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General Report for Theme Seven – Case Histories in Rock Mechanics

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The subject of this session, as designated in the Final Program is "Case Histories in Rock Mechanics". Because there are many aspects to the engineering of structures in and on rock formations other than rock mechanics, the general reporter will hereafter refer to this session and subject as "Case Histories in Rock Engineering".

The successful practice of Rock Engineering involves a working knowledge of at least four broad areas. These areas are:

(1) Engineering Geology
(2) Rock Mechanics
(3) A Knowledge of Precedent
(4) A Knowledge of Construction Procedures

The "Rock Engineering" of a particular structure in rock usually involves to varying degrees the application of a working knowledge in all of the areas listed above. The various areas of activity involved on a rock project can be summarized under the broad areas of Design, Construction, and Performance as shown in Table I.

As indicated in Table I rock mechanics is important, but is only one of the important activities in the total rock engineering of a project. Engineering geology is extremely important in defining the critical features of the geology which may affect the performance of a structure. A coherent picture of the geology is necessary for laying out detailed exploration by means of borings or adits. A knowledge of the geology is also important in defining the geometrical relationship of the geology to the structure and the orientation of critical discontinuities which have a significant bearing on the strength of the rock mass. Critical decisions have to be made during the exploratory phase on whether material properties must be determined from large scale field tests or whether they can be determined from laboratory tests on small samples taken from along discontinuities.

Rock Mechanics is employed in phase 2 of design (Table I) and generally consists of the determination of the appropriate material properties such as shear strength or compressibility from either field tests, laboratory tests, or from past experience. These properties are then used in an engineering mechanics analysis to compute the Factor of Safety, deformation, strains, or stresses which can be compared to the allowable or desired Factors of Safety, deformations, strains, or stresses for the problem at hand.

In phase 3 the structural concepts may be changed, or drainage may be considered, as well as other remedial measures which will make the structure safer and possibly more costly. These changes to the structure or design configuration are then re-analyzed in phase 2 until the appropriate Factor of Safety or deformations are achieved.

As indicated in Table I, the consideration of precedents from other similar structures is considered just as important as the analysis in phase 2 of design. A mature designer must have at his fingertips the facts about the performance and design assumptions for similar structures built in or on rock formations. Of course a detailed knowledge of the geology at the locations of these completed structures is essential such that a judgement can be made about the applicability of a particular precedent to the problem at hand. The use of precedents is greatly facilitated when the experience drawn from these completed projects is based upon well documented case histories. It is most important that the facts be accurate concerning the geology, the design assumptions, and the recorded performance. It is less important, but helpful, if there is a correlation between the method of analysis and the observed behavior. Too often the authors of case histories are so anxious to correlate the design analysis with the observed behavior that the completeness of the factual history suffers in the process. This diminishes the value of the case history over a period of time.

As indicated in Table I, the end result or "bottom line" product of the design phase (Step 5) is a set of plans and specifications which set forth the proportions of the structural elements or other elements of the design to be constructed. One purpose of the plans and specifications is to specify the quality and dimensions required by the design. Another purpose is to portray clearly to the contractor what is to be built and to portray the rock conditions encountered such that judgements can be made on such items as temporary stability during construction and other items that the contractor is extremely interested in which affect the constructability of the geometric configurations specified. The method of bidding and awarding of contracts for this construction must then be selected which will enable a fair comparison of the bids, allow for alternates submitted by the contractors, and which will take into account the possibility of conditions being encountered which may not have been anticipated in the geotechnical report, which is part of the plans and specifications.

As indicated in Table I, the selection of construction procedures and techniques are very important during the construction phase. This is usually done by the contractor, but in some cases may be participated in or approved by the engineer. The engineer is usually responsible for inspection of the contractor's work and observations of the actual geology encountered is usually documented or should be documented on as-built construction drawings. In some instances the monitoring of performance begins during construction and some changes may be considered either in the construction of temporary works or changes in the permanent structure if the mapped geology and measured performance are at extreme variance with what was anticipated.

As a final step in Rock Engineering, the performance of the finished structure under design conditions may be monitored and a comparison may be made between the predicted and actual performance.

Considering the many phases to Rock Engineering as indicated in Table I, it is obvious that there are many ways in which a well documented case history can contribute to our knowledge and facilitate decisions made on figure projects based upon facts which have been gathered from the past. Case histories can involve the use of various exploratory methods, the selection of material properties to be used in a particular method of analysis, the use of various types of analyses, the use of various exploratory methods, the selection of material properties to be used in a particular method of analysis, the use of various types of analyses, the use of a different structural configuration to solve a common type of problem, the use of a different way to write a specification or a different framework for bidding and the awarding of contracts, the use of various construction techniques such as controlled blasting or the progress rates through different kinds of rock with different tunnel boring machines during construction could be documented from actual projects. Many other aspects of the many activities of Rock Engineering indicated in Table I could be the subject of a useful case history. A valuable case history which will stand the test of time, however, is one in which the detailed geology, the design assumptions, and the analysis are well documented. It is also necessary for the project to be completed, and the actual performance of the structure be measured both during construction and in the final design configuration such that engineers reading the written case history can judge how well the assumptions agreed with the measured performance.

It is most helpful if the measured performance of the behavior during construction and final configuration is documented quantitatively rather than including only qualitative observations which may signify a different behavior to different people.

TABLE I

Activities in "Rock Engineering"

A. Design

1. Exploration
   a. General Geology
   b. Specific exploration with borings, adits etc.
2. Analysis Using Rock Mechanics
   a. Definition of Material Properties
      1) Stiffness
      2) Strength
      3) Permeability
   b. Physical analysis using geometry of structure, loadings, material properties, in the framework of engineering mechanics to predict the Factor of Safety, deformations etc.
3. Consider conceptual changes in structure drainage measures, or other changes which will result in desired Factor of Safety or allowable deformations.
4. Consideration of Precedents from the Past Design and Behavior of Similar Structures.
5. Development of Plans, Specifications, and Method of Bidding and Awarding of Contracts for Construction.

