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Fallacies in routines of NATM (RSST) shotcrete supported tunneling and promises thereof

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ABSTRACT: A highway tunneling case in jointed weathered gneiss, documented with measured convergences over a 2-year stoppage with half-section primary lining, had a local collapse 20 days after lowering to full section. Detailed independent analyses of all data indicate that the dominant published premisses of so-called NATM optimizations based on convergences are illusory and misleading, diverting attention from "shake" movements and from rapid shotcrete (plus bolt-and-rib) support guarantees against rigid-brittle collapses.

1. INTRODUCTION.

The RAPID SHOTCRETE SUPPORTED TUNNELING METHOD, RSST, publicised as so-called NATM, has had success along countless tunnels and kilometers, fostering fast-growth marketing. We should not abhor the prevalent acronym NATM, were it not for the fact that it shrouds in mystique the fundamentals on which success is based, while allowing the not too occasional traumatic spot-failures of considerable consequence and heated inquests and debates, to seem far from surprising.

The century-old recognition that a convenient and amenable degree of stress release, and accompanying deformations, leads to economy because of generating co-participation of the geomechanical medium, is akin to the theory and practice of design of retaining walls for Active Pressures K'a as reduced from At-Rest Pressures K'o: so also the 70-year old preference for flexible culverts in lieu of rigid ones. Thus, one must decry that there is nothing NEW or SINGULAR in profiting of the internal shear redistributions, to get the geomechanical body to cooperate with the lining in enhancing economy. The key selling point is of "Flexibility"; not only the primary lining flexibility but, much more pointedly, the "expert's" FLEXIBILITY OF DECISION, based on heralded on-site engineering.

Forget the obvious traumas to Bidders and responsible Contracting, and to Contractor's possibility of organizing routines of speedy rounds. The point is that the mystique favours eluding responsibility, between experienced specialized mentor and fulltime on-site lieutenant, when Contractor's speedy action becomes imperative in any pending emergency.

Flexibility, and on-site engineering to be well-founded on self-appraised experience are emphasized as requisites and guarantees for success. And so we witness rhetoric giving priority to belief, to the detriment of knowledge, logic, reflection and revised knowledge abuilding. We have witnessed it proven that whenever there is a vacuum, "fools rush in where angels fear to tread". As a mere sample I quote Evert Hoek (1994) in two recent submissions, pg. 4-16 and 23-24, emphasizing:-

"One of the major problems in designing underground openings is that of estimating the strength and deformation of the in situ rock mass. In the case of the jointed rock masses, an evaluation of these properties presents formidable theoretical and experimental problems." And, humbly assessing the Hoek-and-Brown rock classification criterion, "Our empirical criterion and our estimates of the input parameters were OFFERED AS A TEMPORARY SOLUTION TO AN URGENT PROBLEM...I am alarmed to see the criterion being applied to problems which we did not even dream about when we made those desperate estimates years ago". Such is the nature of ENGINEERING in distinction to SCIENCE: and if we focus with logic on the right problems, PROGRESS in the experience-cycle becomes inevitable.

The importance of emphasizing RSST in lieu of NATM is that by honestly opening the mind to more realistic diagnosis of dominant vs. secondary problems, we open the road to rapid filling-in of the remedies and recipes, and these become transmissible and public. RSST emphasizes the classic recognition of SPEED as defeating insufficiencies in STANDUP TIME, and primary shotcrete (plus bolts etc..as necessary) with its complex rheology of stress-strain/strength/time permitting a) a short period of flexibility to enhance some rock-shotcrete interaction, b) followed by a rapid set into a rigid-brittle high strength crust for STRUCTURAL support.

2. CASE HISTORY ANALYSED, INCLUDING A FAILURE OF SURPRISING POSITIONING AND TIMING. DESCRIPTION.

The case pertains to the investigation of a moderately inconsequential failure in a highway tunnel executed by drill-and-blast conventional techniques with half-section sequential excavation.

The rock belongs to the well-known archean granito-gneiss Brazilian shield, and is moderately fractured, with joints weathered to the point of being smeared with clayey veneers. The question of geologic classifications is set aside as secondary to the intent of the present paper, although it repeats the persistent experience of gross deficiencies in communication from the presumed root, geology, to the faultily defined endpurpose, the tunnel. While driving the half-section, geologic features were surface mapped in separate by three geologists (Designer, Owner and Instrumentation Co.). Convergence measuring pins were set up at roughly 20m spacings, and continually monitored. Other monitoring included piezometers, surface settlement points, and some deep settlement telltales (all questionable in the steeply rolling pasture countryside). Respecting space restrictions, reference is herein limited to the CONVERGENCE MEASUREMENTS, WHICH ARE PUT FORWARD AS BEING THE FUNDAMENTAL BASIS FOR THE ADJUSTMENT OF THE "CHARACTERISTIC CURVE" of the NATM method. The fact, repeatedly blatant, is that nobody could set any quantitative boundaries on ACCEPT/REJECT criteria for the various instrumented measurements; thereby very often the nominal limits set, for sake of appearance, were greatly overshot, without any decision or action. In general nothing unfavourable occurred. We concentrate attention on Stations 142, 143 which suffered ulterior collapse, and on convergences of Chord C (see Fig.1) in that stretch and elsewhere where significant, for comparisons.

