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Department of Transportation and Communications

THOROLD TUNNEL

Investigations to Determine the
Cause of Cracking in the Structure

ACRES

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March 15, 1972
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Mr. H. W. Adcock
Assistant Deputy Minister
Engineering & Operations
Department of Transportation
& Communications
Downsview 464, Ontario

Dear Mr. Adcock:

Thorold Tunnel
Investigations to Determine
the Cause of Cracking

We are pleased to submit herewith our report on the investigations to determine the cause of cracking in the Thorold Tunnel.

Our studies show that remedial measures are necessary and we would be pleased to discuss these with you at your early convenience.

Yours very truly,

S. Tibshirani
Vice President and
Manager of Operations

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1 - INTRODUCTION

1.1 - Terms of Reference

Cracking has been observed in the tunnel walls over a period of time, particularly in the outside walls of the tunnel beneath the west service building. Minor horizontal cracking in the walls was noted shortly after the west service building was completed in 1967, and the number and extent of open cracks at that location has increased steadily since. Following an inspection of the cracking at the west service building in May 1971, shown on Plate 1, the Department of Transportation and Communications instructed Acres, by letter dated June 15, 1971, to proceed with investigations to determine the cause of the cracking, and to photograph and map periodically cracked areas elsewhere in the tunnel. The results of the investigations are described in this report.

1.2 - Scope of the Investigations

The investigations had, as a primary objective, the determination of the cause of the cracking observed in the south wall of the tunnel beneath the west service building. In conjunction with this, an assessment was made of the effects of the cracking on the structural integrity of the west service building, and any implications regarding the safety of both west and east service buildings.

As a first step in the investigations the cracking was mapped in detail following which preliminry analyses were carried out to confirm the adequacy of the structure under design loadings, and to assist in developing hypotheses as to the possible cause of the cracking. The hypotheses were used to provide guidelines for the field investigations and installation of instrumentation which were commenced in August 1971.

Following completion of work in the field in November 1971, the final assessments of results and detailed structural analyses were carried out to determine the cause of cracking. This required a certain amount of laboratory testing to confirm parameters for use in the analyses, as well as continuing field observations to monitor structural performance.

A reasonable correlation has been achieved between structural analyses and observations made in the field, so that it has been possible to reach a conclusion concerning the cause of the cracking, and to determine the implications regarding future structural performance and safety.

The conclusions reached and resulting recommendations are presented in Section 2.

2 - CONCLUSIONS AND RECOMMENDATIONS

The investigations into cracking in the tunnel have resulted in the following conclusions:

(a) - Horizontal cracking is confined to outside walls of the tunnel, in areas where the gap between the tunnel structure and the face of the rock cut has been filled by concrete, that is, at each service building. The most severe cracking is in the south wall at the west service building.

(b) - The horizontal cracking is caused by a horizontal deformation imposed on the wall of the structure by rock squeeze, associated with relief of in situ stresses in the rock due to excavation for the tunnel. At the west service building an initial deformation was imposed on the tunnel wall by one particular stratum in the bedrock, namely, a shaly limestone layer of the Gasport formation lying at about elevation 530 feet. Deformations imposed on the west service building at the time of the field investigation result from rock squeeze occurring throughout the full depth of the rock cut.

(c) - Although available data on rock squeeze are limited, the indications are that, without adequate restraint, the imposed deformation on the service building structures will continue at a slowly decreasing rate over a period of many years.

(d) - At present stress levels the west service building structure is safe, and the load-carrying capability of the tunnel wall will not be impaired significantly for several years. However, the compressive stress in the tunnel roof slab at the west service building is very high and will increase progressively, rising to a peak each summer, due to thermal effects. This peak stress value cannot be predicted with accuracy and it may prove necessary, on the basis of continuing field observations, to undertake remedial work prior to the end of the summer of 1972 to ensure that the stress is maintained at a reasonable value.

(e) - Remedial work to be carried out at the west service building should be aimed at controlling or reducing the high compressive stress in the tunnel roof and eliminating additional imposed deformation on the tunnel wall. This remedial work is best accomplished by unloading the walls of the building, possibly utilizing slots cut in the rock north and south of the structure.

(f) - Deformation due to rock squeeze is probably occurring at the east service building, but it is considered to be not as far advanced as at the west service building. This should be verified by measurement of actual stresses in the structure.

Based on these conclusions, the following recommendations are presented:

(a) - A program of remedial work for the west service building should be prepared, possibly for implementation in the late summer of 1972, if measured deformations in the structure indicate that stresses will reach undesirable levels at that time.