B. Construction

1. Contractor - Selection of Construction Procedures
2. Engineer - Inspection
3. Observation of Actual Geology
4. Monitoring of Performance
5. Consider changes based on Geology and Performance of Construction

C. Performance of Finished Structure

1. Monitor Performance Under Design Loads
2. Compare Predicted and Actual Performance

The special lecture and the papers submitted to this Session can be readily categorized into three categories according to the type of Rock Engineering problem discussed in the case history. As shown in Table II, the papers included in Category I are Case Histories of Tunnels and Underground Chambers, the papers in Category II are Case Histories of Dam Foundations, and the papers in Category III are Case Histories on Slope Stability. Of the nine papers available for review, six of these involve descriptions of case histories which have been completed. Three of the papers involve descriptions of projects which have not been built. In two of these cases, the design is complete, construction is in the early stages, and information is not given about the behavior or performance of the actual structure. In one of the three cases the design has not been completed and exploration, rock testing, and preliminary analyses are discussed.

TABLE II

Categories of Case Histories, Session 7

CATEGORY I

Case Histories on Tunnels and Underground Chambers

Paper	Title and Author
GL-9	Time Dependent Limit Stability of Tunnel and Dam Engineering in Difficult Rock by Tan Tjong Kie
702	Case History - Stillwater Tunnel, Central Utah Project, Utah, U.S.A. by R. S. Sinha and K. D. Schoeman
705	Pittsburgh's Mt. Lebanon Tunnels - A Case History by G. L. Butler, B. P. Cavan, F. K. Mussger, G. W. Rhodes and H. T. Whitney
707	Mechanised Rock Tunnelling in Adverse Conditions by I. McFeat-Smith
709	The Stability of Underground Power Chambers in Brittle Rock by Weishen Zhu, Kejun Wang and Guangzhong Peng

CATEGORY II

Case Histories of Dam Foundations

Paper	Title and Author
GL-9	Time Dependent Limit Stability of Tunnel and Dam Engineering in Difficult Rock by Tan Tjong Kie
708	The Experience of a Dam Founded on Difficult Rock Foundation by Jian-Yun Mei and Yu-Yin Guo

CATEGORY III

Case Histories on Slope Stability

Paper	Title and Author
701	Shale Pit Slopes: A Case History by D. H. Shields
710	Geotechnical Problems in a Bridge Over Corinth Canal by S. G. Christoulas, N. A. Kalteziotis, and G. K. Tsiambaos
713	Research on Slope Stability of a Certain Open-Pit Mine by Wang Wuling

Case Histories of Tunnels and Underground Chambers

In the first part of the Guest Lecture (GL-9) Tan describes squeezing ground in two different railway tunnels and a series of mining galleries. In Case I, Railway Tunnel I, a horseshoe shaped tunnel was described in a sedimentary sequence of sandstones and mudstones where the walls and crown of the tunnel were constructed of concrete blocks 50 cm thick, tightly cemented together, but there was no structural floor slab or arch to form a complete structural ring. After about 3 1/2 years, heaving of the floor was observed in the worst sections of the tunnel which amounted to about 10 to 30 cm. In Case II, Railway Tunnel II, another horseshoe section in an area with 500 meters overburden was observed to experience squeezing in a formation which consisted of alternating layers of limestone and slate, banded limestones, limestones and conglomerates, and alternating layers of sandstones and slates separated by thin sheets of argillaceous material. In general, the whole formation was faulted and fissured. The most severe squeezing was observed in the middle section of the tunnel where the formations consisted of slates separated by thin weak clay seams. No serious problems were encountered in the other sections which were provided with bottom arches. In the limestone formations bottom heave was not experienced even when bottom arches were not installed. It was found that in the cases of serious damage, the installation of bottom arches to complete the structural lining around the entire opening proved to give satisfactory results and stop the deformations. Further observation is necessary to clarify this behavior. In both railway tunnel case histories continuous quantitative measurements of displacement as a function of time were not available.

In Case III a series of mine galleries were described at a depth of approximately 400 meters in a folded region containing paleozoic rocks composed of metamorphosed limestones, schistose gneisses, and interbedded clayey shale, mudstone, siltstone, and conglomerate. At the depth of the galleries the horizontal tectonic stresses were on the order of 300 kilograms per square centimeter as determined by over coring tests in the more competent rocks. The lining consisted of 35 cm thick concrete blocks cemented together to form a wall on the sides and crown of the tunnel but the tunnel invert was left unlined and squeezing at the bottom was accompanied by bottom heave of the tunnel floor. It was found that a complete circular ring of the concrete blocks had to be built before the tunnels would have a chance of being stable. Experimentation also showed that the tunnels were much more stable if the gap between the rock formation and the circular cement block lining was filled with grout in order to prevent loosening and deterioration of strength of the rock mass. The grout developed the confining pressures of the lining on the rock at lower deformations and resulted in less distortion of the concrete linings. In this mine a complete circular 4 meter diameter liner of cemented concrete bloc was also utilized in conjunction with 2 meter-long rock bolts. This configuration was very stable in the same geology which has been described previously for this case history.

A test section of shotcrete and bolts wa carried out in the same mine in weathered schistose gneiss with many planes of weaknes

The opening had a span of about 5 meters and a height of about 6 meters. It was found that after 2 years that the inward displacements were on the order of 2 to 12 cm and it was found that fracturing in the roof and bottom upheaval were increasing with time. Additional bolts did not serve to arrest these movements and it was attributed to the fact that the bolts were too short (2 m) and did not extend beyond the loosened zone into the undisturbed rock formation.

On the basis of these case histories the author has concluded the following:

1. A completely closed lining is necessary in squeezing ground.
2. Displacements take place preferably along weak discontinuities hence the surrounding rock should be reinforced by bolting and should be thoroughly grouted.
3. The filling of the space between the surrounding rocks and the lining by grouting is necessary.
4. The bolts must be long enough such that the annulus of the loosened zone is exceeded and the bolts can be anchored in the undisturbed regions.
5. The fissures around the cavity must be grouted in order to obtain better cooperation between the bolts and rock.

An elastic-plastic finite element analysis is presented which gives the depth of the yielded zone as a function of the strength of the rock and the level of the tectonic stress but the results do not indicate that there is any time dependency included in the material properties used in the analysis which would indicate the closure as a function of time. Although the analysis does not appear to be to the point where it is useful in predicting squeeze or closure, the empirical observations given in the paper are valuable in that they do indicate that the squeezing problem can be handled if the support system is installed before significant loosening occurs and if a continuous structural ring such as a circular lining capable of putting significant confining pressure back on the rock medium is used.