During the half-section excavation, at Station 143, action was taken because of the unusually high

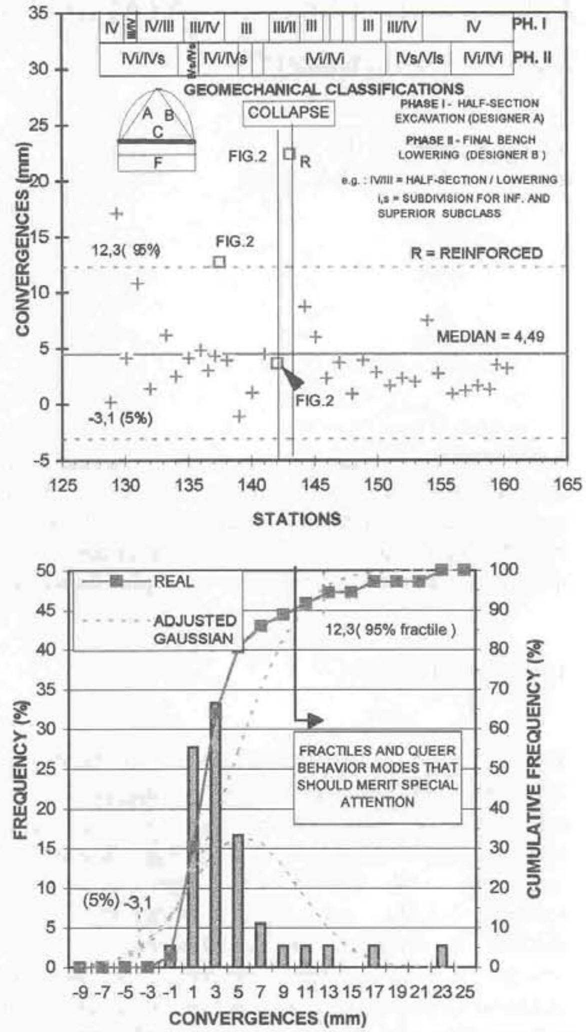

Fig. 1 Convergences registered in Chord C in first 10 days after excavation of half-section

convergence (roughly 3% fractile, cf. Fig.1) observed with accelerating tendency. It was concluded that shotcrete confinement at the foot of the support-arch had to be reinforced: so it was, and within 3 days desired stabilization was recorded.

When the half-section was completed with its 20cm primary shotcrete lining, for extraneous reasons the construction was halted for over 18 months. The project thus profited of an envious opportunity for behavior monitoring. Meanwhile another tunnel had suffered an embarrassing failure, which instigated the Contractor to double his assurance by engaging a second Design Co. (B) to revise every detail, past and subsequent: rock classifications were revised (Fig.1) and several stretches were specifically reinforced with bolts, tendons, and cement grouting. Thereafter the presumed NATM method continued to be applied using the safest of the classifications and criteria superposing Designer B on Designer A and earlier coparticipants (that continued and concurred).

During the roughly 2 years that intervened, Chord C at Station 143 showed a minute tendency to accumulate a total DIVERGENCE of 2,5mm in the first 4 months, followed by undisputable stabilization, all of which was taken as reassuring.

During the half-section excavation there is a delay of a day or two until installation of the convergence points permits the first readings. During Phase II excavation, however, the Chord C measurements permit accompanying all effects. Figs. 2A, B, C present the COMPLETE CONVERGENCE MEASUREMENTS OF THREE CHOSEN SECTIONS, one analogous that did not fail, and the two that ended being involved in a collapse.

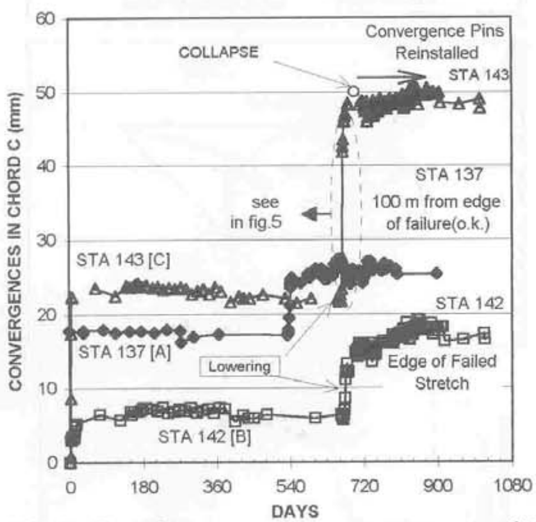

Fig. 2 - Complete convergence measurements of 3 chosen sections.

Proceeding with the description, the fact is that design and monitoring indications were considered satisfactory, and tunneling proceeded routinely. Roughly 20 days after passing the section, and 10 days after completion of the tunnel (excavated from both ends, and terminating about 50 m ahead of the ill-fated stretch of Figs 2B, C) the collapse occurred.

3. ANALYSES CONDUCTED AND STEPWISE PARTIAL CONCLUSIONS.

3.1 Convergence measurements receive great attention, as if they should reflect deformations of the geomechanical body interacting in a structurally analizable manner with the shotcrete crust.

Firstly there is a zero-correction to be made for the halfsection convergences MEASURED. Under postulated "experience" a multiplying factor of 1.75 was considered for transforming MEASURED CONVERGENCES into the desired nominal COMPLETE CONVERGENCES pertaining to the Phase I HALFSECTION EXCAVATION.

In Fig. 3 the two straight lines A, B show how to transform ELASTIC CONVERGENCES for comparison and analyses: starting from the INCOMPLETE REALLY MEASURED x due to Phase I, multiplying on the abscissa by 1.75 to obtain the hypothetical corresponding COMPLETE value Xc , then multiplying by 0.25 to obtain the ordinate y of the presumed COMPLETE ELASTIC CONVERGENCE for the respective Phase II; thereby transplanting back to obtain the coordinates (x , y) on line B. Moreover, the analysis of Fig. 3

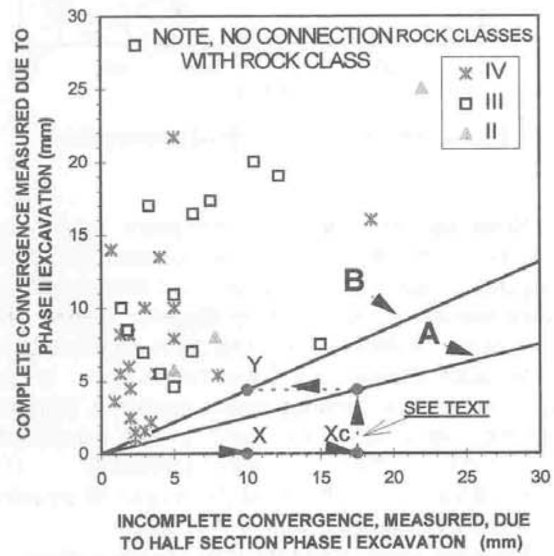

Fig. 3 - Comparison, complete vs. incomplete convergences

starts from the following hypotheses: 1) COMPLETE CONVERGENCE is the sum of BLASTING SHAKE PLUS STRESS RELEASE; 2) SHAKE should not cause DIVERGENCE.

