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PREDICTED BEHAVIOR OF A SUBWAY STATION IN WEATHERED ROCK

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ABSTRACT

São Paulo Subway's Fourth Line will have part of it's underground excavation placed in a weathered gneiss rock mass, fractured, highly altered, and with vertical ubiquitous joints, closely spaced . Pinheiros Underground Station will be excavated in unfavourable geotechnical conditions . Behavior of this excavation was predicted using numerical modelling with distinct elements to analyse rock mass response to excavation . The designer evaluated several options of support excavation, optimizing it in terms of cost and effectiveness . An uncoupled numerical analysis was also made to evaluate water inflow into the excavation . Rock mass discontinuities present three dominant families (one vertical, one sub - vertical and one sub - horizontal) , besides vertically oriented, closely spaced ubiquitous joints . Several options of support systems were analysed . The one with fully grouted rockbolts and shotcrete linings was selected as the most apropriate, based on this analysis . Water inflow inside the excavation was estimated, leading to a fully lined station, in order to reduce inflow to a minimum .

KEYWORDS

Underground Excavation • Weathered Rock • Numerical Modelling • Subway Cavern • Fractured Rock • Water Inflow • Ubiquitous Joints • Grouted Rockbolts • Shotcrete Lining

INTRODUCTION

Underground excavations in weathered rock can show a rather peculiar behavior, in the sense that both continuum and discontinuum modes of deformation and failure can appear simultaneously . Due to this fact, numerical modelling of underground excavations in weathered rock needs special consideration of these features . Distinct element methods (e.g., UDEC - Itasca 1993) show larger capabilities of modelling the behavior of fractured rock masses, dealing with discontinuities, joints, bedding and other structures of the rock mass . Polytechnic School distinct element code (UDEC) was used to perform parametric analysis of this subway cavern, with the purpose of assisting the design of support and lining systems . Previous applications of distinct element methods, at University of São Paulo, have been directed to the analysis of open pit slopes at Brazilian mines (Cella 1993) .

Distinct element modelling of fractured rock masses assumes that the behavior of the system can be simulated by an assemblage of discrete blocks, with boundaries defined by joints or contact planes . It is possible to deal with finite displacements and rotations, with the creation of gaps between blocks, as well as the formation of new contacts . Blocks and joints can be rigid or deformable . The finite difference equations established for the assemblage of blocks are solved using an explicit , forward stepping, numerical procedure . Constitutive equations can be programmed for the joints and blocks . Pre - programmed joint models are available (Mohr Coulomb, Constant Yield and Barton Bandis) . Distinct elements models can be used also to evaluate fracture flow, either through coupled or uncoupled formulations .

PINHEIROS SUBWAY STATION IN WEATHERED ROCK

Pinheiros Subway Station will be excavated as a single tunnel, with a cross section approximately elliptical in shape (with a straigth floor), having a horizontal dimension of 19.40 meters, and a vertical dimension of 10.65 meters . Part of the numerical analysis performed for Pinheiros Subway Station was published previously, and can be found in Kochen , Andrade 1994 .The tunnel will be excavated in a rock mass comprised of weathered gneiss, fractured, and with ubiquitous joints in the vertical direction . Tunnel axis is located approximately 30 meters below ground level, with the groundwater level being coincident with surface level . The rock mass has highly unfavourable deformation and strength parameters, that had to be estimated , in this phase of preliminary design, with emprical correlations, like the ones proposed by Barton to be used with the Q - System (Barton 1995) . Rock mass fractures occur in 3 (three) main directions , being the first family in the vertical direction, the second one in a sub - horizontal direction, and the third one in a sub - vertical direction Typical spacings, inferred from subsurface investigation, are 4.0 , 3.8 and 5.6 meters (for joint sets 1, 2 and 3, respectively) . Rock mass properties were estimated based on inspection and testing of cores obtained from log borings, in situ mapping of joints in nearby excavations, and a tentative classification of rock mass based on available data . Strength parameters (cohesion and friction angle) are referred to effective stress conditions . Main rock mass properties were thus estimated to be the following :

- weathering degree : A4 (very weathered gneissic rock) ;
- Young modulus : E = 15.00 Gpa ;
- Cohesion : c = 0.15 Mpa ;
- Friction Angle : $\phi = 20°$;

Main properties of rock mass discontinuities were estimated as :

- weathering degree : A4 (very weathered , closed joints, no infilling) ;
- normal stiffness : K_n = 6,000.00 MPa / m^2 / m ;
- tangential stiffness : K_t = 3,000.00 MPa / m^2 / m ;
- Cohesion : c = 0.00 Mpa ;
- Friction Angle : $\phi = 30°$;

