

The main causes of the Pinheiros cavern collapse

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Abstract

An extremely large collapse occurred at a metro cavern and station shaft, along the new Line 4 in São Paulo in early 2007. Despite extensive investigation with eleven boreholes close to and even in the centre of the cavern, a high-standing central ridge of less weathered gneiss, with one misleading low point, was missed by all drill holes. Low rock cover was assumed, but the reality was arching compromised by an adverse, wedge-shaped, 10 m high and 15-20,000 tons undiscovered ridge of rock that grossly over-loaded the structural arch and its wide footings.

INTRODUCTION

On Friday 12th January 2007, a dramatic metro construction accident occurred in São Paulo, Brazil. 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. Seven people lost their lives in the collapse.

These station and shaft constructions are close to the Pinheiros River, in the SW sector of the city, and are part of the new Line 4 (Yellow Line) of the presently expanding São Paulo Metro. The Consortium CVA, Consorcio Via Amarela, composed of most of the major contractors in Brazil,



Figure 1. The Pinheiros station cavern and shaft collapse of 12th January 2007.

were awarded the detailed design and construction of Line 4 in 2004. The accident occurred so

rapidly that there was no time for warning to be given.

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, causing an air blast in the running tunnel, actually sucked the seven Rua Capri victims to a lower level in the debris than they would have fallen if materials had been more uniform. Five of the victims were in a small bus; others were pedestrians in Rua Capri, seen to the right-side of Figure 1.

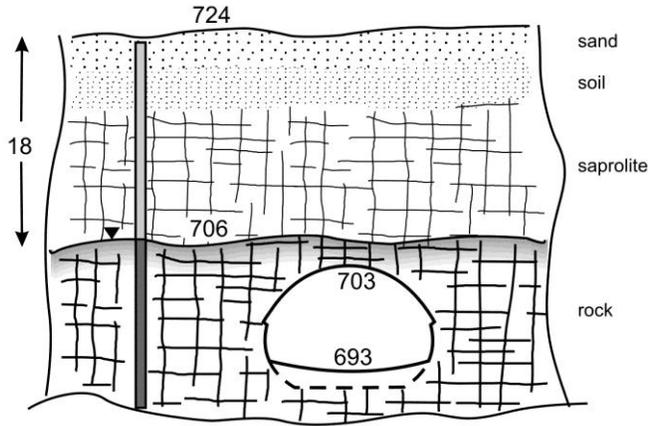
BOREHOLES FOR SITE INVESTIGATION

Prior to final design and construction of the 18 m span station cavern, numerous boreholes had been drilled through the soil, saprolite and weathered Pre-Cambrian gneiss. There were eleven boreholes drilled around the shaft and eastern station cavern. The four boreholes located close to the sides of the cavern, and one almost in the centre of the cavern, had indicated some zones of deeply weathered rock, especially in the biotite gneiss. Foliation was mostly steeply dipping to vertical.

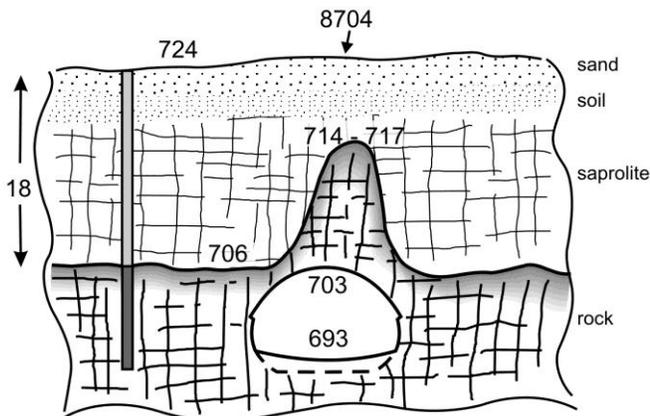
The arch of the Pinheiros station was at a mean elevation of 703 m. Borehole 8704 drilled near the centre of the cavern, had correctly indicated a (local) top-of-rock elevation of 706 m. This was exactly the same as the *mean* rock elevation found in the four other closest holes.

