 0.5$ (lower horizontal than vertical stress).The UDEC modeling was performed for NB&A by Prof. Stavros Bandis.

Failure did not start by the time the top-heading was excavated but stresses were high. When over-loaded lattice-girder elements were softened after the first bench was excavated, general failure commenced, as seen in Figure 25. In practice this failure was very rapid, creating an air blast, perhaps even sucking victims from the surface to greater depth than they would have fallen by gravity alone.

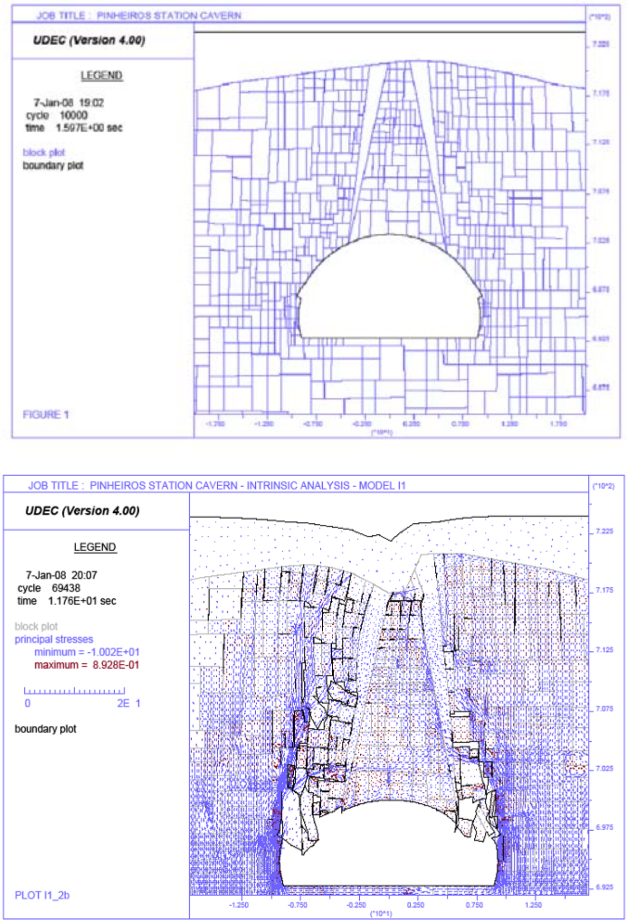


Figure 23. Preliminary modelling without rock support, to demonstrate the gross failure mechanism and actual need for even heavier support than the conservative solution adopted.

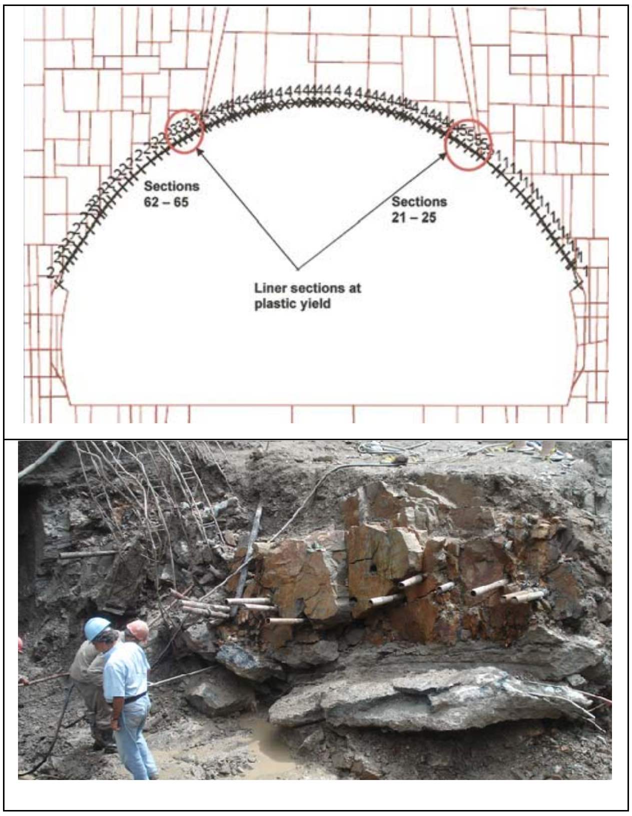


Figure 24. The typical locations in the arch support where over-loading (and plastic yield of the lattice girders) were consistently modelled. Failure beneath the 'elephant-footings' was also modeled in approximate terms in these UDEC studies. The expected improving effect of pre-injection was not modeled.