Striking conclusions from this analysis are: by far the greatest convergence was due to rock accomodation due to blasting shake; there was no evidence of relationship with elastic stress release; there was virtually no connection with rock-mass class. The importance of controlled blasting becomes salient, and, ipso facto, the predominance of differentiated behaviours of a disturbed rock collar interacting with the shotcrete crust.

3.2 Dispersions and significance of deformation measurements.

Principal conclusions for on-site engineering decisions are to be derived from three distinct displacement measurements: surface settlements, deep telltale (T) settlements of points within the mass above the tunnels, and convergence measurements.

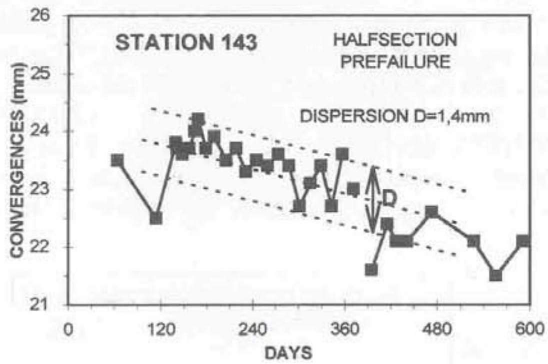

Fig. 4 - Dispersions, trends of convergences

Reverting attention to convergence monitoring (Fig.4) an obvious impression would tend to be that dispersions limited to the order of 2 mm should be quite satisfactory for a 15 m diameter tunnel. The crux of the problem is, however, that decisions have to be taken in hours, or a day or two at most. In Fig. 4 (taken as typical among scores similar) it becomes evident that a behaviour trend is only confirmed, BEYOND THE IRRELEVANCE OF DISPERSIONS, in periods of 21 days to 94 months, because of the very slow rates of trends.

Referring to Fig.5 and the detail of the collapse, if we adapt the dispersion and corresponding period for trend-definition from Fig.4 we should conclude that: a) just before collapse there was a deceiving deceleration to a 5 day period for definition; a) the faster immediate velocity of 0.7mm/day could require a 2 day period for trend-confirmation. Rocks and shotcrete-crust do not merely deform smoothly but by jumps, and even with micro-reversals of mini-trends: that is why since 50 years ago wherever possible preference has been given to monitoring of micro-acoustic generation.

Fig. 6 summarizes additional doubts regarding widely publicised behaviour models to be interpreted via displacement measurements. Note the difference (not unusual) in the long-term trends, and particularly the divergence. The geomechanical models do not include the differentiated behavior possibly forced structurally by the rigid shotcrete lining.

3.3 Useless overemphasis on geologic classifications.

The fundamental need of background information on geology is undisputable. The point, however, is that in moving from the macro to the micro-level of significance of structural behaviours, the overemphasis on geology as single contributor has

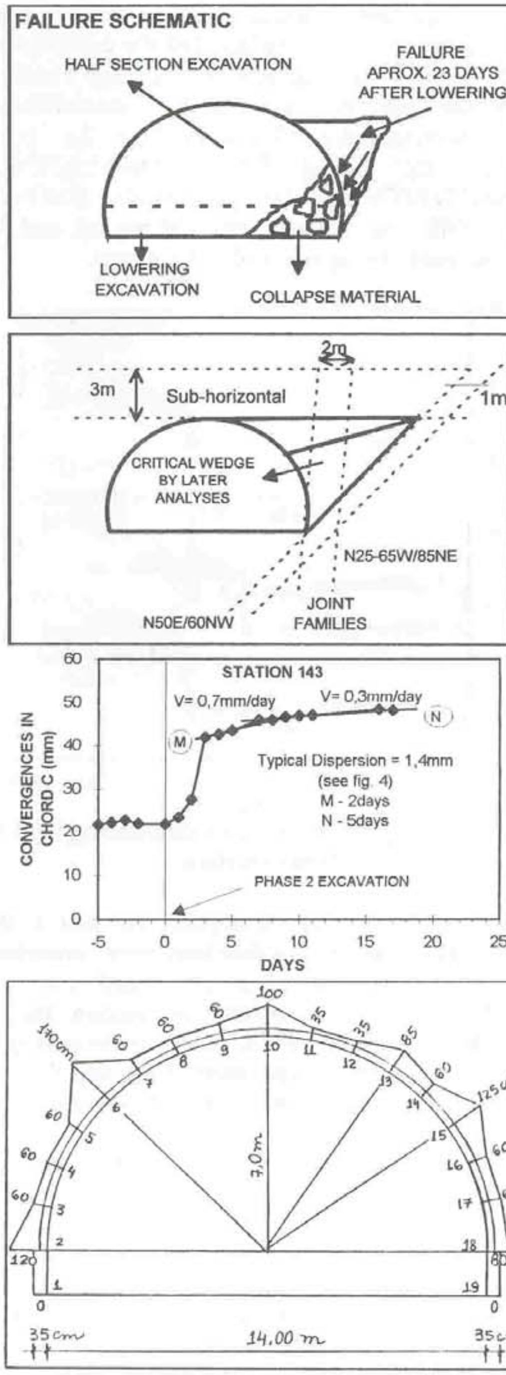

Fig. 5 - Typical overbreak, failure section, and velocities of convergences for decisions.