SUB-SURFACE RIDGE OF ROCK WENT UNDETECTED

The tragic contrast between interpretation and subsequent reality, following 1-year of excavation through 30 m of the collapsed soil, saprolite and jointed and foliated gneiss, is indicated in oversimplified diagrammatic form in Figure 2.



a) Expected mean elevations: The closest boreholes were drilled from 723-724 m surface elevations, and rock was reached between elevation 706-707 m in the majority of cases.



b) The extraordinary reality: 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.

Figure 2. a) Sketch of the anticipated top-of-rock elevations based on the five nearest boreholes, including one hole near the centre of the cavern. b) Sketch of the extraordinary reality, in oversimplified form.

Two central ridges of less weathered rock with sloping sides provided the 'geometry' for potential failure. However, final collapse is believed to have been triggered by water pressure and clay softening caused by leakage from a cracked pipe, which crossed a major discontinuity marking the rear of the slide.

ROCK QUALITY LOGGING AND PRE-GROUTING

During construction of the eastern station cavern, geologists had registered an increasing volume of medium quality Class III rock (RMR = 44-48) in the centre of the cavern in the direction of Rua Capri. The Class III 'core' was surrounded by poorer quality Class IV rock (RMR= 34-36) on either side (see A/B/A structure, Figure 4).

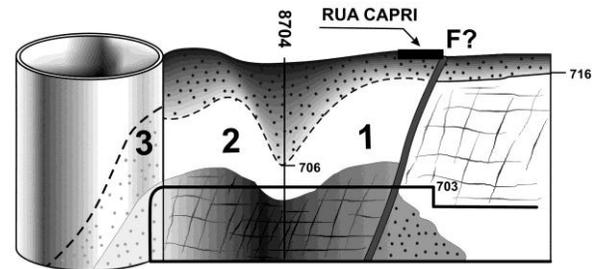


Figure 3. The undetected ridges of rock (1, 2) missed due to the fated location of hole 8704. Seismic refraction was used with limited success and at few locations in this project, due to a combination of 24 hour traffic noise, deep weathering, and inevitable access limitations in this major metropolis of 17 million people and 6 million cars.

That this better quality rock 'core' could be a threat to cavern stability was not of course imagined, until with the benefit of hind-sight following the collapse, the possibility of differential weathering was considered, since a high ridge of rock was now indicated, in contradiction to earlier borehole evidence. Independent Q-logging of the five closest boreholes subsequently showed a range of $Q = 0.1-4$, similar to earlier logging by IPT for São Paulo Metrô, and similar to the contractor's RMR-logging within the cavern.

HEAVY PRIMARY SUPPORT FOR THE STATION ARCH

Normally the process of arching, as with a high quality rock mass, results in the need for the designed support to bear just a small fraction of the overlying load of rock. A conservative primary structural support was used to maintain stability as the cavern arch was excavated. The lattice girders had close spacing (0.85 m c/c) and were embedded in a minimum thickness of 35 cm of steel-fibre-reinforced sprayed concrete. A view of his support near the *western* end of this station cavern is shown in Figure 5a.