In the paper by Sinha and Schoeman the case history of the Stillwater Tunnel in the Central Utah Project is discussed. The majority of this tunnel is on the order of 2600 ft deep and was to be advanced in a Precambrian Formation called the Red Pine Shale. The shale was described as "greenish-gray to black in color; is hard to soft, laminated to fissile and indurated; and has a high clay content of illite and kaolinite and some siderite. It air slakes on exposure and contains some very well cemented interbedded sandstone with beds that vary in thickness. The alignment was cut by faults and shear zones. Jointing and fracturing was more severe near the fault and shear zones. The faults were steeply dipping." The writers gave the results of some rock tests such as unconfined compressive strength, which ranged from 2600 to 12,800 lb/in.^{2}; however, these ranges are broad and the authors did not attach the variations in strengths to variations in lithology within the formation. The reader is left with an inadequate correlation between the measured strengths and the lithology of the rock specimens tested. This is very important because if a large portion of the tunnel were in the weaker strength materials, the overburden pressures could be the same order of magnitude of the unconfined compressive strength of the rock and one might expect areas of some significant movements of the tunnel walls and might wish to account for this not only in the lining design but in the selection of the construction procedures. It was noted by the authors that the tunnel lining design for the first contract was a segmented precast concrete tunnel liner which was 5 in. thick and was about 8 ft in diameter. The design load was based upon a vertical rock load equivalent to three times the tunnel diameter according to Terzaghi (1946). Thus, the design load was based upon a loosening load rather than a load which could be caused by squeezing ground. The authors indicate that the segments were anlayzed and found to be capable of withstanding a uniform pressure of 165 psi but they did not comment on how this pressure capability might be reduced due to unequal loading causing a combination of bending moments and circumferential thrusts in the segments. The majority of the first tunnel contract was bid on the basis of using a tunnel boring machine for excavation and the use of precast segmented concrete liner elements for support. The bids were based in part on unit prices and in part on a bid price per lineal foot of completed tunnel. The tunnel boring machine selected had a long shield and it was ultimately found that the thrusts required to push the shield were higher than the thrusts which could be obtained from the gripper pads. During down times the machine was frozen from time to time as the squeezing ground increased the friction on the shield. In those instances when the machine was stalled an attempt was made to get additional thrust capacity by jacking the TBM longitudinaly against the installed segmented concrete liners with the auxiliary horizontal thrust rings. It was found that the segmented liner elements could not take this thrust without damage as the pea gravel packing alone did not hold the segments firmly in place. During this thrusting, many segments were damaged and had to be replaced. This contract was terminated in September of 1979.

For the completion contract, the USBR requested proposals from contractors on the basis of a fixed-price incentive (firm target) contract. According to the method as described by the authors, the contractors and the owner share the savings if the total cost is below the target cost and the contractors profit is reduced if the target cost is exceeded. The proposals of the contractors were rated on both the target cost, the technical merit of the proposed tunneling procedures and methods, technical experience, and management capacity. The successful contractor happened to also be the low bidder and he proposed to tunnel from both ends of the tunnel. At the Inlet end of the tunnel, a new tunnel boring machine was used which had a shorter shield; the machine utilized a finger shield under which steel support rings and lagging were installed. The steel ribs were expanded tightly against the rock with hydraulic jacks as the shield was advanced. This machine, on the completion contract, mined slightly more than 25,000 ft of tunnel. The machine from the previous contract was modified and started from the outlet end of the tunnel. Even with the modifications of increased jacking capacity the modified machine mined only 3,900 ft of tunnel and the performance was not necessarily satisfactory. Driving of this tunnel has been completed at this point and has been judged by many to be a success. This case history illustrates the importance of the design and selection of a particular tunneling machine which has the flexibility and capacity to handle various combinations of squeezing and ravelling ground. It is also an illustration of the use of a contracting method, for the completion contract which promotes cooperation between the contractor and the owner and which provides a mechanism in which they both share in the risks. This case history serves to illustrate that two of the most important aspects of rock engineering do not necessarily involve the detailed aspects of rock mechanics. These two areas, as I have previously shown on Table II, are those portions of Rock Engineering associated with the selection of the construction procedure and the formulation of the framework for the specifications and bid documents. Another interesting aspect of the first contract case is the illustration that sometimes the forces for which a liner must be designed are not necessarily uniform external loads and that the critical forces which may tend to fail the liner may indeed be jacking forces if the equipment selected requires abnormally high jacking forces to advance the shield. It was also apparent that the pea gravel backpacking in the first contract was not as desirable as a very thick special grout mix which was used as backfill in the completion contract which resulted in a more uniform load on the structural lining.

Several test sections were referred to in the paper where extensometers have been used to measure the displacement at various depths behind the tunnel walls as a function of time; at this time the data does not appear to have been fully analyzed and a correlation between this behavior and the rock properties at these sections should be a fruitful area for research and for future papers about this case history.