NUMERICAL MODELING WITH DISTINCT ELEMENTS

Four excavation phases were simulated to evaluate behavior of Pinheiros Subway Station in this highly weathered rock mass, as described below :

- First phase : in situ stress state, before excavation ;
- Second phase : initial excavation and stress release due to three - dimensional effects at excavation

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face, before rockbolt installation;

- Third phase : rockbolt installation behind excavation face (spaced in a 1.5 × 1.5 meters grid, pre - tension load of 100 kN, with a length of 4.5 meters) ;
- Fourth phase : shotcrete lining installation (150 mm thick), at excavation roof and springline ;

Results obtained with distinct element analysis were evaluated, and the some of them were plotted for the rock mass, support system (rockbolts) and shotcrete lining : rock mass displacements and principal stresses, plastified regions (inside rock ubiquitous joints), axial forces on rockbolts (second and third phases), axial forces and bending moments on lining (third phase) .

Distinct element modelling is very sensitive to geomechanical rock mass parameters, like joint normal and shear stiffness, interface (between rock mass and lining) stiffness, strength and dilatancy of interfaces . Due to the fact that these studies were performed for a preliminary design, no sensitivity analysis was made to account for the effect of variability of these parameters . As a general trend, model results showed that displacements in the rock mass were expected to reach relatively high values (due to the rock mass highly weathered condition), principal stresses were increased in the springline (due to the low K_o - coefficient of horizontal to vertical initial stress - assumed for the rock mass) , plastified regions appeared mainly in the springline (due to the ubiquitous joints in the vertical direction, with low shear strength), maximum axial forces on rockbolts (third phase) reached low values, normal forces and bending moments on the shotcrete lining also reached low values . Rockbolts were simulated considering “fully grouted” and “end anchored” conditions, corresponding to shear stress along full length of bolt and anchoring occurring exclusively at bolt end, respectively . No significant difference of excavation behavior was found between the two anchoring systems for rockbolts . The same conclusion was reached for pre - tensioned and untensioned rockbolts (no significant difference of behavior, as pre - tension axial forces on rockbolts - usually in the range of 100 to 200 KN - are insuficient to affect rock mass displacements and it's behavior) . For this reason, only fully grouted rockbolts (untensioned) results will be shown here .

ANALYSIS RESULTS AND PRELIMINARY DESIGN OF SUPPORT SYSTEM

Figure 1 presents one of the construction stages analysed (rock mass blocks formed by main discontinuities families are shown without ubiquitous joints) , with cross section fully excavated, and rockbolts already installed in the roof and springline .

Figure 2 shows rock mass displacements in this same construction stage (rockbolt installation) . Figure 3 shows plastified regions in the rock mass, and Figure 4 shows rockbolts axial forces . Maximum rock mass displacements reach 16 mm on the rockbolt installation stage, due to the fact that rock matrix is highly weathered (A4 degree) , presenting low stiffness properties . Displacement pattern is asymmetric, due to the fact that discontinuity pattern is also asymmetric, and displacements are due to occur mainly though a mechanism involving slippage through joints .

Plastified regions are concentrated mainly in the springline region, being due to the ubiquitous joints occurring in the vertical direction, presenting low strength parameters . Rockbolts axial forces, in the equilibrium phase of the model, present low magnitude, reaching a maximum value of 87.3 kN/m .

Figures 5 and 6 show axial forces and bending moments in the lining, respectively . Primary lining was designed as a shotcrete layer (150 mm thick) , installed in the roof and walls . The floor, initially, was designed unlined , as from the structural point of view it was unnecessary to line this part of the

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excavation . Shotcrete lining was designed with reinforcement by double steel meshes, in order to achieve adequate structural capacity, and to avoid cracking by shotcrete hydraulic shrinkage . First mesh was placed at the outer side of the lining (close to rock mass interface, with an adequate concrete cover to ensure adherence and avoid corrosion), and the second one was placed at the inner side of the lining, with an adequate cover . Steel fiber reinforced shotcrete was not considered at the time of this preliminary design (1994), as steel fibers were not available in Brazil until 1996 . Secondary lining was not analysed within the scope of this research .

Figure 5 shows axial forces in the shotcrete lining, with an asymmetric diagram due to the asymmetric rock mass fracture pattern . Axial forces in the lining are low, reaching a maximum of 23.7 kN./m . Figure 6 shows bending moments in the shotcrete lining, also presenting an asymmetric pattern, due to the same reason (asymmetric fracture pattern) . It is worthwhile noting that bending moments are almost null, except in s0me regions of the excavation boundary, where there is a tendency of occurring shear displacements between rock blocks . Even so, the maximum value of the shotcrete lining bending moment is reduced (9.15 kN.m/m) .