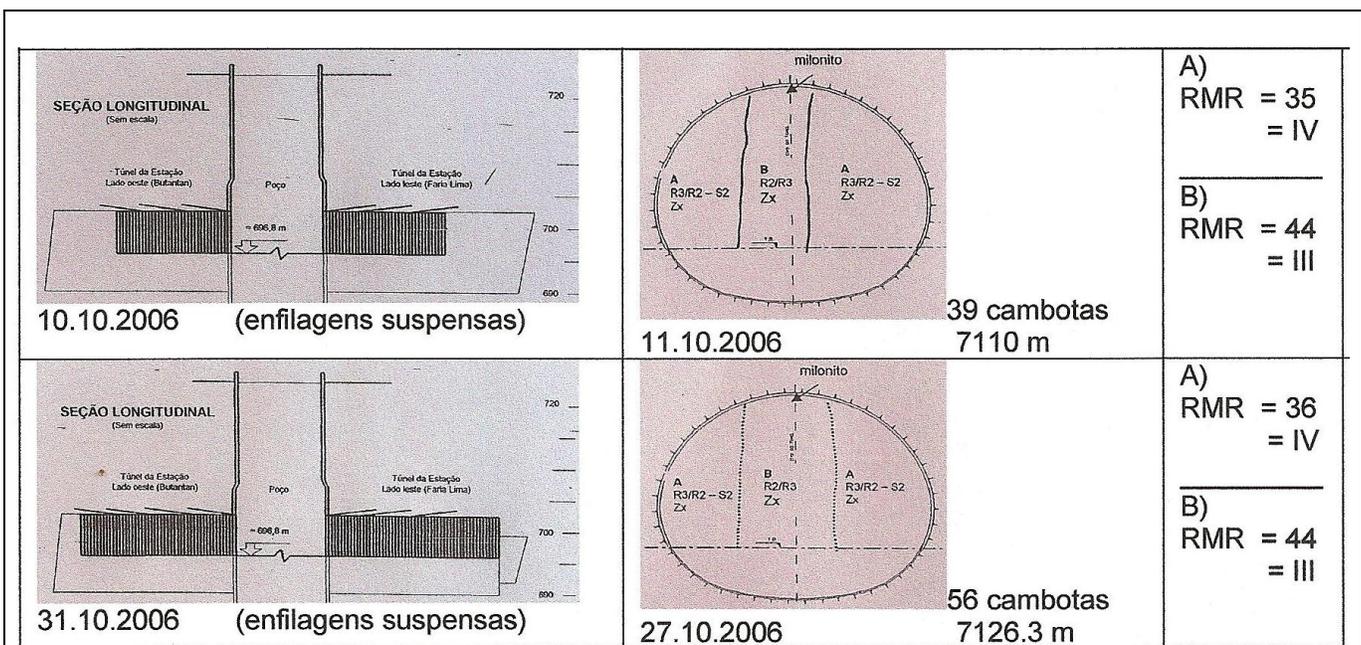


Figure 4. Longitudinal sections showing progressing number of lattice girders at two of the mapped cavern faces. The RMR rock class values of the ‘core’ (B) and the surrounding rock (A) are listed. Pre-injection screens (enfilagens) were suspended after #3 due to improving rock quality and reduced grout take towards the eastern end of the cavern under Rua Capri.

HEAVY PRIMARY SUPPORT FOR THE STATION ARCH

Because of the weaker rock at the sides of the cavern, conservative assumptions were made for the foundation strength and stiffness of the rock beneath the footings of the lattice girders supporting the top heading. The so-called ‘elephant feet’ supporting the structural arch, were placed in large excavated recesses in the rock, at either side of the cavern.

CVA 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 sprayed concrete thickness was 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 in various locations, with UCS expected to be in the range 5 to 10 MPa, sometimes even less than this.

Final support of this large multiple-component station structure was to have consisted of steel-reinforced concrete. However this stage of construction had not been reached at the time of collapse, either in the eastern or western station caverns, or in the central station shaft. A first 4 m high bench to elevation 693 m was completed, prior to accelerated deformation in the last three days before the collapse. This abruptly followed several months of gently increasing, then stable maxima along the cavern, ranging from 14 to 24 mm. There was acceleration in the last days.

It has been assumed by the institution investigating the collapse that the contractor consortium is to blame for alleged design, execution, and management errors. The reality is that the investigating institute has demonstrated limited hands-on tunnelling experience, and has limited rock mechanics experience related to tunnel design and execution in jointed rock.

In an extraordinary oversight, the fact that top-of-rock elevations are far different from the borehole-determined levels has been ignored, despite careful recording of all the elevations of the collapsed rock. The simple addition of 8 to 10 m as the height of fall of the huge ridge-of-rock has not been made in the ‘official’ interpretation, and this means that the main cause of collapse has not been discovered by the investigating institute, since the presence of a high ridge of rock far above expected levels has not been acknowledged.



Figure 5. a) 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. b) View of the cavern (and running tunnel) some days prior to collapse. The last eight lattice girders, beyond a fault in the foreground, did not fail.



Figure 6. a) The increasingly deep open excavation, where a 30 m depth of collapsed material (jointed gneiss, saprolite and soil) was systematically excavated, until reaching the crushed lattice girders at elevation 693-695 m. b) Detail of a portion of the sloping sides of the ridge of jointed rock that had fallen 10 m. These sides and the top were deeply weathered and weak.