In Paper 705 entitled "Pittsburgh's Mt. Lebanon Tunnel - A Case History", Butler et al describe the progress to date on this case history up through the bidding process. Since construction has just recently started it is felt that one must look at this as an interesting progress report and that a more complete case history will result at a later date when the construction is finished. The main theme of the authors throughout the paper is that the Mt. Lebanon Tunnels, using the "New Austrian Tunneling Method, is the first significant application of this foreign technology in U.S. design". This project is a demonstration project of the Urban Mass Transportation Administration (UMTA). For this project, UMTA financed the design of two alternates. Option A is described by the authors as representing U.S. design practice and consisted of an option for bidders with a tunnel driven by blasting, temporarily supported with rock bolts, and lined with a cast-in-place reinforced concrete liner. Under Option A the contractors were to bid tunneling costs as a lump sum and the cast-in-place concrete lining as a lump sum. The bidders were also presented an Option B, which is described by the authors as the New Austrian Tunneling Method. By this method a contractor bids a unit price per lineal foot of tunnel and on this particular option, there was a different unit price to be bid for Type I ground, Type II ground and Type III ground. Unit prices were for excavation and initial support. Inner lining shotcrete and cast-in-place concrete for any permanent portions of the liner, as well as engineer ordered tension rock bolts, were bid also on a unit price. In Option B, there was a definition of the type of initial rock supports which were to be consistent with the classification of the ground as Type I, Type II, or Type III. By this method, however, the type of ground is determined in the field as tunneling progresses with the contractor having the responsibility for initially determining the type of ground, subject to the approval by the engineer. In the event of disagreements, a "unilateral" determination is to be made by the engineer. In order to compare the two options the authors have presented the results of the bids from sixteen different contractors. The bid price was divided between the base bid, which related to non-tunneling items, and the bid price for either Option A or B, for the tunnel portion of the contract. The bid was awarded by the total low bid of these two different items. Only one contractor bid Option A. This is understandable because even though it appears from the profiles described, that a good portion of the tunnel will be in competent limestone, that the only alternative available in Option A would be to go through with the excavation and temporary support and then construct the cast-in-place reinforced concrete tunnel lining in a second phase. It obviously would be cheaper and more economical in the portions of the tunnel where the rock is competent to use rock bolts, shotcrete, and mesh as both a temporary and permanent lining. This type of support, by the way, is not foreign to the United States, nor is it unique to the New Austrian Tunnel Method. Many tunnels and caverns in the United States have been done with both concrete linings and with permanent linings of shotcrete and bolts. What is different in Option B as compared to most frameworks within which bidding takes place, is that the classification of the ground into Type I, Type II, or Type III and thus payment for support is determined in the field during construction and after bidding, thus both the owner and the contractor are sharing in the risk; and, the cost of the tunnel really is not known until the project is finished. Thus it appears to this reporter, that the "New Austrian Tunneling Method" is a bidding framework where both the contractor and the owner share in the risks for the actual ground conditions encountered. This does not necessarily mean, however, that controversy is eliminated since there is still room for disagreement between the engineer and the contractor on the classification of the ground and the support system to be used, even though the geology is exposed at that time. It has also been pointed out in the previous case history (Paper 702) that there are other forms of sharing the risk such as the target cost type of contract used by the Bureau of Reclamation in the completion contract on the Stillwater Tunnel. This reporter does not accept the premise of the authors that installation of initial supports quickly and the use of rock bolts and shotcrete are foreign to U.S. practice and at the same time, unique to the New Austrian Tunnel Method. There have been many specifications written for projects in this country where the initial support has been required to be close to or at the face at all times. There are large caverns, already constructed, where the only means of supports is either rock bolts or rock bolts and shotcrete.

It is interesting to note that the low bidder for the Mt. Lebanon Tunnels was among the highest bidders on the tunneling portion of the project and was low bidder primarily because of his low bid on the base bid portion which had nothing to do with tunneling. It is also interesting that the only bidder for Option A gave a firm lump sum bid of $10,800,000.00 for the tunneling portion whereas the low bidder for the project bid $10,000,000.00 for Option B. In view of the fact that the engineer's estimates for Option A and B were $16.8 million and $14.7 million respectively, it appears that, within the accuracy of the bidding that the tunnel portion of the bidder's bid and the tunnel bid of the only contractor to bid Option A were virtually identical. The only basic difference between the bids is that the price of bidder #15 of $10,800,000.00 would be firm, and the price of $10,000,000.00 for bidder #1 under Option B has yet to be determined and could be greater or less than $10,000,000.00 because of the classification of ground which takes place in the field. The other difference for the owner is that for bidder #15, the owner would be getting a reinforced concrete liner for the entire tunnel under Option A, whereas for bidder #1 under Option B, according to and depending upon the agreements or disagreements in the field, the owner could be getting as little as 6 in. of shotcrete for a permanent lining with no pattern bolting if the engineer and contractor agree to this support requirement in the field.

It appears to this reporter that a real opportunity for a comparison for both cost and performance was missed on this demonstration project because it would have been possible to do one of the twin tubes by Option A and one of the twin tubes by Option B. This would have enabled a direct comparison of the costs and performance of both options in nearly identical geologies.

It is also emphasized that this is an extremely short tunnel, so short that the investment of any tunnel boring machine for this project would have been unwarranted. For longer tunnels in the geology present at this location, contractors would have most likely been bidding the project using some type of tunnel boring machine which could have made the economics entirely different.

In Paper 707, McFeat-Smith gives two case histories concerned with mechanized rock tunneling in adverse conditions. The first case history involves the use of a roadheader tunneling machine in a sequence of limestones, mudstones, argillaceous siltstones, and argillaceous sandstone. From this case history it is shown that the roadheader tunneling machine makes an average progress on the order of about 25 meters per week in the mudstones and about 15 meters per week in cherty limestones. The rate of progress is reduced to about 7 meters per week in a silaceous standstone which is definitely a rock which is too hard and strong for this type of machine. It is concluded that the machine is appropriate for the mudstones, sandy mudstones, and argillaceous siltstones, but that progress by drilling and blasting is bette than the roadheader progress in cherty limestones and in silaceous sandstones. Various data on the wearing down of the picks is given and generally this is a case history which give the reader the experience for this type of machine in the rocks encountered for a 4 meter diameter tunnel. It would be helpful if the author would discuss the types of laboratory tests which were used to test the hardness of the rocks since they are not explicitly discussed in the paper.

In case history 2, the author describes tunneling progress with a tunnel boring machine with disc cutters in a geologic sequence involuntary mudstone, silty mudstone, silaceous sandstones, and dolerites. The average progres for this 3 1/2 meter diameter tunnel was about 100 meters a week in the mudstone and diminishe to as little as about 30 to 40 meters per week in a dolerite sill where button cutters had to be used in place of the disc cutters. Progress was on the order of about 70 meters a week in the silaceous sandstone where button cutters were also used instead of disc cutters. This is a project where there was a changed conditio claimed by the contractor because from the initial geology it was not apparent that the dolerite sill, which was the main obstacle to progress, would be encountered in the tunnel alignment. It had been assumed that it would be below the tunnel alignment. This case history is helpful and typical of the problems which could be encountered with tunnel boring machines in rock strata of varying hardness.

In Paper 709, Weishen gives a paper entitled "The Stability of Underground Power Chambers in Brittle Rock". In this paper it is observed that core discing occurs during drilling for rock cores and, as is well known, this is an indication of high stresses in the rock mass, such that relief fractures occur as the core hole is advanced. In-situ stress investigations have indicated that horizontal stresses as high as 650 kg per cm^2 are present in the area of the planned powerhouse where this coring was done, and it has been further shown that core discing occurs mainly in a seyenite rock which has been intruded into a basalt formation. Although finite element analyses and model tests have been conducted to infer the possible behavior caused around underground openings subjected to this stress field, the design of these chambers has not been completed so it is not obvious what the effect will be on the design selected for this project. This project is another case history which will be very interesting when the design and constructio are complete.