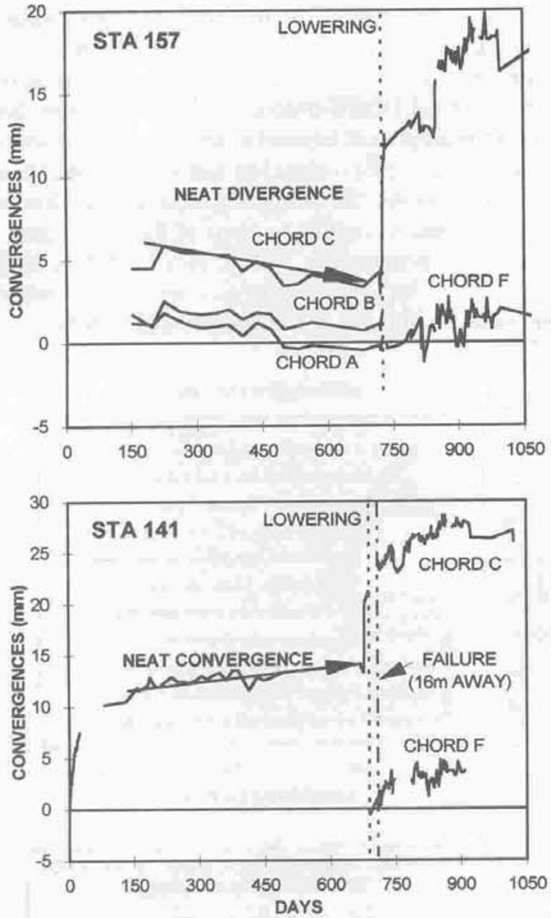

Fig. 6 - Further questions on indications from movements.

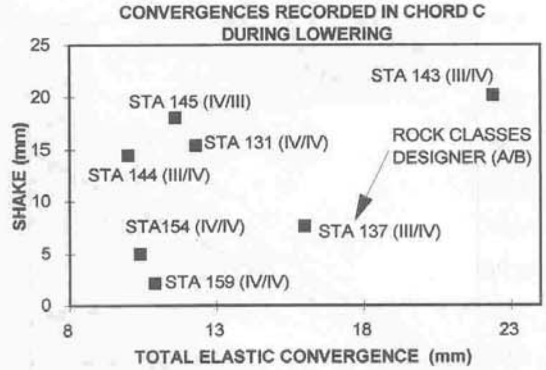

Fig. 7 - Irrelevance of rock classes in practical consequences.

resulted detrimental to pinpointing real needs. Gross subdivisions into ROCK CLASSES based on success/distress in executed tunnels have spread faiths unfounded, shrouding such crucial factors as the convergence/divergence behaviours of arches, bearing capacity of arch-and-rib footings, passive failure of arch base into temporary bench, and so on.

Meanwhile, one cannot deny the umbilical chord still prevalent from Soil Mechanics, because of Terzaghi (1946), whereby different Rock Mass Classes are transformed into EQUIVALENT ROOF SUPPORT PRESSURES. The dominance of SHAKE has been shown in Fig. 1 wherein the shocking irrelevance of ROCK CLASSES to the SHAKE CONVERGENCE becomes patent (Fig. 7).

3.4 Computational models and rock mechanics parameters.

Consistent with the importance self-assumed by geology, the systematic procedure was followed of mapping the principal discontinuities. Thereupon, three computer programs of unassailable respect that have been in current use were duly used.

In Fig. 8 we reproduce the shear strength parameters adopted for two principal joint-families. It must be noted only intrinsically soft materials can limit themselves to such paltry variations from class to class. Small-scale joints can well exhibit a ZERO STRENGTH (open joint, zero normal stress) and, at the other extreme, because of small rock-rock contacts or "bridges", may well behave with a high cohesion and friction. In low stress ranges and under stress- release the effective $ds/d\sigma$ friction can well be of the order of 70° . In a near-surface rock mass with open joints and joints weathered to clay: it is promptly reasoned that normal stress on such joints may be zero over significant areas. Finally, as can be easily seen from geometry (cf. also block theory application, as for instance in Goodman's 1995 Rankine Lecture) it is visionary to achieve arching in material separated by really parallel joints: desired behaviour relies on the stochastic asperities and angular dispersions around the nominal joint dips, plus consequent trapezoidal wedge actions, as Romans had discovered 2500 years ago in developing the masonry arch.

In short, the analytic renditions of schematic[1] "characteristic curves" as well as the conventional computer model results of rock-lining interaction all tend to emanate from frauds on present-day Rock Mechanics.

[1] Has anybody seen real case-history PREDICTION characteristic curves with their successive on-job revisions as direct experience should dictate? Or are such elusive predictions kept in the waistcoat pocket until after tunneling finishes and the time comes for publishing? Can experience be gained without a fixed target for aiming and progressively correcting?