(b) - The performance of the west service building should continue to be monitored, using the installed instrumentation to ensure the best possible prediction of future structural deformations, with possibly the addition of stress monitoring devices.

(c) - Stresses at key locations in the east service building should be measured as soon as possible, and instrumentation installed to monitor changes in deformations occurring in that structure.

3 - SITE CONDITIONS AND FIELD INVESTIGATIONS

3.1 - Tunnel Structure

Typical details of the tunnel and west service building structures are given on Plate 2 with the longitudinal profile through the tunnel on Plate 3. In general, the reinforced concrete tunnel was constructed in open cut with the structure backfilled with compacted rock fill between the concrete and the rock cut faces. At each service building, however, the rock fill is replaced by concrete bulkheads cast in place between the structure and the bedrock forming barriers to retain the water in the canal. Horizontal cracking in the structure is confined primarily to the areas with concrete bulkheads. The only other place at which the tunnel structure is connected monolithically with the bedrock is beneath the west canal wall, and no horizontal cracking is evident at this location.

Rock excavation in the vicinity of the west service building was completed in January 1966, with the tunnel section under the west service building constructed by July 2, 1966. The bulkheads were poured in two lifts, with the one to the north completed by August 25, 1966, and the one to the south by September 6, 1966. Backfilling around the west service building was completed to ground level by July 1967. The first minor horizontal cracking at the west service building was observed in September 1967. Further details of the construction sequence are described in Appendix A. JAN 66 7 MONTHS SEP 66 CRACKS SEP 67

Waterproofing at the service buildings comprises one layer of bentonite filled Volclay panels. At the start of construction of the west service building these panels were protected by 1-inch fibreboard, but after problems encountered with the fibreboard during the initial 29-foot pour in the north bulkhead, this protection was changed to polyethylene sheeting. Hence, there is a 1-inch layer of compressible material outside the north wall of the west service building and none outside the south wall. FIBREBOARD

3.2 - Geological Conditions

Geological conditions at the tunnel are shown on Plate 3. The tunnel is founded in rock of Middle Silurian age overlain by overburden varying in thickness between 8 and 30 feet. The overburden consists of silty clay of glacio-lacustrine origin over a thin layer of silty till. The upper 55 feet or so of the rock strata lay within the Lockport formation and are comprised of dolomite of the Goat Island Member, dolomitic limestone and shaly limestone of the Gasport Member, and dolomite of the Decew Member. Below the Lockport formation, the upper portion of the Rochester shale formation was exposed in the rock cut during construction. This formation consists of approximately 44 feet of calcareous shale with occasional thin limestone beds.

Recent experience by the United States Army Corps of Engineers, during investigations of the American Falls at Niagara, has confirmed the presence of locked-in stresses and therefore the likelihood of rock squeeze phenomena occurring in rock cuts in the Niagara Peninsula. Certain data on this were made available to Acres on an informal basis for use in the present investigations. Follow-up research into both published and unpublished information has confirmed that other data on rock squeeze are available, mainly from Ontario Hydro's observations on power plants at Niagara Falls. Data on rock squeeze are given in Appendix D.

Rock squeeze is defined as deformation resulting from removal of confining stresses on the rock and consequent relaxation due to redistribution of previously existing compressive stresses in the rock. This phenomenon may be generated by an excavation with elastic rebound at the time of excavation, followed by long-term creep. The elastic rebound will probably occur during a normal construction period, but the long-term rock squeeze phenomenon will generally continue after construction is complete. DEFINITION

Rock squeeze would impose a steadily increasing deformation on a structure placed against the rock, as at the west service building, until either equilibrium is reached with the restraining support from the structure, or until the rock squeeze ceases. Data indicate that the supporting force required to restrain rock squeeze is substantial due to the high in situ compressive stress level in the order of 1,000 psi.

Observations considered particularly relevant to the problem at Thorold pertain to the problems encountered over the years in the wheel pit at the Canadian Niagara Power Company in Niagara Falls. Rock squeeze at all levels of this 182-foot deep cut has increased steadily for more than
40 30 years, and has constituted a continuing structural problem with the total deformation associated with the shaly limestone of the Gasport Member being several times greater than that for the other rock strata. This shaly limestone unit correlates well with the shaly limestone encountered behind the walls of the west service building at Thorold.

3.3 - Field Investigations

The prime objective of the field investigations was the determination of the physical conditions of the tunnel wall, including the depth and extent of cracking, the obtaining of concrete and rock samples for visible inspection, uniaxial compression and material testing, and the examination of the concrete/bentonite and concrete/rock interfaces. Included in the program was the determination of stresses in the structure by means of an overcoring technique and an assessment of ground-water conditions in the surrounding rock.