Case Histories of Dam Foundations

In a guest lecture, GL-9, Tan describes foundations for Ghe Zou Ba Dam which is a crete dam located on horizontal layers of ernating mudstone and sandstone layers arated by weak argillaceous bedding layers. was pointed out that these layers, parallel the bedding, had a very low cohesion on the er of 0.1 kg per square cm and tan ϕ values ging from .19 to .23. It was pointed out t these layers contained clay minerals conting of illites and montmorillonites. It pointed out that for stability of the dam, art of the horizontal stability to resist water forces was due to the shear forces on base of the dam as well as a "reaction ce" which is a passive force imparted to the by the rock formations downstream which are ve the base of the dam key. In the passive e in the rock just downstream of the dam, cern was expressed by the author for the sible buckling of beds. In the design, beds were bolted to prevent disintegration the thinner layers by buckling. It was also wn that a cutoff was needed on the upstream le of the dam to reduce uplift, however, re is no discussion of possible drainage ts in the dam which could be used to increase bility by means of drain holes beneath the e of the dam but downstream of the cutoff. seems as though this dam design is somewhat ing in that a downstream passive block is lied on for stability, whereas in most cases rall stability is usually dependent upon a nbination of an upstream cutoff, drainage neath the base of the dam, and the weight of dam such that the base shear will keep structure in equilibrium without the benefit a downstream passive block. This is diffilt, however, to achieve where the foundations ve the low angles of shearing resistance, as dicated for the test data given in this paper. reporter is in full agreement with the thor in the use of a very low cohesion interpt. Other case histories available in such dimentary sequences as those described in is paper really show from the back calculaon of shear strengths at failure that if a hesion can be counted on for these cases, it indeed very low and one must depend primarily on the angle of shearing resistance for resisnce.

In Paper 708 Jian gives a case history for concrete dam founded on a sedimentary rock undation. For this case the sedimentary rocks e dipping in a downstream direction on the der of 10 to 15°. The sequence consists of claystone, fine sandstone, a clayey silty ndstone, and some sandy shales. In-situ rect shear tests and in-situ compressibility sts were conducted on these various formations om tests in construction adits. In the interetation of shear strength parameters the thor uses yield values of the shear strength r design rather than the ultimate peak values the shear strength at a given normal effecve stress. In using these shear strength lues in a limit equilibrium method of analysis presented in Table IV of the paper, the ctors of safety without considering hydraulic lift on the sliding plane, the ranges in ctor of safety using slightly different methods of calculation range from 1.11 to 1.26. If the uplift pressures are assumed, the factor of safety ranges from 1.03 to 1.19. These are factors of safety lower than we would normally work with, however, it must be pointed out that the ultimate shear resistances may be 20 to 50% higher than the yield shearing resistances which partially compensates and which would probably yield factors of safety on the order of 1.5 if the ultimate values of shearing resistance had been used. It is very important to note that for this particular dam that an upstream cutoff was used and this is very important in a situation where strata are dipping downstream as it would be very easy to develop high excessive poor pressures on the downstream side of the dam. Additional stability was obtained by adding more weight to the structure downstream than would normally be added for a concrete gravity structure. It is pointed out that no discussion is given concerning drainage holes downstream of the cutoff which would also greatly facilitate the reduction in uplift pressures.

Case Histories Involving Slope Stability

In Paper 713, Wang discusses the stability of a specific open pit mine which was initially designed and found to be unstable two years after the mine came into operation. Redesigns were twice made in 1961 and 1963 for the final slope boundary. Even then final slope stability was not achieved. After 1963 detailed rock mechanic studies were used to investigate the shear strength and other slope stability considerations. These investigations included insitu shear strength tests in the field along discontinuities, it included detailed investigations on the effects of blasting on slope stability and the subsequent control of blasting. Unfortunately there is not a cross-section in the paper of the open pit mine given for the initial condition of the slope and the flattened condition of the slope such that comments can be made relating these slope angles to other case histories for which the behavior and the slope angles are known. It would be interesting if the author could show the cross-sections for the open pit in relationship to the geology for the initial condition when it was found to be unstable versus the cross section of the open pit for the last 20 years where it was supposed to have been stable.

In Paper 710 Christoulas et. al. discuss the slopes of a canal in the vicinity of a railroad bridge which runs across this canal at a location near Corinth, Greece. The canal is located in a profile of marly limestone overlying a whitish yellow marl, a marly sand, and a whitish gray laminated marl. The canal slopes are as much as 75 meters high and although a cross section is not shown in the paper it appears as if the canal slopes are on the order of 50 to 60° with the horizontal. In addition to the horizontal bedding in the marls it has been observed that there are some nearly vertical joints which run parallel to the main tectonic faults of the area and which make an angle of 30 to 40° with the axis of the canal. After the earthquake of September 5, 1953 an extension of these existing joints was observed close to the northern abutment of the bridge and after the 1981 earthquake of February 24th, a study was initiated to investigate the safety of the bridge because small pieces of glass placed on the joints as instrumentation were broken as these vertical joints were again extended during the earthquake. The slopes have generally been found to be stable statically and as a result of this study a series of untensioned grouted rock dowels were inclined at 45° and directed normal to the strike of these joints to tie the mass together around the abutment of the bridge such that opening of the vertical joints would not be observed. This seemed to be an appropriate course of action since static stability of the canal had not been observed to be a problem since it was built in 1882.

In Paper 701, Shields discusses a case history of a proposed open pit mine in Indonesia. The open pit mine is to be a coal mine which is a formation composed of coal seams, claystones, sandy claystones, and coarse sandstones. The stability, of course, in such an instance would be governed by the claystones. The investigation consisted of conducting direct shear tests on the clay shales which involved shear strength tests of the intact peak strength, and the peak shear strength along existing discontinuities, as well as the residual shear strength along precut and cut polished surfaces. For design, the peak shear strength along discontinuities was used with the assumption that enough drainage would be installed to bring the water levels beneath any potential failure surfaces. This resulted in using effective cohesion intercepts of 0 and peak shear strengths along discontinuities ranged from 22 to 35° in the various layers above the coal. The overall slope angle selected for design was on the order of 22° and it was anticipated that the factor of safety at this slope angle would be on the order of 1.1. This case will be an interesting case history when the pit actually gets under construction. At this time this reporter does not want to get into the pros and cons of using probability analysis in slope stability but I would simply state that I am in agreement with one of the conclusions of the author that it is not yet ready to be used for the geotechnical engineering of slope stability problems.