INFILTRATION RATES AS PREDICTED BY NUMERICAL MODELLING

The first version of the lining design system for Pinheiros Subway Station considered leaving the floor without a shotcrete lining, using instead a drainage system to collect water inflow, direct it to a storage reservoir, and pumping it outside of the excavation . The same distinct element model, described before, was used to predict water inflow through station floor . Although it is possible to compute flow rates in a fractured rock mass using a coupled stress - flow distinct element analysis, it was decided , for the sake of simplicity (and due to the fact that it was a preliminary design) , to use an uncoupled model (fixing rock joints hydraulic aperture) .

Figure 7 shows joint pattern in rock mass, enhanced to allow good visualization of possible flow paths through rock mass . Due to the fact that an impervious shotcrete lining was installed around excavation boundary, except at the floor, the only possible way of water flowing inside the excavation was through floor joints . For comparison purposes, the model computed flow rates assuming that joints were persistent through the rock mass, with a fixed hydraulic aperture of 1 (one) millimeter . As the flow rate is proportional to the cubic power of hydraulic aperture, having the infiltration rate for an aperture of 1 mm would allow the designer to estimate the flow rate for any other aperture (assuming an uncoupled model, with no interaction between water pressure and joint aperture) . The real hydraulic aperture will be known just after field investigations (to be performed in the detailed design) . Assuming the uncoupled analysis is a valid hypothesis (for the preliminary design stage), the infiltration rate can be evaluated for any value of the hydraulic aperture .

Figure 8 shows flow rate computed for the 11 (eleven) rock joints that crosses tunnel floor . The total flow rate, computed by distinct elements, reached a value of 8.4×10^{-5} m^3 / s / m, which is roughly equivalent to 300 liters / hour / meter of tunnel (in the longitudinal direction) . This fairly high flow rate would have required the designer to provide the subway station with a high capacity pumping system . Reducing the hydraulic aperture in the rock mass for 0.5 mm would reduce the inflow rate to 38 liters/ hour / meter, which is still a high value compared with the standard limit value in Europe (between 1 and 2 liters / hour / meter of tunnel) . Excavation blasting can also result in considerable damage to the rock mass, increasing it's hydraulic conductivity significantly . For this reason, another design alternative (lining the floor , to impermeabilize it and reduce infiltration rates to a minimum compatible with standard requirements), was also considered in the design .

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SUMMARY AND CONCLUSIONS

Numerical modelling of urban tunnels has been traditionally made using continuum methods, like finite elements or finite differences . This approach is valid for tunnels in soft ground, and can be valid sometimes for unweathered rock masses, with a low density of fractures, and closed discontinuities (that remain closed after deformation , without developing shearing strains) . For the geological conditions (revealed by subsurface exploration) at Pinheiros Subway Station , the continuum approach was clearly not valid, and it was necessary to consider a discontinuum approach (using distinct elements), in order to evaluate with accuracy rock mass displacements, rockbolts axial forces, and lining stresses . The overall behavior of the excavation could not be predicted either using a continuum model . Main conclusions of the studies performed to assist support and lining design may be summarized as follows :

- rockbolt support in this excavation is necessary for stability reasons, avoiding opening of rock joints in the roof and the formation of failure mechanisms ;
- shotcrete lining is most important at springline, where plastified regions appear, due to the presence of ubiquitous vertical joints, closely spaced, in the rock mass ;
- rockbolts must be installed right behind excavation face, to avoid joint opening and degradation of geomechanical properties of rock mass ;
- rockbolts may have to be used with a larger length, if excavation shows that rock mass joints have a larger persistence than assumed in the preliminary design ;
- rock joints located in the roof and floor regions have a tendency to open following excavation ;
- excavation behavior (regarding rock mass displacements, principal stresses and plastified regions) do not vary significantly with bolt characteristics (pre - tensioned or untensioned , fully grouted or end anchored) ;
- without lining , infiltration rates through excavation floor reach high values, favored by the fact that this region shows joint opening due to stress release . This situation would require grouting of the rock mass joints (to avoid excessive water inflow into the subway station), and a high capacity drainage and pumping system ;
- with lining , infiltration rates through excavation floor will be reduced, being due mainly to leakage in the shotcrete layer (through cracks or voids) ;

At Pinheiros Subway Station, numerical modelling using distinct elements allowed the designer to evaluate excavation behavior, as well as defining with higher accuracy the dimensioning of support systems, like rockbolts and the shotcrete lining . This purpose was achieved for a large cross section excavation, in an unusually highly weathered rock mass, highly fractured, and with ubiquitous joints closely spaced in the vertical direction . The model allowed, also, the estimation of infiltration rates through the excavation floor, and the adaptation of design regarding excavation impermeability requirements .