The extent of the drilling and sampling program is shown on Plate 4. Detailed drill-hole logs and photographic records for the borehole television camera are included in Appendix E.

The results of the field investigation are as follows:

3.3.1 - Mapping of Cracking

The cracks mapped in the south wall of the tunnel under the west service building are shown in elevation on Plate 5. The trend of the cracking is essentially horizontal rather than parallel to the road surface and is confined to a relatively narrow band. Although attempts were made in the drilling program to follow individual cracks through the wall, they were unsuccessful. However, in a number of drill holes, cracks were intersected at depths up to 3 feet 6 inches from the face of the tunnel wall, and from these it was concluded that the cracked planes were within plus or minus 5 degrees of the horizontal.

To monitor variation in crack widths in the tunnel wall, a series of surface studs was installed and deformations measured by means of a Demec mechanical strain gauge. Results to date indicate that the crack widths are affected by atmospheric temperature changes in the tunnel, with daily variations indicating expansion and contraction of the surface of the tunnel wall, and a seasonal variation indicating that the cracks in the wall are tending to close up in the winter.

Relationships between deformations measured and temperature are discussed further in Section 7.

3.3.2 - Condition of the Concrete

The investigation was concentrated on the south wall of the tunnel at the west service building, with additional holes in the north wall at the west service building and in the north and south walls at the east service building. Many of the holes intersected reinforcement confirming that the steel is of the specified size. The spacing of the steel was confirmed to be as specified by means of a magnetic reinforcement locator.

Visual inspection of concrete cores taken from the tunnel walls, roadway, and tunnel roof strut in the area of the west service building and elsewhere in the tunnel reveals no sign of deterioration due to chemical or other action. There were signs of leaching in a number of the samples of bulkhead concrete. Conc. leaching

In the south wall of the tunnel at the west service building, cracks were intersected in a number of drill holes. A visual inspection of these holes, using the Ontario Hydro borehole television camera, confirmed the existence of cracks to a considerable depth. In the case of hole OC14 cracks were visible, using the above equipment, to a depth of 3 feet 6 inches from the face of the tunnel. The inclination of the crack was assessed to be rising at 5 degrees from the tunnel wall face toward the rock. Also in hole OC14, there was an indication of a crack plane dipping at 5 degrees away from the tunnel wall. This crack appeared tightly closed and could not be seen on the surface of the tunnel wall.

No open cracks were observed in other samples taken from the tunnel walls.

Some minor cracking of bulkhead concrete was observed at the south wall of the tunnel at the west service building, but no definite pattern could be established. One hole in the north bulkhead did intersect a near vertical crack in the north-south direction, and a white powdery deposit on the faces of the crack was probably precipitated during the passage of water.

Concrete cores obtained from the tunnel roof strut exhibited no sign of cracking, although it was noted that the cores separated at both top and bottom layers of reinforcement, indicating at least a lack of bond and possibly the existence of a high compressive load in the reinforcement. Roof

In general, more cracking was noted in cores and drill holes in bulkhead concrete than in those from the tunnel structure. Signs of leaching were also noted in bulkhead concrete but were not evident in the structural concrete.

3.3.3 - Detailed Geology at West Service Building

Rock conditions behind the south wall of the west service building were examined in detail, using core from the diamond drill holes and the borehole television camera rented from Ontario Hydro. Stratigraphical information obtained prior to and during construction of the tunnel was confirmed, as shown on Plate 4. Rock close to the excavation face exhibited more intense shattering than at depth in the drill holes. No unusual jointing or bedding features were noted.

The rock consists of strata of dolomite, limestone, and shale. Particular attention was paid to the shaly limestone stratum in the Gasport Member of the Lockport formation. Within this stratum the bedding structure of the shaly layers can be observed, although individual layers are not well defined. The drill cores from this stratum did not display tendency to decompose upon wetting and drying. Calcareous inclusions are frequent, indicating a correlation with the observed high calcium content of the ground water.

Detailed examination of the cores in the shaly limestone stratum revealed the following:

(a) - Development of cleavage usually occurs normal to the maximum axis of compression. The orientation of the cleavage in this rock is approximately horizontal, indicating that during the relevant geological period maximum compression was applied in the vertical direction.

(b) - There is visual evidence of distortion of fossils in this stratum, suggesting a strain condition and that the rock has undergone lateral flow during its geological history. Orientation of these fossils indicates that the major compressive forces caused flow along a north-south direction. ~~It is felt that this~~ interpretation provides evidence that would suggest the possible occurrence of a relatively high in situ stress condition.