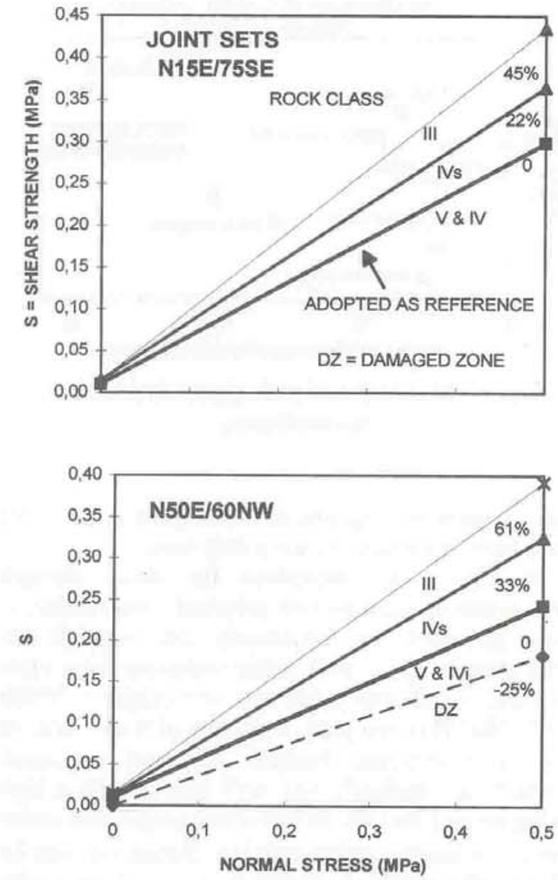

Fig. 8 - Shear strength equations attributed to joints.

Among the many questionings that suggest themselves from the case under discussion, I limit myself to one additional gross fraud on presently usable Rock Mechanics. It is quite frequent, and understandable, that in facing acceptance of magnitudes of convergences, the reasonings are based on the insignificant proportion as compared with the diameter. Indeed, 50 mm for a 15 m tunnel seems insignificant, 0.33%. However, three errors of concept intervene with pointed significance. Firstly, materials testing reports to % strains of the material, and not to % displacements as referred to the cavity.

The conventional errors thereupon start from the fact that geomechanical strains are very low in any case, because the percent face displacement corresponds to an integration of much smaller micro-strains. Conventional Rock (and Soil) Mechanics testing has rarely defined behaviours below 1% strain, leave alone around 1‰.

Finally, regarding equations of mobilized shear strenghs two grave omissions generally prevail: (a) the fact that in prepeak behaviour, at constant strains the cohesion component (in the linear equation) tends to be proportionally much greater than the friction one (de Mello, 1966); (b) the obligatory progressive failure assumption in the rigid-brittle materials.

Fig. 9 is presented to exemplify results extracted from a typical UDEC analysis. It is clearly seen that micro-strains prevail beyond a narrow collar of about 2-4 m (or is it proportional to diameter ?), and these are quite beyond the scope of conventional Mohr-Coulomb peak strength equations of Rock Mechanics Testing. A progressive failure condition is clearly suggested, even in such analyses adopting constant parameters within the entire geomechanical mass.

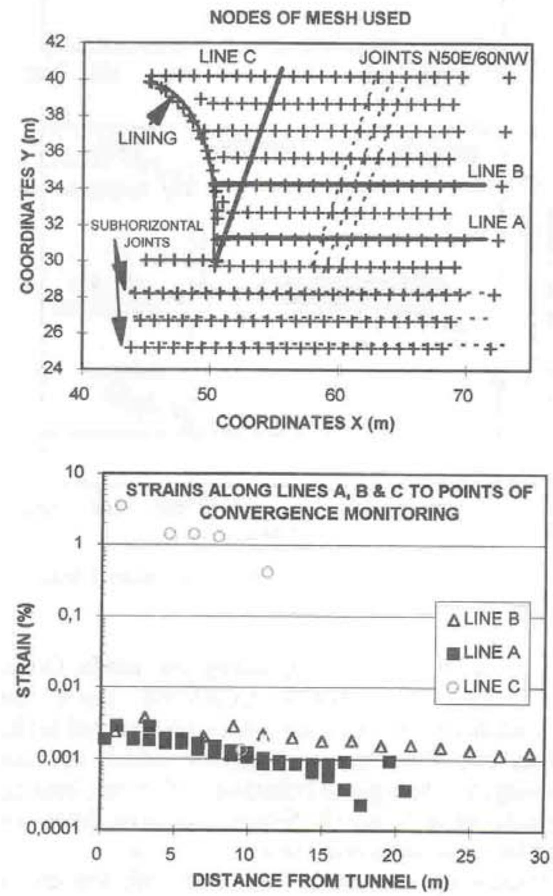

Fig. 9 - Example of results interpreted from a typical current analysis by UDEC.

3.5 Minimal structural considerations.

One of the troubles with would-be professional theorizing on RSST lies in the fact that it is often forgotten that the shotcrete crust should satisfy functions easily tackled in the domain of STRUCTURAL ENGINEERING, and that it is common in Civil Engineering to check on limiting hypotheses.

Assuming in rough approximation that the rockconcrete asperity obliges the primary lining arch to work in unconfined compression.

A finite element program, ALGOR, was used for further analyses, to diagnose effects of less uniform hypotheses, and detect the more critical tendencies: for acceptable simplicity, however, and in the limit, loadings were kept as SOFT, on the isolated shotcrete arch. Several runs were made, details of which do not pertain herein. Results to be emphasized, often unsuspected in the field, are: 1) if overbreaks produce considerably varied cross-sections, the limiting load to failure decreases very much; with the overbreak shown on Fig. 5D the decrease is to 50%; 2) in exactly comparable analyses the drop of loading capacity is very marked if the feet of the arch are permitted to move (analysed a softening elastic medium to simulate a frequent failure condition of lack of vertical and lateral bearing capacity) instead of being immovable (rotation free); as the "footing" modulus of elasticity was lowered to about 10000 kg/cm^2, the arch failed; 3) a concentrated loading applied on an area of 20 x 20 cm (loose rock block) causes punching failure of the shotcrete if it reaches about 16 tons; 4) using ribs composed of rebar cages (total area of steel of 2,4 cm^2) the arch resistance increases only about 5 %, 12 %, 22 % for c. to c. spacings of 1.5 m, 1.0 m and 0.7 m respectively, compared with the pure shotcrete arch, prevailing if spacings are greater than 1.7 m.