ACKNOWLEDGMENTS

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FIGURES

Paper 160, Figure 1.

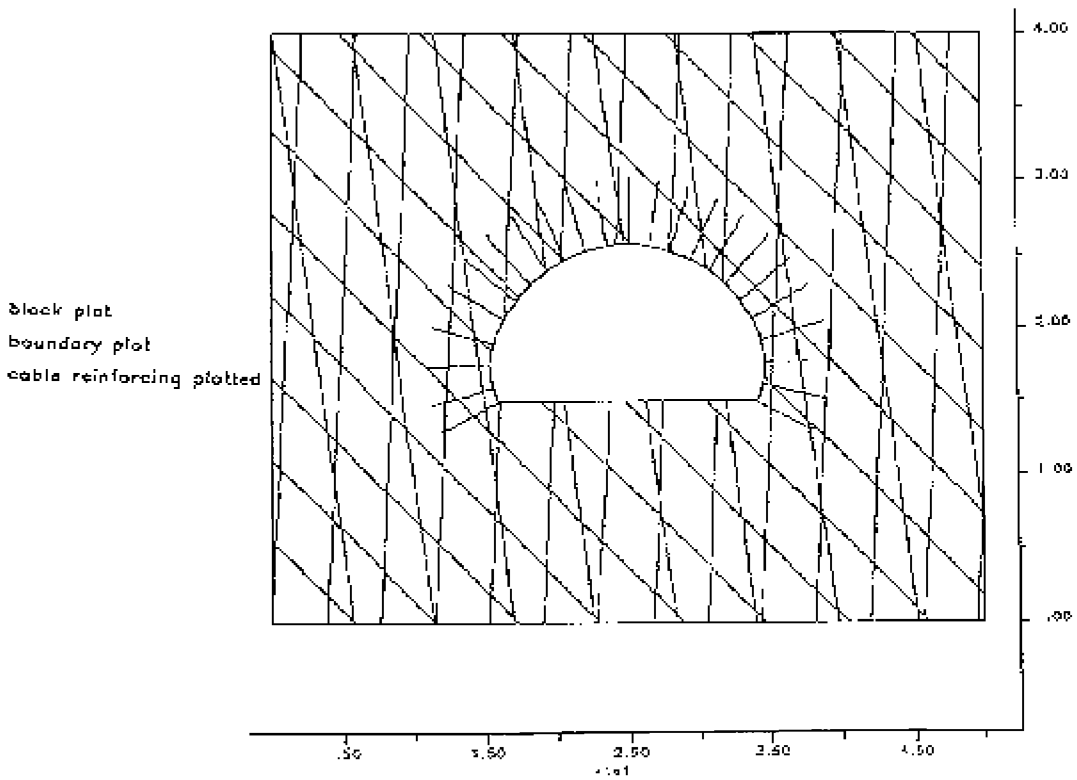

Figure 1. Distinct Element Model for Pinheiros Subway Station Analysis

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Paper 160, Figure 2.