(c) - Minor drag folds also suggest that the rock may have been subjected to pressures both in the east-west and in the north-south directions.

It is not possible to confirm from the visual assessment of rock conditions that any unusual stress or rock squeeze conditions exist at the tunnel. However, there are indications that the shaly limestone stratum in particular has been subject to high stress conditions during its geological history, and that residual stresses were locked into the rock, resulting in a high in situ stress at present. This correlates well with the observed rock squeeze in the wheel pit at the Canadian Niagara Power Company in Niagara Falls.

3.3.4 - Tunnel Wall Concrete Bulkhead Interface

This interface was examined in the recovered core and also by direct inspection, using the Ontario Hydro borehole television camera. The photographic records of these observations are included in Appendix E.

Examination of the drill cores indicated that, in most holes, the findings were consistent with construction details described in Section 3.1. No significant voids were noted, and in most cases complete discs of the bentonite panels from the interface were recovered, together with the protective plastic sheeting. The plastic shows in the photographs as a black band. The in situ thickness of the bentonite panels was measured and ranged from 0.18 to 0.24 inch, compared with a nominal thickness of 0.2 inch for the original panels in the dry condition.

Minor amounts of precipitated carbonate were noted in the interface area in some holes, probably indicating a minor movement of water.

Other holes were drilled in each of the service building walls through this interface, and all but one recovered bentonite panel and plastic. In the north wall at the west service building, fibreboard panelling was recovered from behind the bentonite, that is, between the bentonite and the bulkhead concrete. A second hole, drilled to the east in the same vicinity, did not, however, recover the fibreboard, and there appeared to be no protection for the bentonite. These findings were consistent with the construction records as described in Section 3.1. NORTH WALL

In general the interface is tight with no voids, and any movement of water considered to be minor. No significant swelling of bentonite could be noted.

3.3.5 - Concrete Bulkhead/
Rock Interface

Rock core recovered from the area immediately adjacent to the bulkhead concrete was heavily fractured, probably as a result of the excavation blasting. Inspection of the interface by the television camera gave no indication of infilling material, or of a continuous gap at the concrete/rock interface. In general, it appeared these were in intimate contact, although there were some local voids with occasionally some evidence of precipitated carbonate. Little segregation of the concrete constituents at the interface, due to the action of water, was evident, indicating successful control of seepage water during construction.

In a number of holes, plastic sheet was found between the rock and the bulkhead concrete. This sheeting was placed during construction to aid in control of water and was apparently left in place during concreting of the bulkhead.

3.3.6 - Stress Measurements

To evaluate the stresses currently existing in the tunnel structure, an overcoring technique was used. This is described in detail in Appendix A, together with a description of the results. Stress values resulting from the overcoring tests are shown on Plate 7. In reviewing the test results it was noted that the cold weather during the inspection period (October 1971) may have reduced the magnitude of the compressive stresses in the tunnel roof strut, due to thermal contraction.

Two test holes in the south wall of the tunnel at the west service building indicated that, despite possible local anomalies due to inaccuracies inherent in using the overcoring method on cracked concrete, vertical stresses in the wall vary from compression at the rear face to tension at the front face. These results are consistent with bending of the wall under an external horizontal load. Axial stresses in the tunnel floor and roof are consistent with the performance of these members as struts, supporting a side load applied to the tunnel wall.

Compressive stresses measured in the wall and tunnel floor are reasonably low,but those in the tunnel roof strut, being of the order of 2,500 psi, are considerably higher than those allowed by the design criteria. Laboratory tests described in Section 4 show that the concrete is of a high strength, such that even with the high measured stress, the factor of safety based on the ultimate strength of the concrete is greater than two. Nevertheless, in view of the anticipated increase in the roof strut stress, it is considered to be unacceptably high.

In view of the adequate results obtained from the overcoring tests, no attempt was made to measure stress in reinforcing steel as the results would not add significantly to the knowledge already obtained, and cutting of any more reinforcement than absolutely necessary was considered undesirable.

3.3.7 - Piezometric Levels and Water Inflow

A number of holes were drilled in the south wall of the tunnel at the west service building to determine the piezometer levels and associated flows of ground water at the interfaces between structure and bulkhead, and between bulkhead and rock. Piezometric levels resulting from this investigation are shown on Plate 4. Results of the investigation are as follows:

(a) - Nowhere in the holes drilled did the piezometric levels exceed the hydrostatic pressures equivalent to canal water level for which the tunnel was designed.