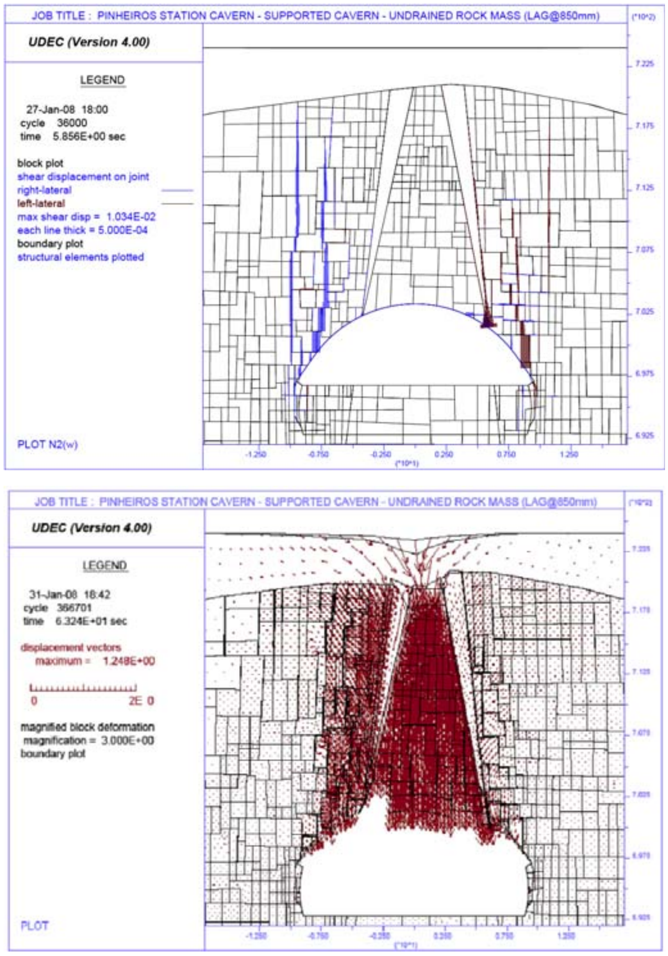


Figure 25. Models of the top heading and first bench, showing shearing and deformation, and the development of massive failure when over-loaded elements in the arch were softened.

10. CONTRIBUTORY ADVERSE FEATURES CLOSE TO RUA CAPRI

A multiple collapse of this magnitude, occurring with a speed sufficient to cause an air-blast that blew over a distant fleeing tunnel worker, obviously required other adverse features for it to occur at this location. As in recent airliner accidents, there was a convergence in time and space of unpredicted elements, that perhaps separately could have been predicted, but joined together were beyond imagination.

There were three such additional adverse elements exactly beneath Rua Capri. Taken alone these additional factors would not have been a threat to stability, but *in unexpected combination* they caused one of the largest urban civil engineering tunnelling accidents on record. The triggering mechanism for this loading to be released proved to be totally unexpected.

10.1 A planar discontinuity

Geological faults or major discontinuities crossing tunnels or caverns occur so frequently that the tunnelling industry developed standard support

Figure 26. The eastern boundary of the collapse, beneath Rua Capri, was marked by this planar discontinuity that paralleled the dominant joint set crossing the cavern almost at right-angles. By chance it crossed a storm drain that was gently flowing after the collapse.

measures long ago. In the case of Pinheiros, a major and rather smooth-planar discontinuity crossed the cavern at a steep and nearly perpendicular angle (Figure 26). This is most favourable in normal circumstances. The particular discontinuity was not a geological feature that had been noticed until after the collapse, where it formed the upper meters of the rear failure surface beneath the Rua Capri pavement. It was not a fault as first suspected: there was no trace of it in the eventually cleaned cavern floor (see last figure of paper).

At cavern level 20 m below, this feature presumably did not distinguish itself from the smooth, planar set of rock joints that consistently crossed the cavern at the same steeply-dipping angle (as seen in Figure 12b). The standard heavy support was continued to the eastern end of the cavern. In this end of the cavern, beyond the rear discontinuity, no collapse in fact occurred.