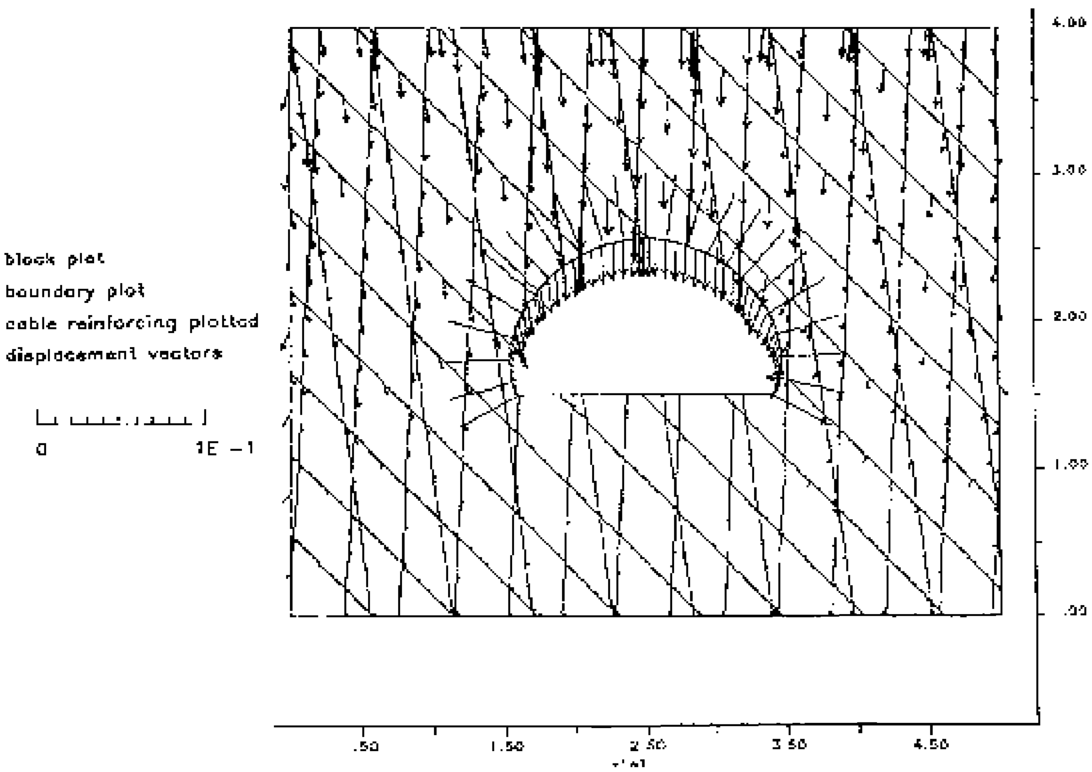

Figure 2. Rock Mass Displacements After Rockbolt Support Installation

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Paper 160, Figure 3.

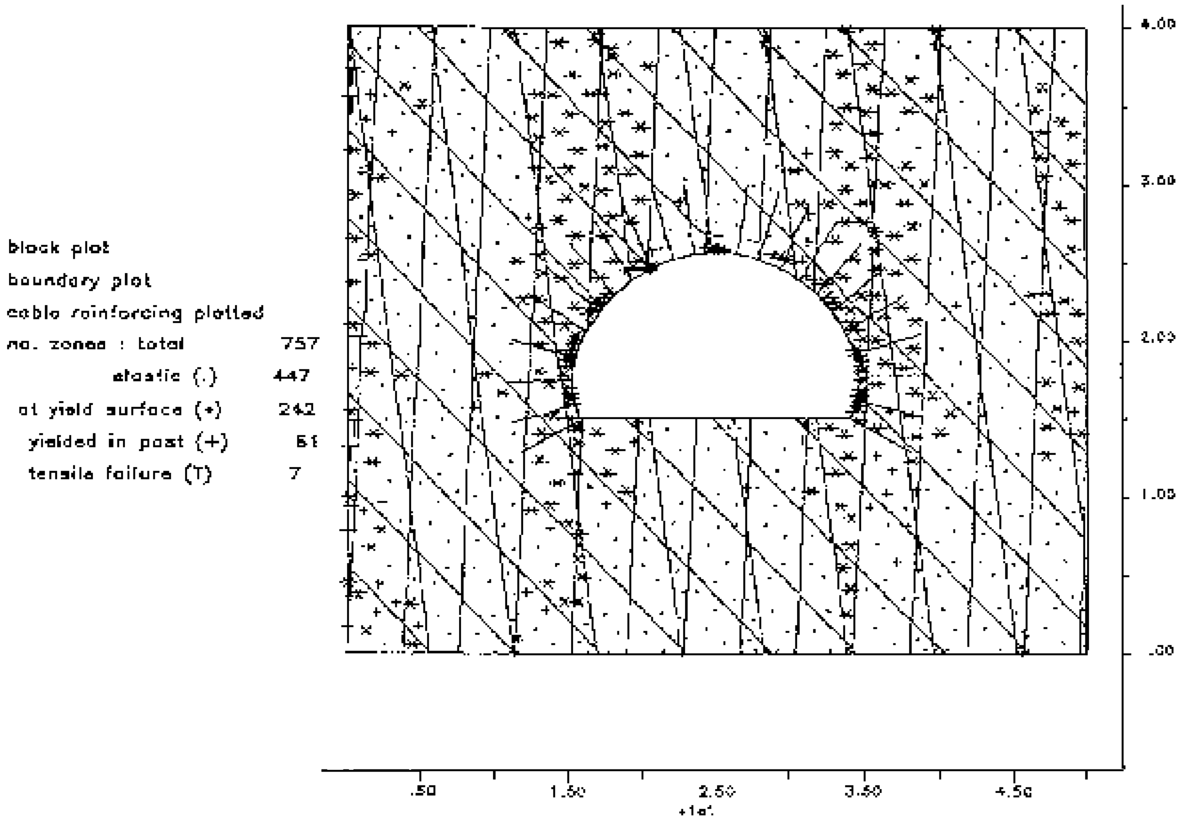

Figure 3. Rock Mass Plastified Regions (After Rockbolt Support Installation)

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Paper 160, Figure 4.