(b) - At both the bentonite panel/tunnel wall and the concrete bulkhead/rock interfaces, piezometric levels decrease approximately linearly from east to west, indicating a normal flow pattern for the ground water. The piezometric level in the rock, as determined in hole P1, elevation 564.5 feet, is in agreement with that observed during construction, prior to the blanket grouting.

(c) - During drilling of some of the holes it was noted that seepage from the visible cracks increased. As the drill water pressure was initially as high as 200 psi, it was concluded that this water was connecting to the crack planes within the tunnel wall. The increased seepage ceased as soon as the drilling stopped.

(d) - Measured flows from the holes indicated that no major water-carrying void was intersected.

4 - LABORATORY TESTING

4.1 - Concrete

During the field investigations concrete cores were recovered and subsequently tested for uniaxial compressive strength and elastic parameters. The results of these are given in Table I and show that the strength is now considerably in excess of that originally specified, being approximately 6,000 psi for the tunnel walls. Results of control testing during construction are summarized in Appendix B.

During testing the samples were instrumented to determine values of Young's Modulus (E) and Poisson's ratio. These were found to be approximately 5.4×10^6 psi and 0.22, respectively.

Logging of concrete cores taken from the tunnel walls, bulkhead, roadway and service building floor was carried out in the laboratory. No sign of significant deterioration due to chemical or other action could be detected.

The results of chemical analyses performed on concrete core samples are included in Appendix B. The cement content was determined to be commensurate with that specified in the mix design. The measured content of aluminate and magnesium, 6 to 8 per cent, was slightly higher than that which would normally have been expected, 5 per cent. These constituents, if present in excess, could cause the generation of sufficient heat during hydration of the cement to cause internal micro-cracking. However, as no indication of such cracking could be detected, it was concluded that the chemical composition had not significantly affected the concrete strength or other structural properties.

4.2 - Ground Water and Associated Deposits

4.2.1 - Ground Water

Samples of the ground water were analyzed as part of the original geotechnical investigation for the tunnel. These were confirmed by results of subsequent analyses during the present investigation, which are given in Appendix B, together with those from further analyses performed on samples taken from drill holes in the area of the west service building. The results of the

$\frac{\#}{ft^2}$ 2.8 x 2000 $\frac{\#}{ton}$ x $\frac{1\ ft^2}{144\ in^2}$

= $\frac{5600}{144}$ psi = $\underline{45-50\ psi}$

analyses are consistent with the lithology of the rock surrounding the tunnel, namely limestone (calcium carbonate) and dolomite (magnesium carbonate).

4.2.2 - Deposits

Deposits laid down as white powder in the crack intersected in the north bulkhead were analyzed for chemical composition. The predominant constituents were calcium and magnesium oxides with a yellow streak of iron oxide at the fibreboard interface.

A white precipitate obtained in drill holes in the rock behind the south bulkhead contained calcium and magnesium carbonates. These compounds are consistent with the lithology of the rock surrounding the tunnel, being principally limestone (calcium carbonate) and dolomite (magnesium carbonate).

The major conclusion derived from these analyses is that the ground water does not contain material which is likely to be harmful to hardened concrete. However, it is possible that some of the ground water may have been mixed with the fresh concrete, thereby increasing slightly the magnesium and carbonate content of the concrete.

4.3 - Bentonite Panels

A number of laboratory tests were carried out on samples of bentonite panels, as used to waterproof the tunnel, to confirm swelling pressures under various conditions for use in structural analyses. The results of these tests are described in detail in Appendix B.

It was concluded that active pressure exerted by bentonite swelling due to action of water is a maximum of 2.8 tons per sq ft, and that passive pressure to cause bentonite to consolidate is a maximum of 6 tons per sq ft.

5 - STRUCTURAL INVESTIGATIONS

It can be concluded from the results of the field and laboratory investigations that the cracking of the tunnel wall, at the west service building particularly, cannot be attributed to defects in materials or poor construction. Hence, the cracking must be caused by loadings or deformations applied to the structure. This is confirmed by the stresses measured in the west service building structure, particularly the very high stress in the tunnel roof strut. The structural investigations were aimed at confirming the acceptability of the design of the west service building, and determining, by means of analyses, the probable loading configuration and method of deformation for the structure.

5.1 - Assessment of Original Design

The as-built structure at the west service building was analyzed using the original design loads applied to a representative two-dimensional framework, based on the service building cross section. Loading conditions and results of the analysis are described in detail in Appendix C and the calculated stresses in Table II.