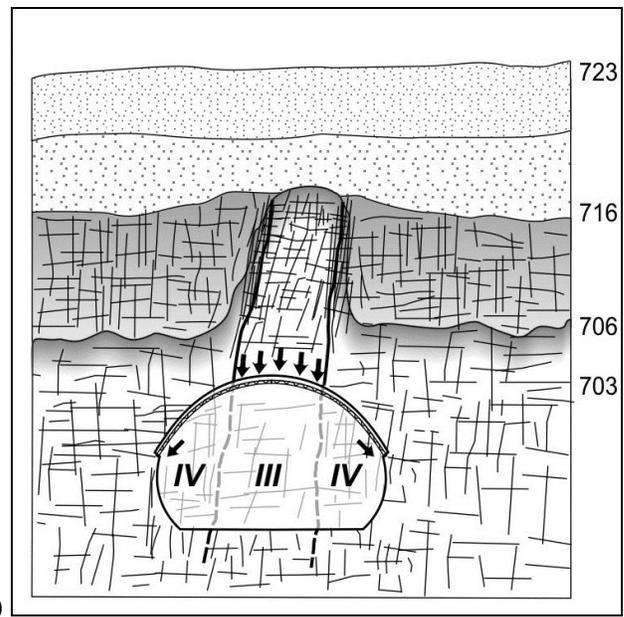
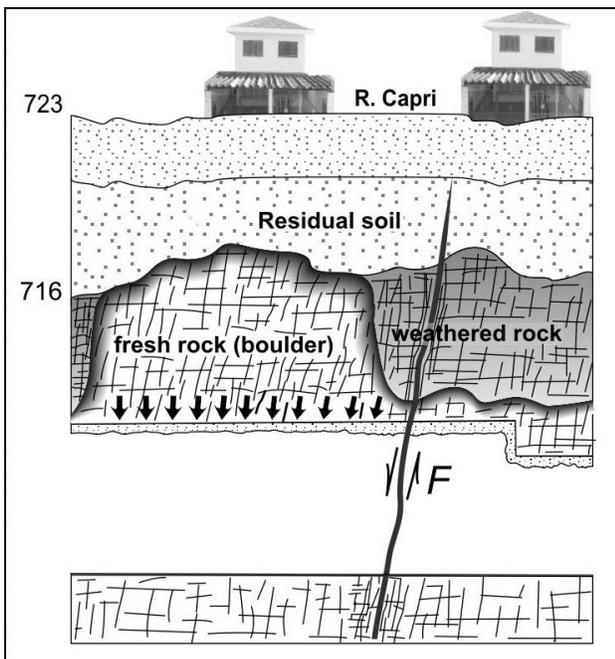


Figure 7. Conceptual model that was developed as a possible explanation of the final stage of differential weathering, leaving a threatening ridge (and wedge) of rock that threatened stability as it prevented efficient arching above the cavern due to clay along its sides. Evidence for such a ridge was gradually exposed, during careful excavation through the fallen rock debris.

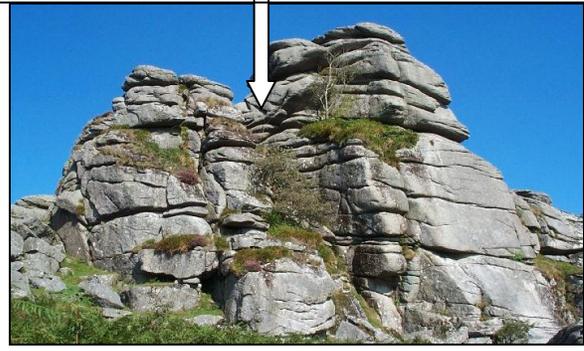
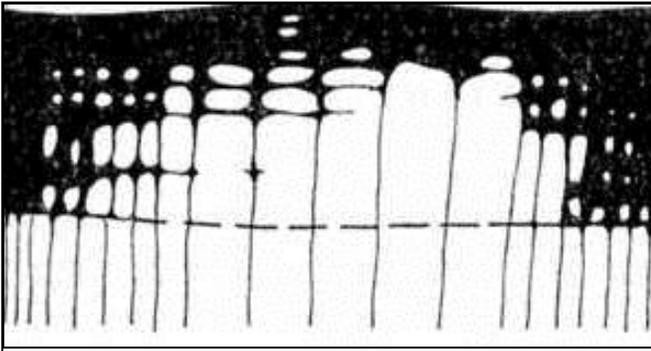


Figure 8. Core-stone phenomena in massive granites. The sketch is from Linton, 1955. ('The problem of tors'). Despite the much *less massive* nature of gneiss as found at Pinheiros, the remnants of more jointed, and differentially weathered structures were clearly evident throughout the stage-by-stage excavation. (Arrow: conceptual SM-8704 location, between two towers (in this case remnant towers or 'tors' of granite from SW England).