10.2 A cracked storm-water pipe

The unpredictable event that *triggered* the massive instantaneous failure along the multiple adverse rock structures lying undetected above the cavern is believed to be the cracking of a 30 years-old 700 mm diameter storm water and sewage pipe that crossed the same discontinuity exactly beneath Rua Capri (see Figure 26). Compounding the situation was the fact that this artificial fluid supply was located immediately following a change of cross section of the pipe, from 1000 mm to 700 mm. (Figures 27 and 28). This represents a 50% reduction in flow area, which probably caused an elevated water pressure and unwanted water supply in just the wrong location, if and when rainfall was high.

10.3 Pore pressure and strain softening

Naturally there had never before been a cavern under this discontinuity marking the eastern boundary

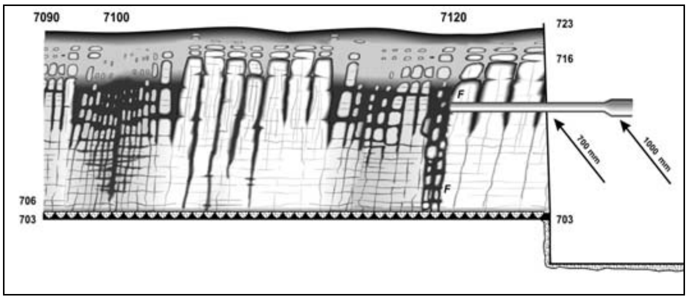


Figure 27. The fractured pipe, with the change of cross-section that possibly raised the water pressure in the most adverse location. The location of Rua Capri corresponds to the 7120 chainage in this conceptual sketch of most of the ridge-of-rock. The low point sampled by borehole 8704 is at chainage 7100.

of the collapse. It is surmised that there may have been a small down-dip shear deformation as a result of the approaching and passing cavern. This can never be prevented, and may indeed occur ahead of the tunnel faces, where deformation has already begun. It is likely to have been of small millimetre-scale magnitude. However, it may have allowed storm water to flow more easily into the discontinuity, many meters above the cavern, due to shear-induced dilation, possibly transmitting water under pressure further into the unknown, adverse rock/soil structures. It is notable that a concrete apron or pavement (Figure 2) covered the future eastern cavern area, so ‘piped water pressure’ could have been a more active source than groundwater rising.

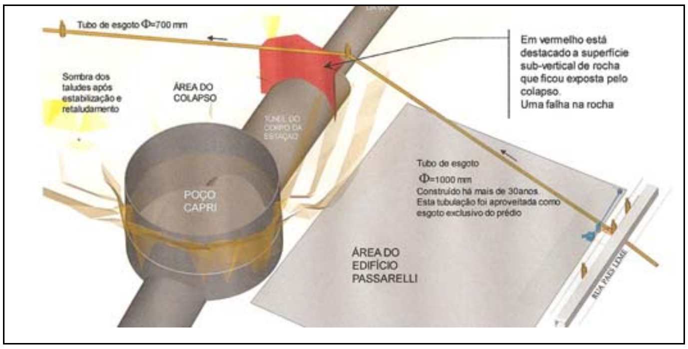


Figure 28. The storm drain that fate determined should cross the geological discontinuity surface (red) at the rear margin of the collapse, with reduction of this tube to half its cross-section at just the most adverse location, if close to other adverse features lying unknown above the cavern axis.

Rainfall was unusually heavy for many days, approximately three weeks preceding the collapse, but in the last week just before the collapse, it was not unusual – it was just wet. It is surmised (Barton, 2008) that strain softening of clay surrounding the differentially weathered ridge-of-rock, and the *assumed* ‘piped’ adverse pore pressure effects, may therefore have initiated some weeks before the collapse occurred.

A residual artificial water supply was seen flowing from the broken pipe (Figure 26) in a video film taken immediately after the collapse. Such a source, at higher pressure, could three weeks previously have helped to soften and lubricate the weathered clay-like boundaries of local parts of the adverse wedge-shaped ridge of rock, which ran undetected above much of the cavern arch. But we will never know for certain, why the collapse occurred exactly on 12th January 2007.

10.4 Planar joints crossing the cavern

The block release surface at the other end of the ridge of rock may have been the deeply weathered boundary between the two 'halves' of the ridge, in the approximate location of borehole 8704, at an original chainage of 7,100 m. This is sketched in Figures 6 and 7. Alternatively there could have been 'down-stepping' across the smooth steeply dipping cross-joints (JRC 2 to 4, seen in Figure 12b), that crossed the cavern in numerous locations. The second smaller rock ridge (Figures 6 and 7) effectively had the shaft wall as its western release surface.