Stress levels within the major members of the framework proved to be less than the maximum allowable design stresses. On this basis it is concluded that design of the building is adequate for the loads assumed at the design stage, but that the building has been exposed to more severe loadings than anticipated.

Inspection of the pattern of cracking and the measured stress distribution indicates that the walls of the structure must be subjected to a severe external loading, causing bending in the wall and resulting in high axial loads in the floor, and particularly the roof of the tunnel. No other loading configuration could be developed to fit the observed conditions.

5.2 - Loading Hypotheses

Consideration was given to a number of different loading hypotheses to account for the formation of cracks in the wall, as observed. These hypotheses are as follows:

5.2.1 - Freezing of the Ground Water Behind the Wall

This hypothesis was based on the assumption that freezing might occur outside the wall of the tunnel, resulting in expansion of the free water at either the tunnel wall/bulkhead, or the bulkhead/rock interfaces, or both, causing a pressure against the structure. In fact, temperature observations during the winter 1971-72, using thermocouples installed in drill holes through the tunnel wall, indicate that the temperature behind the wall is not affected significantly by atmospheric temperature variations in the tunnel, and hence freezing does not occur.

It was concluded that freezing of ground water was not creating a load on the wall.

5.2.2 - Active Pressure by Bentonite

In this case, the bentonite waterproofing outside the tunnel wall is assumed to be exposed to sufficient water to create a swelling pressure. Because the bentonite is confined between the tunnel and bulkhead, an active pressure will be applied to the tunnel walls. This loading condition was analyzed in the structural investigations.

2.8.T.S.f

5.2.3 - Passive Pressure by Bentonite

This loading hypothesis is also dependent on the swelling properties of the bentonite, but assumes the development of a passive rather than an active loading condition. The passive condition would be created by thermal contraction of the tunnel roof strut in the winter, which would allow the walls to move away from the bulkheads so that the bentonite can swell and fill the gap, assuming that sufficient water is available. When the temperature rises again, expansion of the tunnel is resisted by the expanded bentonite, causing a passive load on the walls. To prove if this was the loading condition which caused the initial cracking of the wall, analyses were carried out, based on data for bentonite swelling pressures obtained in the laboratory.

6.T.S.f.

5.2.4 - Wedging Action

In the event that thermal contraction and expansion of the west service building structure occurs due to seasonal temperature changes in the tunnel roof strut, a small gap could form at either of the interfaces. Deposition in this gap of suspended material, or of material precipitated from the ground water, might restrict expansion of the structure as the temperature rises, causing a passive load condition on the outer walls. No evidence of a gap filled with compressed deposits was found in the drill core or with the bore-hole television camera. Hence, it was concluded that the wedging action, as described, had not occurred and that this loading condition need not be considered further.

5.2.5 - Rock Squeeze

Consideration of the effect of rock squeeze on the west service building indicated that almost any size loading could result, depending on the amount and rate of increase of deformation and the degree of flexibility of the structure. Since the pattern of cracking suggests a horizontal line or narrow band loading, analyses were carried out to incorporate loads with a similar configuration. The size of the loads was calculated on the basis of the estimated deformation of the wall under such a line load.

5.3 - Analyses of Possible Loading Systems

In addition to analyses of the west service building frame-work for the loads described in Section 5.2, temporary construction loads were also analyzed to determine if a possible locked-in stress condition could account for the observed structural problems. Results of the analyses are shown in Plate 6 with calculated stresses in key members in Table III.

The best correlation, with measured stresses in the tunnel south wall and floor, was obtained with a line load at about the level of the shaly limestone layer in the Gasport Member, namely, elevation 532 feet. All other loads considered resulted in stresses too low to be correlated with the measured stresses. It proved impossible to obtain any correlation with the high measured stress in the tunnel roof strut. It was concluded that this stress must result from a thrust applied direct to the strut, probably due to rock squeeze. The loading condition created by rock squeeze was examined in greater detail.

5.4 - Analyses of Rock Squeeze Loading Condition

Based on the assumption that the cracking in the wall was caused initially by rock squeeze occurring predominantly in the layer of shaly limestone at about elevation 532 feet, a number of framework analyses were carried out to determine if it is possible to have a line load, at or above elevation 532 feet, which would reproduce the measured stresses. The various analyses were then modified by addition of a direct thrust in the tunnel roof strut to reproduce seasonal thermal effects in the strut.

The resulting stresses in the tunnel roof strut were still too low, and hence it was concluded that rock squeeze at about strut level is resulting in direct compression in the strut. This was confirmed by field observations which show that the length of the strut is not varying with changes in temperature, indicating that it is, in fact, prestressed to a stress higher than the maximum due to thermal effects. Hence, the load in the strut is the sum of the loads due to rock squeeze acting directly at roof strut level and from the shaly limestone layer on the tunnel wall, together with thermal effects and backfill to the structure.