Figure 9. Despite falling as much as 10 m this weathered residue of the Class III 'core' still rests at a top elevation of 707 m. This suggests that it was previously at elevation 717, or 11 m above assumed mean top-of-rock levels, based on five nearby boreholes.

POST-COLLAPSE EXCAVATION REVEALS LIKELY COLLAPSE MECHANISMS

During most of 2007 and in the first 3 months of 2008, the fallen rock sketched in Figure 3 was carefully excavated, under the supervision of a government institute IPT, working on behalf of the Police. This post-collapse excavation was performed from the base of an increasingly deep open excavation (Figure 6) supported eventually by hundreds of tie-backs. It will eventually be completed as a cut-and-cover station platform construction.

Differential weathering along the sides of the 10 to 13 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 clay or soil-filled boundaries which hindered arching, and

instead stood ready to supply a huge load onto the lattice girders and steel-fibre reinforced shotcrete support.

Figure 7 shows conceptual drawings of what is believed to have caused the failure of the cavern: a jointed and variously weathered 'ridge-of-rock' structure, that must have had its origin in *differential weathering* between what, at cavern level had been class III rock (the 'core') surrounded by the poorer class IV rock which presumably weathered more easily as the surface was approached.

The unusual opposed dip of foliation on either side of this ridge-of-rock also contributed to the 'definition' of an adverse, wedge-shaped ridge that seriously compromised arching, therefore throwing an impossibly high load onto the heavy support.

THE COLLAPSE MECHANISMS SEEN IN THE COLLAPSED SUPPORT

The collapsed parts of the cavern's structural support were reached by February 2008, at 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 had remained to be excavated below this level, in mostly sound rock.

Evidence for extreme over-loading of the structural support, causing its immediate collapse was eventually exposed near the base of the excavations, which continued through March 2008, more than 14 months after the collapse. In part of the cavern there was evidence of footing failure, meaning fracturing of the rock beneath the 'elephant-footings', followed by folding and



a)



b)

Figure 10. a) Evidence for ‘elephant-footing’ failure and wall-support displacement. Note fractured rock in wall, and refer to Figure 12a. b) Bending of cambota resembling the modelled ‘plastic hinges’ in predictive numerical modelling (see Fig 12).



a)



b)

Figure 11. Evidence of the huge loads involved. a) Crushed excavator, with ‘moulded’ debris caught in the folded limbs. b) One of numerous tensile failures of the lattice girder steel, suggesting rapid failure due to impossibly high loading rates.

inwards displacement of wall shotcrete and mesh. An example is shown in Figure 10a.

There was however, more extensive evidence of extraordinary ‘punch-loading’ of the heavy arch support, with multiply folded layers of structural support, and even of lattice girder steel failed in tension (Figure 11b). This is evidence of extremely unusual, and probably high-velocity loading levels, with ‘plastic hinge’ development.

COMPUTER MODELLING OF COLLAPSE MECHANISMS

The likely mechanisms of failure of the support 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 Dr. Baotang Shen (FRACOD) and by Dr. Stavros Bandis (UDEC). Figure 12a shows cracked foundations beneath the ‘elephant-footings’, when realistic levels of rock strength, fracture toughness, and exceptional rock ridge

loadings of up to 20,000 tons were modelled. There was no cracking in any of the three cases (UCS = 5, 10 or 15 MPa), when load levels were low, as reasonably expected in the design.

When load levels were much higher than obviously designed for (not knowing the presence of the high ridge of rock), and rock strength was low, extensive cracking and 20 mm of vertical deformation were predicted, similar to measured.

Figure 12b shows a final stage of cavern collapse in progress, as the wedge-shaped-ridge of rock begins to fall. UDEC with jointing, was not used in design studies due to the limits imposed by investigation via small-diameter drillcore.

ADVERSE FEATURES CLOSE TO RUA CAPRI

A 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, and with such speed of failure.