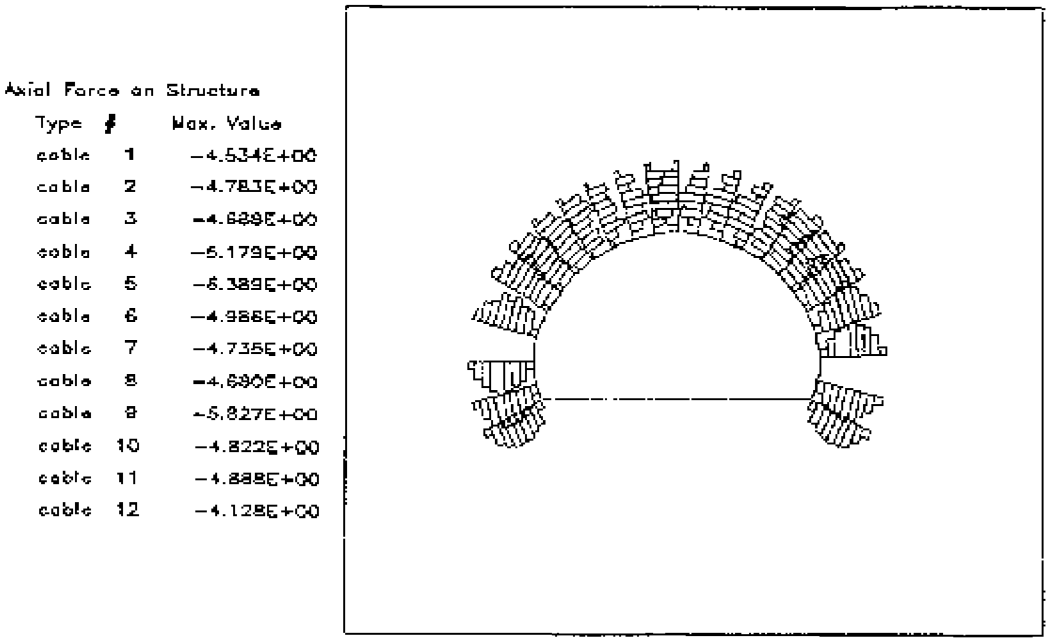

Figure 4. Rockbolts Axial Forces Pattern After Installation (Maximum Value = 6.39 MN/m)

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Paper 160, Figure 5.