EXPLANATION OF HIGH STRUT STRESSES.

It is concluded that, due to the nature of the rock squeeze phenomenon, the load in the strut is increasing steadily and will do so for many years.

The calculated deformations in the west service building structure under the probable loading conditions are shown on Plate 7. A detailed description of the derivation of the probable loading condition is given in Appendix C. The effect of the increase in rock squeeze deformation with time is based on the data obtained from Ontario Hydro, as described in Appendix D, and calculated deformation to date.

During the analysis of the framework under the rock squeeze load, it was realized that the cracked portion of tunnel wall indicates the development of structural hinges in the wall. Since the rock squeeze load is in fact an imposed deformation, this is causing an increasing rotation at these hinges, with an associated increase in strain in the reinforcing steel.

At present the reinforcing steel has yielded at the cracks, although the strain is considerably less than that associated with ultimate load. Hence, the wall will withstand a substantial increase in imposed deformation. Piezometric observations indicate that the tunnel wall is already subjected to full hydrostatic loading, in addition to the imposed deformation due to rock squeeze.

No theoretical justification could be found to explain why the north wall of the tunnel at the west service building has not yet cracked as badly as the south wall. It is concluded that the effect of the imposed deformation, due to rock squeeze, has been delayed by the compressibility of the 1-inch fibreboard placed behind the north wall. This 1-inch fibreboard, when recovered from the north wall, had an approximate thickness of 0.75 inch, indicating that a significant compression of the fibreboard has in fact occurred.

6 - CAUSE OF CRACKING AND ASSESSMENT OF STRUCTURAL SAFETY

6.1 - Cause of Cracking

The field investigations have confirmed that the tunnel structure was constructed with only one significant variation from design specifications, namely, the elimination of the 1-inch fibreboard protection for bentonite from bulkhead areas after completion of the lowest pour in the north bulkhead at the west service building. Where the fibreboard was used, it appears to have only delayed imposed deformation due to rock squeeze. Concrete quality was proved to be good and cannot be considered as contributing to the observed cracking.

Review of design analyses for possible loading and stress conditions and examination of the cracking configuration all indicate that the cracking in the walls at the west service building is caused by a severe external loading on the wall of the tunnel. This is confirmed by the field measurements of stress in the roof and floor of the tunnel in the south tube. These stresses indicate that the side loading on the tunnel wall is considerably higher than that considered in the design.

Review of the design loads indicates that reasonable assumptions were made, consistent with normal anticipated conditions. However, these normal conditions cannot provide the severe side loading which obviously exists. Investigations into the swelling pressures of bentonite and the forces exerted under both active and passive conditions show that the bentonite is not capable of creating the loads required to crack the structure. This is confirmed by the apparent compression of the bentonite panel and relatively low moisture content in the bentonite observed in drill hole OC14.

The pattern of cracking at the west service building is significant as the cracks tend to lie in a horizontal plane rather than parallel to the road surface, and they appear to be confined to a fairly narrow band along the wall. This pattern is not consistent with that caused by any kind of uniform or triangular loading on the wall. In fact, the only loading mechanism that could be found to develop, even approximately, the cracking pattern observed is that of a horizontal line or narrow band load applied to the outside wall of the tunnel at about elevation 532 feet. This corresponds closely with the stratum of shaly limestone in the Gasport Member.

Although it has been concluded that the cracking of the wall was caused initially by rock squeeze, predominantly in the shaly limestone stratum, the measured stresses in the tunnel roof strut can only be explained by rock squeeze acting at the level of the strut. It must then be concluded that rock squeeze is occurring to a greater or lesser degree in all strata at the tunnel. New vertical cracks in the south tunnel wall at roof strut level, first observed in February 1972, confirm that a substantial loading is acting at that level.

The fact that the south wall of the tunnel at the west service building is worse than the north is difficult to explain, unless it is assumed that the 1-inch thick fibreboard applied to the north wall has taken up a certain amount of the imposed deformation that has occurred to date in the rock, thereby delaying application of the full rock load to the tunnel structure.