3.6 Detailed analyses of the convergence records, referred to theorizations.

Because of the premisse that the tunnel design with progressive adjustments was based, as prescribed in the RSST method, on the convergence monitoring and its cycling through conventional models, it was felt important to submit the records to closer analysis, partially in conjunction with highly reputed computer programs.

The monitored data of Chord C resulting from the delayed stage (LOWERING) of the tunnel offered the basis for the following studies.

Intuitively the deformation was considered subdivided in three phases: (1) initial, spurious, pertaining to "installation effects", herein called SHAKE because of logical association of immediately surrounding rock blocks accomodating because of the blasting vibrations; (2) the second, presumed elastic (rapid, idealized as instantaneous and linear-reversible; (3) the third, elasto-plastic, evolving with time, either asymptotically decreasing, or tending towards slow failure, "plastification".

Fig. 10 presents some typical curves of convergences along 20 days (17 rounds, 80 m, approximately) from the date of the excavation at the respective station. Meanwhile Fig. 11 details the subdivision used for analysis of each curve starting from a few days before the partialized lowering excavation reached the station. For normalization, in subsequent analyses the interval of 5 days was arbitrarily adopted to correspond to the period of "elastic accomodation": attention simultaneously paid to the number of drill-and-blast advances, at increasing distances and times.

The dominance of convergences due to the shake component called for further analyses (procedures

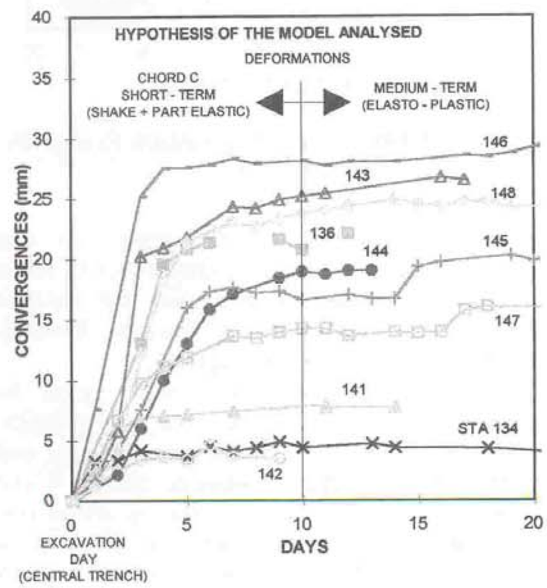

Fig.10 Examples of the behavior caused by lowering.

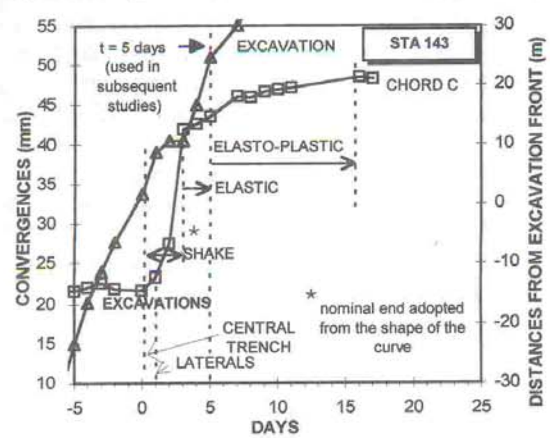

Fig. 11 Detailed subdivision of early convergences.

maintained strictly constant within conventionally "good construction practice"). Firstly the total of three partial bench-lowerings was considered as one, and the distance was assumed to influence by an

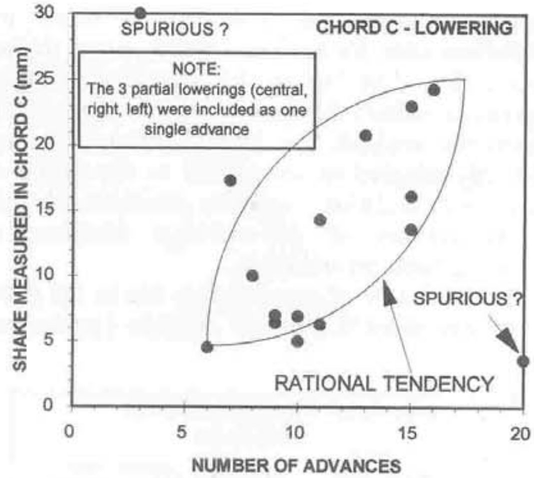

Fig. 12 - Attempt at correlating shake with intensity of advances.

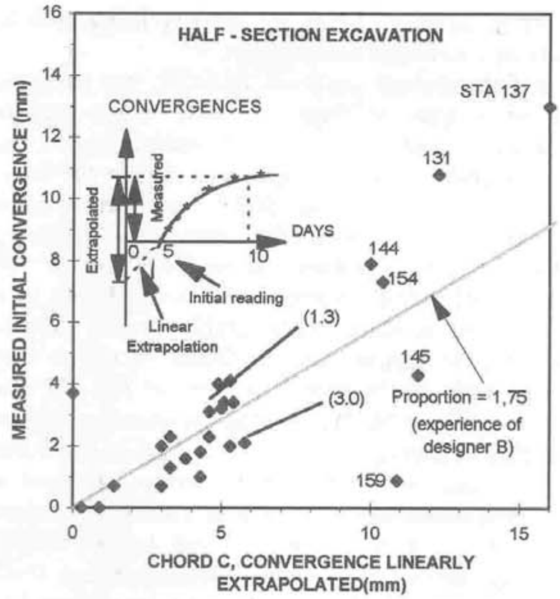

Fig.13 Further analyses of presumed elastic behaviors

inverse square law: the resulting appearance was frustratingly chaotic. In Fig. 12 the number of blasts was taken as 3 per advance, and the distance influence was taken to be linearly inversely proportional: some trend might appear.