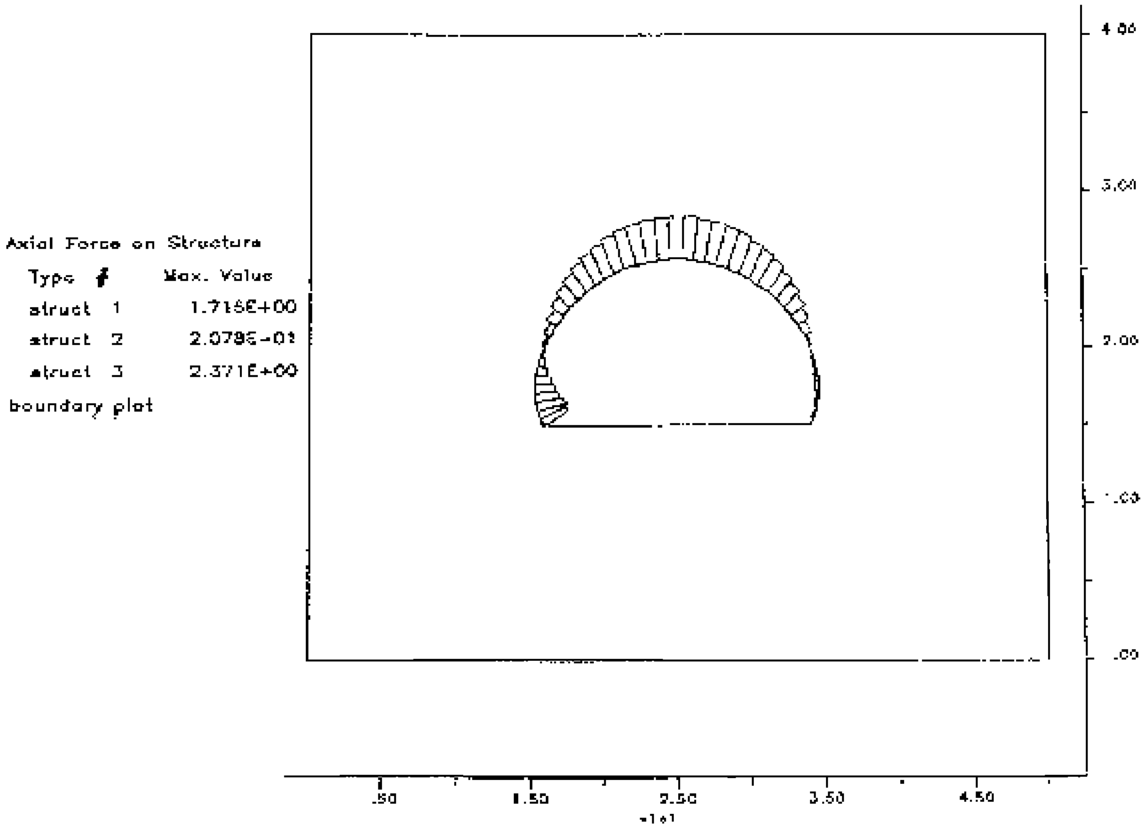

Figure 5. Axial Forces on Shotcrete Lining (Final Equilibrium Condition)

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Paper 160, Figure 6.

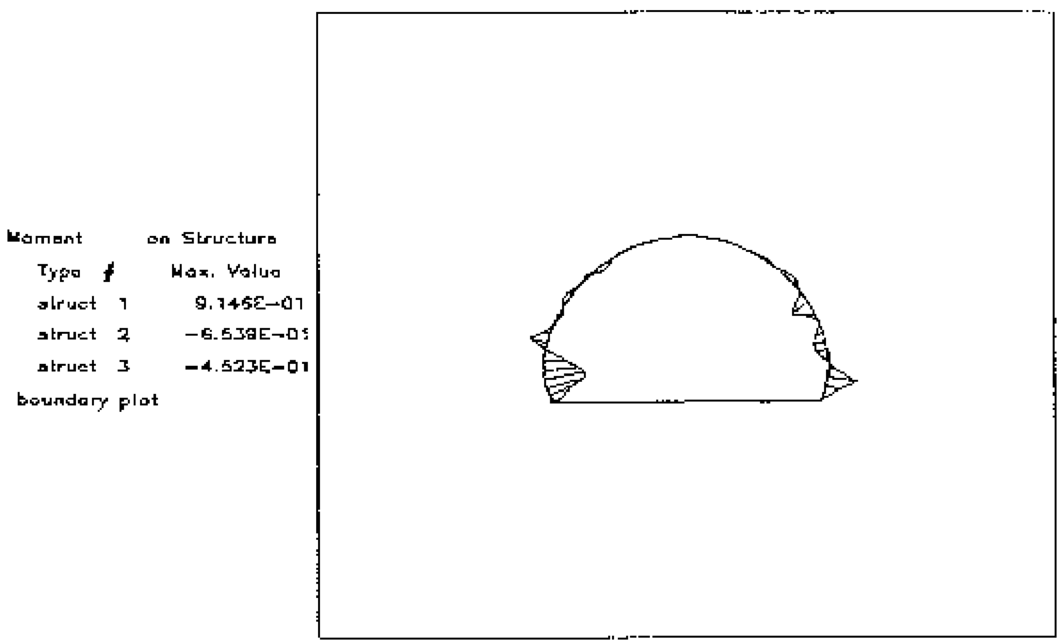

Figure 6. Bending Moments on Shotcrete Lining (Final Equilibrium Condition)

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Paper 160, Figure 7.