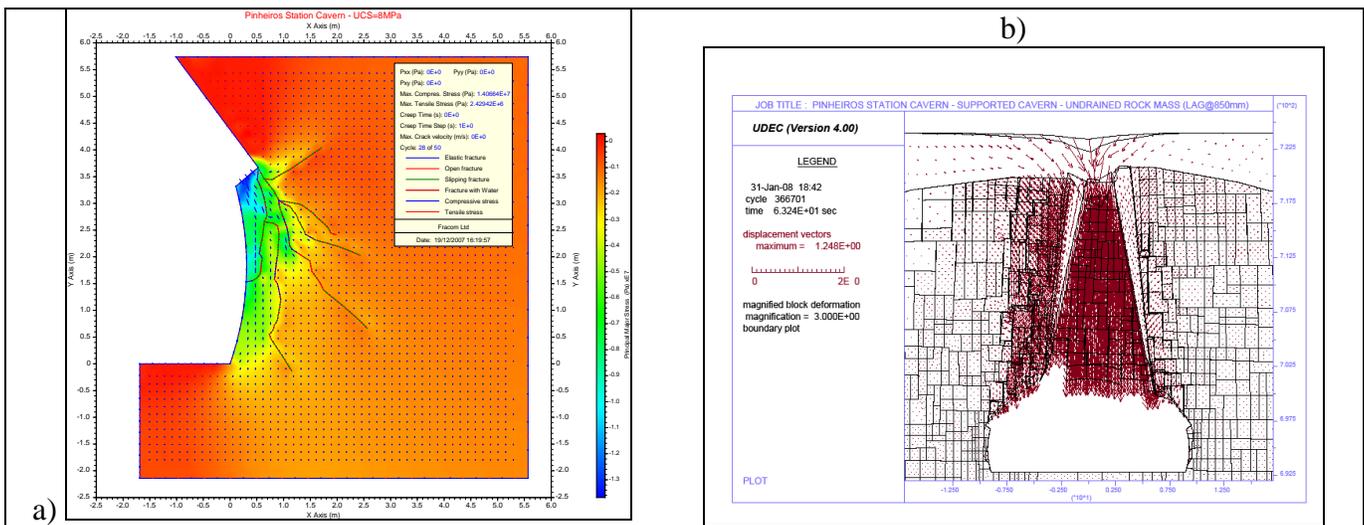


Figure 12. a) FRACOD modelling of rock fracturing caused by over-stressing of the lattice girders in the cavern arch, caused by the unknown elevated ridge of rock. b) Final stage of collapse in a UDEC model of the lattice-girder and S(fr) supported cavern, following softening of overloaded plastic hinges in the lattice girders in the arch. Footing failures involving rock fracturing and interaction with local jointing were also seen.

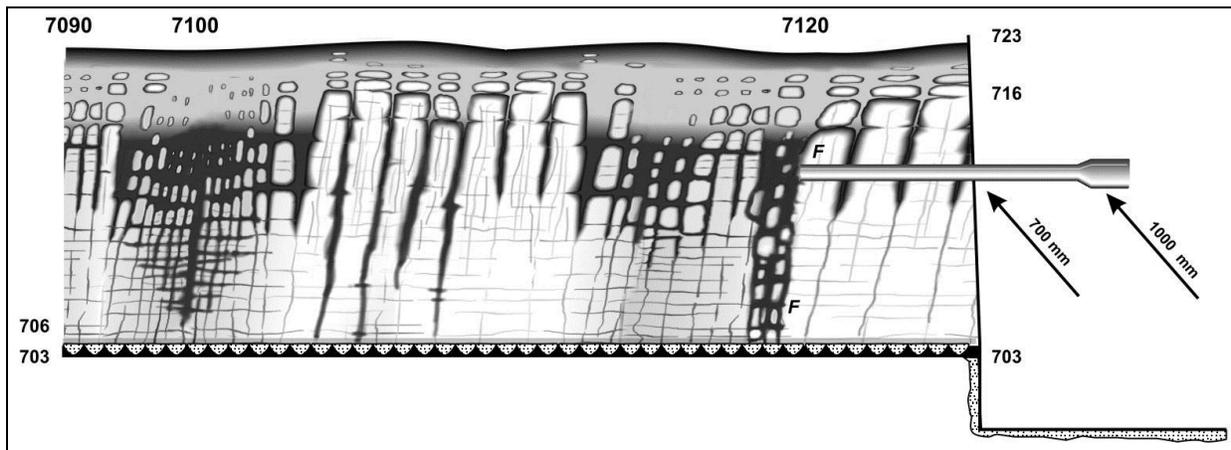


Figure 13. The symbolic fractured pipe, with the change of cross-section and possible raised water pressure occurring in just the wrong location. Rua Capri pavement is at ch 7120 m.