General Report: Session 7 by A.J. Hendron

Discussion by Tan Tjong Kie (China)

With regards to the remarks of the general reporter Professor Hendron to my guest lecture. "Time Dependent Limit Stability of Tunnel and Dam Engineering in Difficult Rocks". I will make the following comments and further explanations.

In order to get an overall insight into the squeezing process in tunnels, and the measure to be taken to prevent it, it is firstly of primary importance to get an idea of the volume of rock which can be loosened and move towards the cavity. For this purpose a pre-analysis with the help of finite elements is helpfull. Usually an elastic plastic finite element analysis is performed based on the Drucker Prager theory, whereby the associated flow rule is assumed. This rule is known as the normality principle, as the plastic strain increment vector is normal to the yield surface. However this concept leads to unreasonable volume dilatancy, hence some modification is introduced, and other forms of plastic potentials are assumed.

On the basis of many experimental results of rocks, I have found that void and fissure formation is generated as soon as the stresses in $\sigma_1, \sigma_2, \sigma_3$ space exceed the limiting surface of the upper yield value f_3. — and on this concept I have derived the consitutive equations for creep and time dependent dilatancy. (Tan, Kang, 1980; Tan, Kang, 1983).

Instead of the costumary Drucker Prager theory I prefer to make use of these new equations, which give a relationship between the stress-strain tensors and the time and describes both regions for stresses less than f_3 and higher than f_3. For practical purposes however it is sufficient to estimate the extension of the dilatant region and for this purpose it is sufficient to consider only the instantanious part of the deformation. In this special case the constitutive equations are largely simplified and can be expressed as follows (Tan, Wen, 1983).

$$p = I_1 = (\sigma_x + \sigma_y)/2 = (\sigma_1 + \sigma_3)/2$$

$$\sigma_{oct} = \sqrt{\mathcal{J}_2} = \left[\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2 \right]^{\frac{1}{2}} = \frac{\sigma_1 - \sigma_3}{2}$$

$$e_d = e_{dx} + e_{dy}$$

$$e_{dx} = \left(\frac{\sigma_1 - \sigma_3}{2f_3}\right)^n \left[D^* + C\frac{\sigma_x - \sigma_y}{\sigma_1 - \sigma_3} \right] \qquad (1)$$

$$e_{dy} = \left(\frac{\sigma_1 - \sigma_3}{2f_3}\right)^n \left[D^* + C\frac{\sigma_y - \sigma_x}{\sigma_1 - \sigma_3} \right] \qquad (2)$$

$$\gamma_{xy} = 2e_{xy} = 4C\left(\frac{\sigma_1 - \sigma_3}{2f_3}\right)^n \frac{\tau_{xy}}{\sigma_1 - \sigma_3} \qquad (3)$$

$$e_d = 2D^* \left(\frac{\sigma_1 - \sigma_3}{2f_3}\right)^n \qquad (4)$$

In the above equations:

$\sigma_x, \sigma_y, \tau_{xy}$ = stresses; e_{dx}, e_{dy}, γ_{xy} = dilatant strains, e_d = dilatant volume strain C and D^*, n are material parameters. The solution of the above equations is shown in fig 8 of the lecture.

The extension of the dilatant zone is sensitive to the magnitude of the upper yield value f_3 under similar stress conditions. From the extension of these unstable zones the engineer can have an idea how to bolt and anchor and strengthen the tunnel. The computation of the time dependent squeezing of tunnels based on the complete equations of creep and dilatancy is very laborous as it involves a finite deformation incremental strain technique progressing with the time and is now being studied.

With regards to the foundation design of the Ghe Zhou dam, which I describe in my guest lecture, I wish to give some supplementary information. Amongst dam engineers the idea is circulating that the weight of the dam is such that the base shear will keep the structure in equilibrium without the benefit of a downstream passive block. In this type of design there are important factors which are uncertain as for instance:

a. The shear strength parameters in such a case are usually derived from short-term routine shear tests, and it is known that these routine tests gives unreasonably larger cohesians and frictional angles than the long term tests. For instance the parameters for routine tests are $C = 0.60$; $\tan\varphi = 0.24$ in comparison with $C \sim 0.10$ and $\tan\varphi = 0.20$ for long term creep tests;

b. Another uncertain factor is the resisting force of the downstream block;

c. Further the mutual interaction between the resisting block and the bedding layers is unknown. Crucial is the ratio: the resisting force of block/the resisting force of bedding layer on the long term.

Since in the Ghe Zou dam the cohesion $C \sim 0$ and $\tan\varphi = 0.20$, an increase in weight of the dam then will not be of much help; we must either make proper use of the resistance of down stream block or we must transfer the horizontal stresses to deeper and stronger layers by oblique piles or concrete columns. In our case the first alternative was preferred i.e. we make use of cutoffs, aprons, screens, strengthening of the resisting block.

Of course the reduction of the uplift pressure by means of drainage galleries within the front and downstreams cutoffs in combination with the customary drainage aidits, an important item in the design (Fig. 1)

A large number of finite element computations and model tests was necessary. In view of the fundamental importance of this dam in the Yangtze-river, the most sophisticated methods were applied and all possibilities investigated thoroughly, before we could obtain the ultimate efficient design of the project.

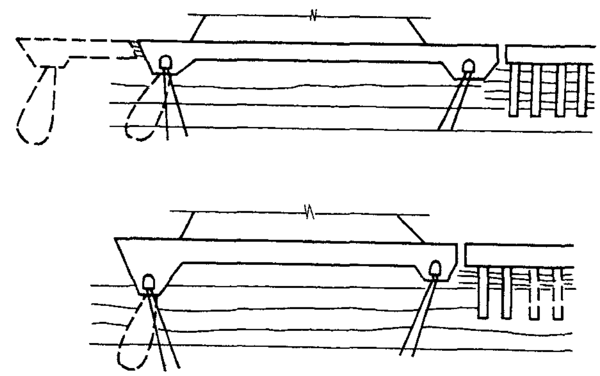

fig 1

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scussion by D.H. Shields,
·ofessor of Civil Engineering,
.iversity of Manitoba, Winnipeg,
.nitoba, Canada R3T 2N2 on "Research
ι Slope Stability of a Certain Open-
.t Mine" by Wang Wuling.