A point of apparent incompatibility must be emphasized, possibly calling for more early attention to crown settlements (within the disturbed-ring, and not transmitted to slightly distanced telltales) and convergences of Chords A and B. Having proved (via Chord C and reasoning only) that there are delayed effects, it should stand to reason that at a given station x there should have been some accumulation of effects also from earlier sections x - 1, x - 2, etc. The constancy of Chord C does not necessarily exclude as a first and anticipated reaction the crown accomodation, BY PRESSURE INCREASES and thereupon retardedly by settlements (duly overcoming the relative lining rigidity). The slight DIVERGENCE (besides generalized near-constancy) recorded in Chord C in many stations (cf. Fig. 6A) just before excavation reaches the station may well be compatible with an OMINOUS CROWN COMPRESSION opening the legs of the arch, before side pressures recover dominance.

Setting aside the dominant shake component, it is still of considerable interest to inquire into the presumed theorizable "elastic component". Because of inevitable delayed setting of observation points, the presumed elastic convergences have to be extrapolated backward. In Fig. 13 we examine the Chord C data, and establish that for most cases the "total elastic convergence" might be fitted within the range of 1.3 to 3 times the actually monitored "nominal elastic development".

The search for rationality in convergence measurements (Chord C) as design indices continued. If a geomechanical behaviour was dominant, the total convergence accumulated until before the lowering excavation should bear some relationship to the nominal total elastic convergence due to the lowering itself: a wildly scattered plot resulted. Again, if the shake effect S had some dominant geomechanical component, some relationship should show with respect to the total elastic convergence EC: again, Fig.14 clearly proves that no such relationship existed, since the shake S varied between 1 and 30 times the respective EC value.

Although the vastly dominant convergences followed no model, it is of great interest to examine Fig.15 with respect to comparative "elastic" convergences of Chords C and F at the same dates, after full excavation and primary lining achieved. The very great succession of points of each station (Fig.15A) clearly establishes trends. Although perceptibly different (Fig.15B) it can be interpreted that as the ratio of chords C/F increases, the indication is that the wall lining of the upper half-section was more effectively mobilized.

Further, we analysed the convergence data of bigger values in an adimensionalized manner. Seven stations gave reasonably similar trends. As seen in Fig. 16 the bands including the shake gave a totally different pattern from the bands excluding the shake: the very perceptible attenuation with time only stands out in the curves including shake, and not in the pseudo-theorizable curves excluding shake.

Finally in order to check on the longer-term

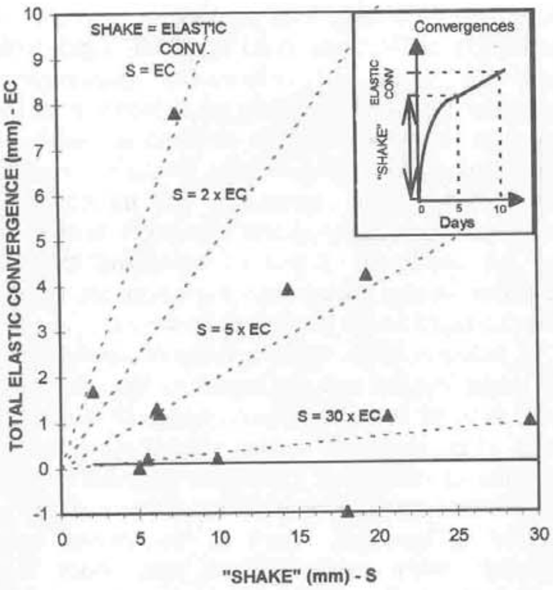

Fig. 14 - Investigating geomechanical relationship between shake and elastic convergence.

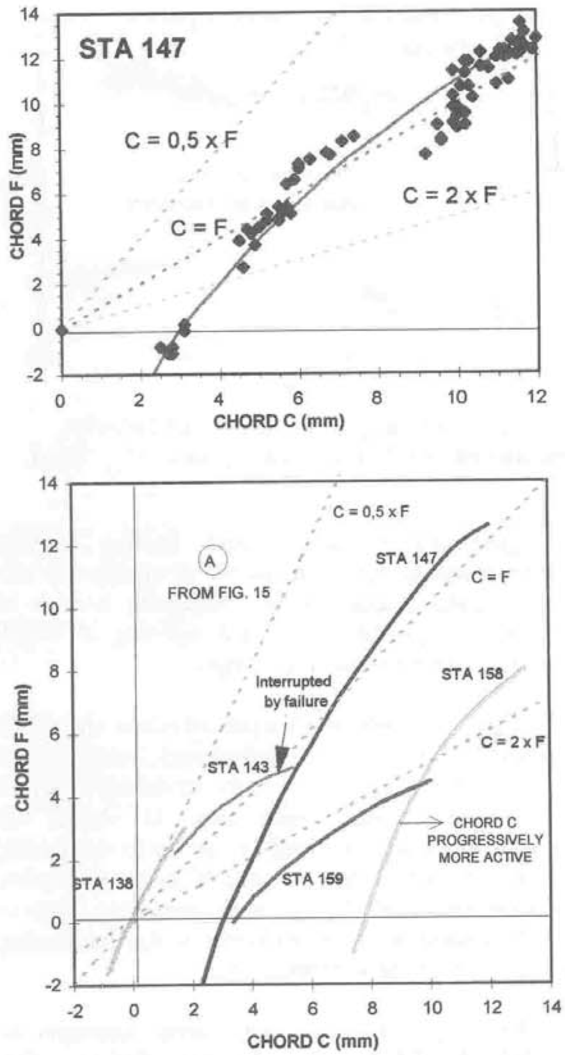

Fig. 15 - Example of comparative behavior of chords C, F (elastic, same dates).