There were by chance three additional adverse features exactly in the wrong place and occurring at the wrong time.

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.

Geological faults or major discontinuities crossing tunnels or caverns occur so frequently that the tunnelling industry developed standard support measures long ago. In the case of Pinheiros, a smooth major discontinuity crossed the cavern at a steep and nearly perpendicular angle. This is most favourable in normal circumstances. At cavern level 20 m below, this

feature did not distinguish itself from the smooth, planar rock joint (fracture) set that consistently crossed the cavern at the same steeply-dipping angle. The standard heavy support was continued to the eastern end of the cavern. In this end of the cavern, beyond the major discontinuity, no collapse occurred.

The unpredictable event that probably *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 sewage and storm water pipe that crossed the same discontinuity exactly beneath Rua Capri. Compounding the situation was the fact that this potential artificial water supply was located immediately following a change of cross section

of the pipe, from 1000 mm to 700 mm. This represents a 50% reduction in flow area, which probably caused an elevated water pressure and unwanted water supply in just the wrong locations.

Naturally there had never before been a cavern under this discontinuity marking the eastern boundary of the collapse. It is surmised that there may have been some down-dip sliding deformation as a result of the approaching and passing cavern arch. This can never be prevented, and is of small millimetre-scale magnitude, but it may have allowed the water from the cracked pipe to flow more easily, transmitting pressure further into the unknown, adverse rock structures.

The artificial water supply, seen flowing from the broken pipe (Figure 14) in a video film taken immediately after the collapse, would have helped to soften clay along the boundary discontinuity (marked FF), and also have had the potential to soften and lubricate the weathered boundaries of local parts of the adverse wedge-shaped ridge of rock running undetected above much of the cavern arch. Reduced effective stress resulting from increased pore pressure is another possibility for accelerating the onset of failure.



Figure 14. Shear deformation on this major discontinuity when the cavern approached and passed below, may already have fractured this 700 mm pipe.

The final block release surface at the other end of the largest rock ridge 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 7100 m. Alternatively there could have been ‘down-stepping’ across the smooth steeply dipping cross-joints that crossed the cavern in numerous locations. The

second smaller rock ridge (Figure 5) 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 the distant 75° to 80° dipping rear discontinuity (FF) under the eastern pavement of Rua Capri. Although nearly 40 m from the shaft, the down-dip component of sliding during the 10 m 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.

Inevitably, when an adjacent circular shaft that relies on circular and radial loading, suddenly loses a large portion of its circumference, due to collapse of the station cavern, there is insufficient stiffness in the primary lining phase to resist the uneven and dynamic load. Failure of part of the shaft is then inevitable.

CONCLUSIONS

1. The 2007 accident at Pinheiros has focussed engineers and planners attention on risk, especially in the case of too shallow, sub-urban tunnelling in São Paulo.
2. The physical impossibility of performing necessary but unreasonable levels of sub-urban site investigation will prevent the execution of shallow city metro projects, unless a limited level of risk is accepted.
3. Elimination of risk would involve socially and commercially unacceptable degrees of disturbance beneath too many roads and buildings.
4. Deeper construction from the underground, as practiced of necessity in many cities lacking suitable geology, could be a future, cheaper, and safer solution for São Paulo, and would also result in less settlement damage.
5. Rock conditions for tunnelling are invariably more favourable at depth, whereas the ‘near-surface’ is more unpredictable due to the effects of deep weathering, unexpected clay zones, and locally reduced rock quality.
6. It is too optimistic to expect ‘almost zero’ risk just because of numerous prior projects in a city, or because of the insight of talented geologists. Conventional site investigation can never expose all adverse structures, as even excessive numbers of boreholes have shown.