The author makes a valuable contribution to the :story of open pit mine engineering. It is heartening › note the care and attention given to understanding ıe physical geology of the rock surrounding the pit, ı particular to understanding the fracture pattern. ıe approach seems to have been that by understanding ıe deformation processes that the rock has undergone ı the past (folding and faulting mainly), the direction ıd frequency of fractures and joints can be predicted ith relative certainty. Given the complex fracture attern at the site and the varying conditions in ıe fractures themselves (ranging from crushed breccia, ırough calcareous deposits to argillaceous infilling), t would be interesting to learn more about the design hilosophy. Were 'worst case' scenarios used to design ach slope i.e. lowest strength, worst possible joint racture direction, and highest probable water ressures. Or were average values used, for example.

Judging from the statement that "Practice of hese twenty years have proved that all slopes are table and safe that were treated in the light of uggested reasonable slope angle and required measure", he writer infers that the 'worst case' philosophy s the more probable. If this is so, it is unlikely hat the design slopes were the most economical. .ssuming, say, higher strengths, less critical fracture ·rientation and lower water pressures would have led o steeper slopes with, admittedly, a higher probability ›f failure; safeguards in the form of pit slope ıonitoring could have been implemented to ensure there ·ere no catastrophic failures. The point being made ıere is that it is not good enough to simply design ın open pit slope, one has to live with it on a day :o day basis and improve the design as experience lictates. Only then can one be certain that the mining ›peration was carried out at lowest cost.

Response to Session 7 General Report by Alfred J. Hendron, Jr. on the paper "Pittsburgh's Mt. Lebanon Tunnels - A Case History" presented at the International Conference on Case Histories, 1984, St. Louis, Missouri, USA.

The authors' would like to express their appreciation to the Conference Organizing Committee for the opportunity to reply to Professor Hendron's General Report. Because of time overruns by other participants during Session 7 we were not permitted to make our scheduled presentation and replies.

Professor Hendron's comments appear to fall into four broad categories: 1) Practice in Rock Engineering, 2) U.S. Tunneling Practice, 3) Bid Documents and 4) Construction Bid Summary. Additionally, in a manner which was very general and unrelated to the subject of the paper, Professor Hendron indulged in speculation concerning the use of alternate methods of excavation at this project.

With regard to these broad subjects, the authors offer the following comments:

Practice in Rock Engineering: As most experienced designers know there are several phases and many activities involved in project development. The relationship of these in the case of the Mt. Lebanon Tunnels is demonstrated in Figure 1. Rock engineering activities as outlined by Professor Hendron played an essential role in each phase of this project. The authors' purpose was to demonstrate this interaction from the Planning through the Final Design Phases. The label "an interesting progress report" is, in the opinion of the authors, both misleading and inaccurate.

The essence of the NATM is a philosophy which originates in the Planning and Design Phases and is carried through the Construction Phase. This philosophy is outlined in great detail in the paper and affects not only the selection of rock mass properties, tunnel geometries and support systems but also the manner in which the design is implemented in the field, how variations in ground conditions are handled and how evaluations of design during construction are made.

U.S. Tunnel Practice: Tunnel Option A, as described in the paper, was designed by Parsons, Brinckerhoff, Quade and Douglas, one of this country's foremost and respected tunnel design firms. The authors make no assertions as to the degree of representativeness of Option A to U.S. design practice. Law/Geoconsult was engaged to provide a "state-of-the-art" design representing worldwide NATM practice.

THE CHALLENGE OF PROJECT DEVELOPMENT

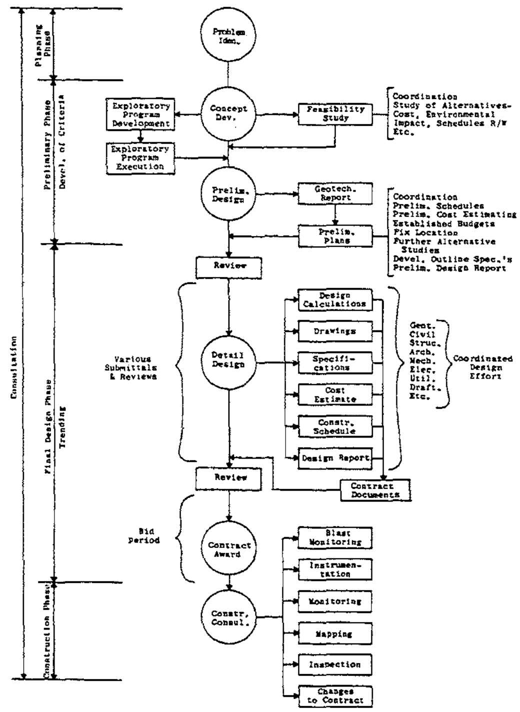

Figure 1

As the authors clearly point out, NATM is not confined to a particular method or sequence of excavation, any type of specialized equipment or single type of ground support. Furthermore, as pointed out in the paper, many elements of the Option B design are currently used in U.S. tunneling practice. The prequalification process used during the Bid Period on this project demonstrates this clearly. Nineteen contractors out of a total of twenty one who qualified to construct Option A were also qualified to construct Option B. Of these nineteen contractors, eighteen were United States companies which were able to demonstrate 10 years of experience in the basic construction procedures anticipated by the NATM design.

Of particular concern to the authors was Professor Hendron's apparent prejudgement of the adequacy of the shotcrete liner, designed under Option B, as compared to the cast-in-place concrete lining designed under Option A. This was apparently done with no knowledge of the design requirements of either option. In fact, properly designed and constructed shotcrete linings have a demonstrated record of good performance throughout the world.

The authors recognize that many forms of risk sharing have evolved over the years on many different types of tunneling projects. Some of these have even been reported in the United States, most notably: "Better Contracting for Underground Construction", U.S. National Committee on Tunneling Technology, 1974. Option B brought some of these practices into the contract framework for the Mt. Lebanon Tunnel.

The authors readily admit in the paper that certain practices routinely in use in Europe, Japan and South America, do not as yet fit into traditional U.S. contract documents and specifications. These practices principally have to do with resolution of disagreements between the Owner and Contractor without costly litigation and are only one facet of NATM. In the authors' opinion there is no validity to Professor Hendron's contention that NATM is a "bidding framework" only. The authors do not present nor imply any condemnation of U.S. tunneling practices. There is, however, room for improvement through a natural evolution.

Bid Documents: One key to reduced tunneling costs is the optimization of ground support. In the NATM this is done by adjusting specified support to meet the actual conditions encountered. This requires flexibility not only in the design but in the Contract Documents as well. As the authors point out in the paper, classification of the ground is one way of doing this. The ground types in Option B specify tunneling sequence, type of ground support and length of acceptable heading advance. This is consistent with the NATM philosophy previously discussed. As the authors point out in the paper there is additional flexibility built into each of these ground types. This allows movement from one ground type to another with a minimum of disruption to tunneling cycles.