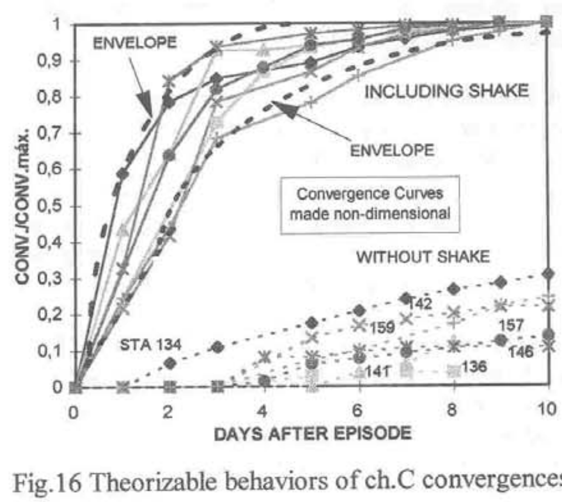

Fig.16 Theorizable behaviors of ch.C convergences

nominally-plasticising behaviors, generally associated with linearity in log-time, the data from stations with bigger measured convergences were plotted as exemplified in Fig.17. Varied and varying behaviors were obtained. The observations are presumably significant, beyong the error band established: what shotcrete-rock interaction models are supposed to be signified ?

There should be considerable room for reconsiderations on the intensely publicised pseudo theorizations that survive principally because in most frequent routines the final lining is very robust on purpose (and rightly so, by sound engineering concepts), and the temporary lining is not allowed to stand alone for such long periods.

4. PRELIMINARY CONCLUSIONS FOR A CONSTRUCTIVE APPROACH, IN FRACTURED WEATHERED ROCK.

4.1 Careful analyses and control of blasting. If blasting and accumulation of blasts provoke the dominant effect: a) it is fundamental to begin by conscientious optimization of a blast pattern with due on-site monitoring of pilot rounds; b) sequential partial excavations might be much less favoured or accepted than appears current. Note that the partial section preference is not only associated with presumed face-stability limilations (less probable in such rocks than in soils) but also often favoured by Contractor's cycle and logistics. Note also that

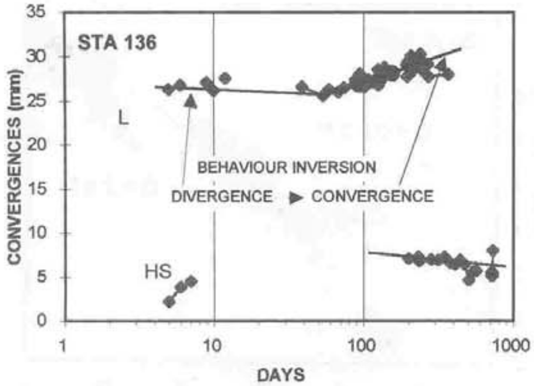

Fig. 17 - Example of convergence behavior interpreted as presumed elasto-plastic (t > 5 days).

damaging blast-vibrations, and limiting particle velocities and accelerations do not fit moderately into most frequent analogous data, principally because of the extreme proximity and the opening at which dynamic tensile stresses reflect back.

4.2 In practice very much more attention should be given to (systematic ?) immediate rock bolting of the disturbed rock ring: so also to the structural action of the shotcrete crust, with ribs. It should be fundamental to develop much more the in situ testing for quality control (via changes of index properties, microseismic geophysics, radar scanning, etc.) than to spend unfairly large proportions of the monitoring budget on deformation monitoring.

4.3 The importance of time, classic concepts of STANDUP-TIME, needs to be emphatically recalled on many counts, but principally two: the tendency to use more reinforcing treatments, insuperably at the expense of time; the tremendous incentive to the development of INDEX TESTS, preferably on-site and at-face, to begin quantifying comparative standup-time experiences.

4.4 Recalling the starting admonitions by Hoek, from the present case one sees that it is terribly important to add three other "formidable problems" which have not been emphasized: a) the difficulty of evaluating the stress-strain-strength properties of the rock mass in medium dimensions under the dynamic conditions caused by blasting vibrations, and the somewhat pointless quest of undisturbed in-situ properties comparison with those of the rock "as blasted".; b) the difficulties of evaluating the static stress-strain-strength behavior of the nearby rock mass after the initial adulterations by the stress-releases and micro-deformations coupled with vibrations; c) the frustration of dealing with progressive behaviors of cumulative microstrains in an essentially rigid-brittle mass for which the deformation measurements transmitted through the rigidly set shotcrete crust are expected to provide the hint of when one might be approaching "the last straw that breaks the camel's back". More direct monitoring via microacoustic emissions (practically almost impossible because of the job conditions) and of increasing stresses, causative factors, rather than displacements, remote effects, should be the avenues to be sought.

In jesting analogy to the benefits of standup-time, we might imagine that the important thing in RSST work is to move fast and loud enough to hurdle the point of no return of making oneself taken for an experienced consultant. Thereupon persistent success becomes inevitable, because one is always engaged to analyse in hindsight...which is the easiest task, especially when confused logic joins hands with extreme dispersions to make predictions so very fluid that moulding observations to wishful thinking is simplest and unperceived.

ACKNOWLEDGEMENTS.

Deepest thanks are due to all entities and persons involved, who rightly followed authoritative publications, and especially to the owner and contractors who suffered the material vicissitude. Since the opinions embodied are strictly personal and recognized as still controversial, the author begs leave to maintain their anonimity until recognitions become unquestionably constructive, as fully merited.

REFERENCES

de Mello, V. F. B. 1966. With regard to in situ shear tests used in Rock Mechanics. *Proc. 3rd. Bras. CSMFE,* Vol.I, VIII: 23-41.

Goodman, R. E. 1995. Thirty-fifth Rankine Lecture: Block theory and its application. *Gèotechnique.* V.45 n.3:381-423.

Hoek, E. 1995. Strength of Rock and Rock Masses. The Challenge of Input Data for Rock Engineering. *The News Journal of ISRM.* V. 2/2.

Terzaghi, K. 1946. Theoretical Soil Mechanics. John Wiley & sons. 3rd. edition.