A final unexpected factor that may have *compounded the scale of collapse* at Pinheiros, was this distant 75° to 80° dipping rear discontinuity under the eastern pavement of Rua Capri. Although nearly 40m from the shaft, the down-dip component of sliding during the 10 m 'vertical' collapse, may have pushed *both* the falling ridges of rock some meters towards the side of the shaft, thereby further guaranteeing the shaft's partial failure, when only having at this stage, a temporary S(mr) lining.

11. A GEOPHYSICS AND ROCK QUALITY BACK-ANALYSIS OF THE SUB-SURFACE

Seven lanes of traffic and twin rail lines shown in Figure 29, made application of *seismic refraction profiling* to supplement the extensive drilling a very unreliable prospect. At a limited number of quieter locations in the city, *SRP* was attempted by the IPT institute investigating the route 10 years before on behalf of São Paulo Metrô, but velocities could not be estimated, nor were given. Since a consistent bedrock surface was indicated by the six nearest boreholes to the cavern, one on its central axis, no additional seismic investigations were performed by CVA.

IPT who were now involved for the prosecuting authorities, had however performed a three-dimensional study in the late nineties, of permeability and seismic velocity, using seven boreholes for their 3D hydro-tomography, and three boreholes for their cross-hole velocity measurements.

The earlier cross-hole measurements (down-hole with little noise), shown in Figure 30 were compared

Figure 29. Aerial view of Pinheiros station prior to construction. It is close to a lot of traffic arteries, which made seismic refraction impossible. However, cross-hole measurements were performed close to the river, near the western station cavern, ten years prior to construction, by the same institute IPT.

with independent post-collapse Q- logging of the five boreholes nearest to the cavern. The results, and comparison with the geologist's RMR-logs of the advancing cavern face prior to collapse, are compared in Figure 31.

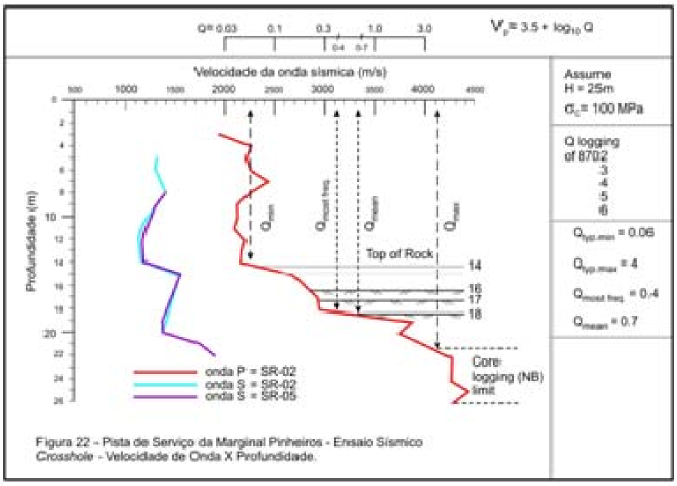


Figure 30. Cross-hole seismic velocity (V_P red, V_s blue) measured close to the western station cavern at Pinheiros, as a function of depth, with comparison to Q-values logged by the writer using the eastern cavern borehole cores. The correlation $V_P \approx 3.5 + \log_{10} Q$ was used.

In this comparison, two well known correlations between RMR and Q were used (Barton, 2002). It can be noted from Figure 30 that a very rapid improvement in P-wave velocities (and therefore also Q-values) was recorded from about 14 to 24 m depth, with velocity gradient km/s/km of about 200 s^{-1}. (This in itself supports the earlier recommendations to accept longer escalators, and avoid unnecessary metro construction problems).