The authors purpose in presenting the bid items for both Option A and B was to demonstrate the flexibility of the bid items which are felt to impact cost-effective tunnel construction. Many times actual construction costs are hidden in bid items, e.g. contact grouting of cast-in-place concrete linings. In Option A grouting is an identified but a virtually indeterminate quantity. Professor Hendron's apparent contention that Contract Documents exist which are free of potential conflicts and hidden costs is simply untrue. Again this is the reason the authors stress that NATM is a philosophy which is applied to all phases of a project. It is not as Professor Hendron contends, confined to the Bid Period alone.

Professor Hendron's evaluation of the bid items was done based only on the information presented in the paper. This was not the intent of the authors. An intelligent comparison of design differences as reflected in the Bid Items requires a review of the design drawings and specifications and close scrutiny of conditions actually encountered during tunneling. For example, tunnel excavation is a lump sum item under Option A which includes direct rock support (steel ribs, chain link fabric and shotcrete), but rock reinforcement in the form of dowels, rock bolts and sealing shotcrete were included as unit price bid items. The specified limits, by tunnel station, where these direct support and reinforcement elements were to be placed were shown on the contract drawings.

Contrary to Professor Hendron's opinions concerning the lack of a firm bid price for the work, both Options A and B were bid under the same General and Special Contract Conditions and both had finite limits as to the contract price for the work specified.

Construction Bid Summary: The authors presented the Construction Bid Summary for the purpose of demonstrating the reaction of the construction industry to Option B. Unbalanced bidding is a fact which has to be accepted, Professor Hendron's comparison of the tunnel price of the successful low bidder to the only Option A bidder is misleading. In the authors' opinion any analysis of the bids beyond this point is purely opinion and certainly beyond the scope of the subject of this paper.

Another erroneous conclusion reached by Professor Hendron is that "a good portion of the tunnel will be in competent limestone". In fact, as alluded to in the paper and since confirmed in the tunnel excavations, the majority of the rock along the alignment consists of poor to fair quality, interbedded siltstone, shale, sandstone and limestone. The comments, therefore, regarding the impact of a competent limestone condition on the economics of the project are irrelevant.

The authors strongly disagree with Professor Hendron's opinion that "a real opportunity for a comparison was missed". Presumably the engineer's estimator considered the various differences in construction sequencing, materials and time in arriving at an approximately $1.1 million difference between Option A and B. Implementation of a program as outlined in the General Report would require that the Port Authority of Allegheny County issue either separate contracts or two sets of bid documents for construction. Systemwide constraints would not permit this luxury not to mention the construction management difficulties inherent in such a scheme. Additionally, construction economy due to optimizing the work sequences by using the same equipment in both tunnels would be lost. The authors believe, as did the funding agency, that by allowing both options to compete in an open market, the purpose of demonstration of applicability is better served.

General: In the Planning and Preliminary Design Phases of this project, alternative excavation methods by tunnel boring machine and other mechanical means were studied for both options. The authors believe that second guessing of excavation methodologies and their impact on project economics is opinion at best and not really relevant to the subject of this paper.

Reply to discussions of Alfred J. Hendron, Jr., on the paper "Case history - Stillwater Tunnel, Central Utah Project, Utah, USA" by R. S. Sinha and K. D. Schoeman; presented at the International Conference on Case Histories, 1984, St. Louis, Missouri, USA.

At the outset, the intended purpose of the captioned paper by Sinha and Schoeman was to point out that for the successful completion of a very deep and very long tunneling project, the essential elements are (1) adoption of flexible design, construction, and support methods; (2) development of a contracting procedure which provides incentive to the Contractor and promotes goodwill between the owner and the Contractor; and (3) documentation of the intent to share the risks between the concerned parties. As pointed out in the discussions of Dr. Hendron of the subject paper, the authors have successfully demonstrated the validity of those essential elements.

The areas that needed more attention in the subject paper, according to Dr. Hendron, are (1) correlation between the lithology of the rock and the compressive strength of the rock; (2) identification of tunnel areas that could create problems in excavation and support during construction; (3) prediction of magnitude of loading on tunnel supports based on the correlative index of lithology and compressive strength; and (4) documentation on the strength reduction mechanism of precast concrete segments due to nonuniform loading, machine jack thrust, and circumferential loading. The authors comments are as follows:

a. Correlation Between Lithology and Compressive Strength

Based on evaluation of regional and site geological information and laboratory tests on rock samples obtained from 18 boreholes in portal areas, the rock lithology was considered to be uniform, that is, Red Pine Shale. Therefore, correlative index between variation of lithology and variation of compressive strength was considered not practical and was not developed.

b. Identification of Problem Areas

The tunnel areas that could create either support or excavation problems during actual construction were identified based on the evaluation of variations in compressive strength, joints, and fracture patterns and estimated locations of fault zones.

These considerations are documented in the USBR publications "Geologic Factors of Engineering Significance for Stillwater Completion Contract" - March 1981 and "Construction and Foundation Materials Test Data and Stillwater Tunnel Instrumentation Data" - February 1981.

However, as pointed out in the subject paper, "The fault that finally stopped the TBM of the first contract was not mapped at the prebid stage." The problems in predicting the behavior of supports during actual excavations for deep tunnels are adequately highlighted in the subject paper, section No. 3 "Tunneling for Deep Tunnels."

c. Prediction of Magnitude of Loading

The tunnel loading was predicted on Terzaghi's rock load as contained in "Rock Tunneling with Steel Supports" Proctor and White, 1968 Commercial Shearing Inc.

This assumption that Terzaghi's load is applicable for the major portion of this deep tunnel proved to be correct and was verified by the instrumentation programs during actual construction.

d. Reduction in Strength of Precast Segments

The mechanism of reduction of the strength of the segmented liner under nonuniform load, jack thrust, and circumferential forces was intentionally omitted from the subject paper because the limitations of space would not permit such discussions. Building Code Requirements for Reinforced Concrete (ACI 318-83), chapter 10 provides adequate information on the strength reduction possibilities of a member subjected to flexure and axial load and those were considered during the design of the precast segments.

The geotechnical analysis at the several test sections of the tunnel as referred to by Dr. Hendron has now been completed by the geotechnical consultant and can be obtained through the Bureau.

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