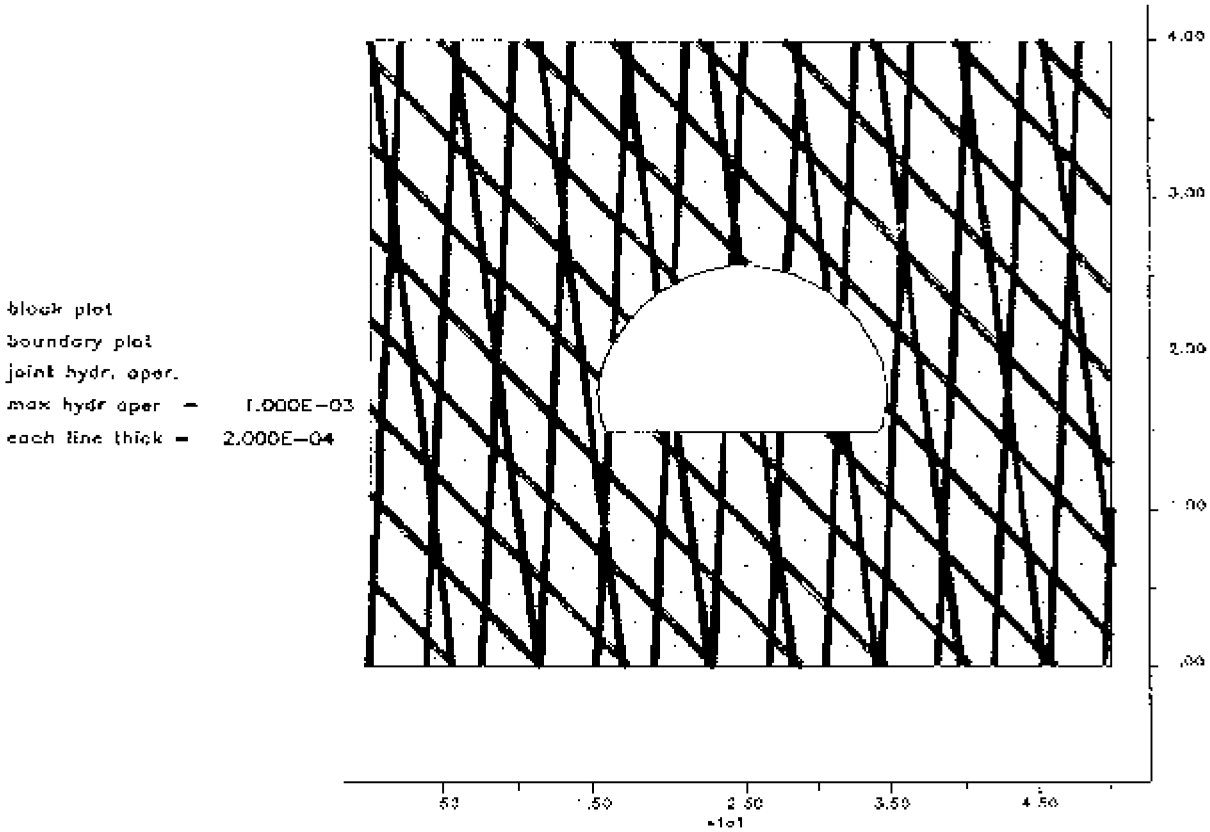

Figure 7. Possible Hydraulic Flow Paths Through Rock Mass

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Paper 160, Figure 8.

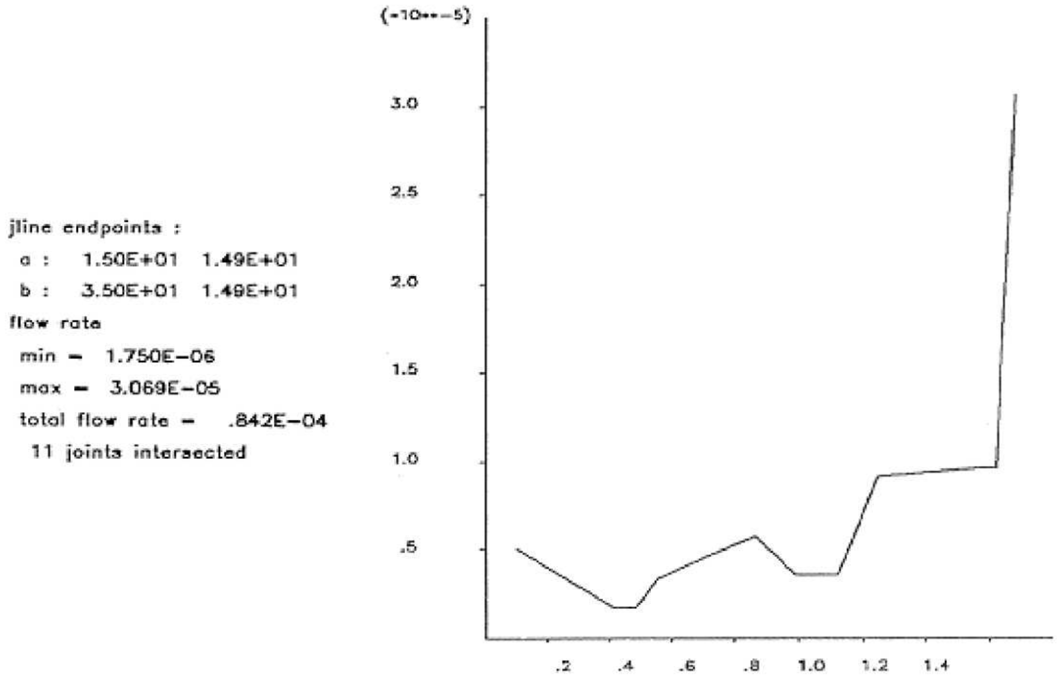

Figure 8. Total Flow Rate Through Excavation Floor (No Invert Lining , Hydraulic Aperture of 1 mm)

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