The writer's independent post-collapse Q-logging using the histogram method (Barton, 2002) showed $Q_{\text{most frequent}}$ of 0.4, $Q_{\text{typical maximum}}$ of 4, and $Q_{\text{typical minimum}}$ about 0.1. $Q_{\text{weighted mean}}$ was 0.7. The influences of clay-filled discontinuities was clearly influencing the low Q-values (recorded in 1997 and now 2007) and also influenced the low RMR values logged on either side of the cavern (Figure 8). The robust arch

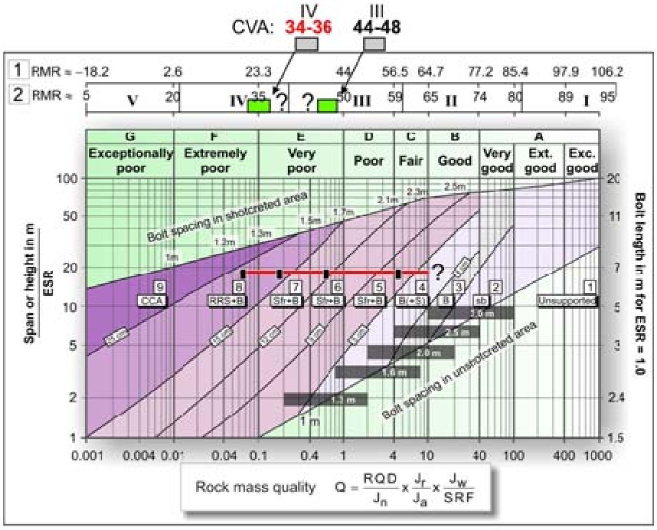


Figure 31. The five drillcores from the holes closest to the cavern that collapsed indicated Q-values (logged by the writer, post collapse) that were consistent with earlier Q-logging by IPT in the late nineties, and also consistent with the cavern face logging where only RMR was used.

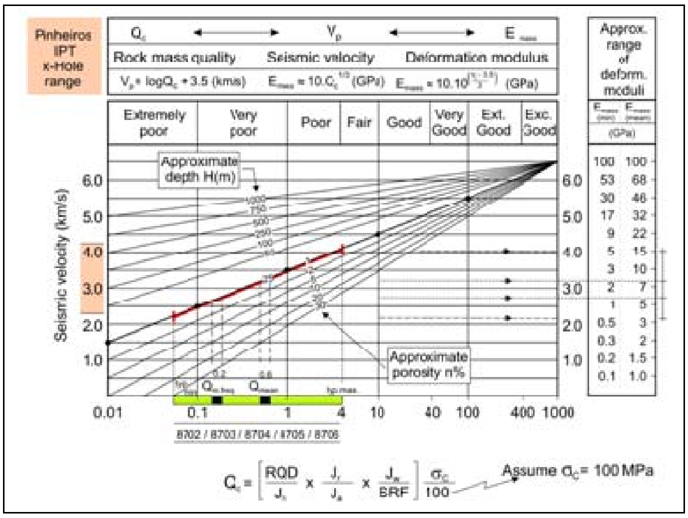


Figure 32. Correspondence of post-collapse Q-logging of the five nearest holes (8702 to 8706) with the cross-hole velocity measurements of IPT, performed in 1997.

support for the assumed low rock cover was a logical choice by the designers, based on the evidence of the five nearest boreholes surrounding the cavern. However, all types of support, including an ultra-conservative Q-system based RRS (Figure 31) would have failed to survive the concentrated loading of at least 15,000 tons from the ridge or wedge of rock lying undetected along the cavern axis.

12. CONCLUSIONS

Unless a limited level of risk is accepted, *the physical impossibility of performing necessary but unreasonable levels of sub-urban site investigation* will prevent the execution of *shallow* city metro projects. Some risk may be inevitable.

Figure 33. Final view of post-collapse excavation to the floor of the station cavern that collapsed. Clearly the rock mass quality at this level in general reflects the higher of the RMR and Q-values logged. However, deep weathering that resulted in clay lenses in the left foreground, may have compromised the way loading from the 'elephant-footings' several meters above, was resisted in this location (see local wall failure in Figure17).

Deeper construction from the underground *in rock*, both of the tunnels *and* station caverns, as practiced of necessity in many cities lacking suitable geology, clearly represents a cheaper and safer solution, and would also result in less settlement damage, and more effective pre-injection when needed.

Rock conditions for tunnelling are invariably more favourable at depth, whereas the 'near-surface' is more unpredictable due to the effects of deep weathering and locally reduced rock quality. This delays both D+B and EPB-TBM tunneling, and obviously may prejudice station cavern design and construction, and also the level of risk of settlement damage.

Longer escalators, in dedicated inclined shafts, leaving station and related commercial facilities at the surface, may save an owner one to two years in construction time, and save money for new metro lines to be developed earlier. This may also be positive for consultants and contractors.

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