6.2 - Assessment of Structural Safety

It is apparent that the south wall of the tunnel at the west service building has been strained excessively and that the structure is no longer performing as designed. In fact, the previously rigid wall is developing a structural hinge at the zone of cracking. This hinge will allow a substantial additional amount of deformation to occur without impairing the load-carrying ability of the wall. The rate of increase in imposed deformation due to rock squeeze is slow, so that the load-carrying ability of the wall, even without remedial work, will not be affected significantly for some years. However, based on available data on rock squeeze, the imposed deformation will increase steadily for many years and, eventually, remedial measures to restore the wall will be required. It is impossible to establish the total amount of deformation to be expected, although data on rock squeeze in similar strata under an unrestrained condition show that as much as 4-1/2 inches of total movement have been observed.

The effect of rock squeeze on the axial compressive stress in the tunnel roof strut at the west service building creates a far more serious problem than the cracking in the tunnel wall. The stress of about 2,500 psi, measured in this strut in October 1971, is already excessively high, even though the concrete has a measured ultimate compressive strength of more than 6,000 psi. As the imposed deformation due to rock squeeze increases over the years, the stress in the strut will increase accordingly, and stresses in the remainder of the tunnel roof will be comparable. These stresses will all be at their maximum values in the summer due to seasonal thermal effects.

In view of the new cracks observed in February 1972 in the wall of the west service building at the tunnel roof level, continued deterioration of the stress condition in the tunnel roof slab must be assumed. It is concluded that the compressive stress in the tunnel roof slab at the west service building will reach a seasonal peak value in the summer of 1972, and if continuing field observations prior to then confirm that the stress will reach an unacceptable level, then remedial work to relieve the stresses will be required accordingly.

Based on a visual assessment, any deterioration of the structure at the east service building is minor. However, ~~the layer of shaly limestone which caused the~~ initial deformation at the west service building is at floor level in the east service building. Hence, the imposed deformation due to rock squeeze at the east service building is probably such that an unacceptable increase in stress in the tunnel roof or floor may occur without being accompanied by a corresponding cracking of the wall. It is concluded that stress conditions in the east service building should be measured to confirm if stresses are increasing above design levels.

EAST BUILDING

7 - CONTINUING OBSERVATIONS

To determine the long-term trend for behavior of the west service building structure under the imposed loading condition, instrumentation has been installed which will allow continuing observations to be made. These will monitor the relative movement and deformations of components of the structural system at the west service building, as well as changes in temperature and piezometric levels. Observations to date are plotted on Plate 8.

7.1 - Crack Movements

The crack movements are being measured by use of a Demec gauge. This is a mechanical strain gauge which measures deformation to an accuracy of approximately 1×10^{-5} in/in between metallic studs fixed at a standard 8-inch gauge length. The studs were positioned across the cracks and joints in the tunnel wall as shown on Plate 5.

Observations to date indicate that the cracks opened at the surface as the temperature decreased. After an extended cool period the cracks closed again, presumably due to a more uniform thermal distribution in the wall. Observations on the tunnel roof strut show that no significant movement has occurred, indicating that the strut is precompressed by a load greater than that due to restraint of thermal expansion.

7.2 - Deformations

Borehole extensometers have been positioned through the tunnel wall and anchored at various depths in the rock. These will allow measurement of any relative movements between the tunnel, the bulkhead, and the rock. In addition, an extensometer has been installed across the roof of the south tube to enable overall changes in width to be monitored.

Observations over a period from December 1971 to February 1972 indicate that a slight compression has occurred between the tunnel wall and the rock.

7.3 - Temperature

Thermocouples have been installed at varying depths in two positions in the cracked area of the south tube and in the tunnel roof slab at the west service building. Results to date show that freezing has not occurred at the outside face of the tunnel structure. It is confirmed that the temperature in the tunnel roof slab does not vary significantly from atmospheric temperature.

7.4 - Piezometric Levels

Piezometers installed under the present program will be left in place and will allow monitoring of the piezometric levels and quantities of flow in all but severe winter conditions.

Piezometer readings to date indicate no unusual or unpredictable water pressures outside the tunnel.

7.5 - East Service Building

No instrumentation has been installed at the east service building under the present investigations. It now appears likely, however, that rock squeeze deformations are probably developing at the east service building, possibly at a slower rate of increase than at the west service building.

In view of this it is concluded that at least a minimum amount of instrumentation should be installed at the east service building, to provide data on increasing structural deformations.

8 - REMEDIAL WORKS

The prime objective of proposed remedial works must be to reduce the stress in the tunnel roof slab at the west service building to an acceptable permanent level. Having achieved that objective, however, it will also be necessary to eliminate the effect on the tunnel wall of any further imposed deformation due to rock squeeze.

After consideration of various alternatives, it was concluded that both objectives can only be met by physically isolating the wall of the west service building from the surrounding rock. This can best be accomplished by means of a slot cut in the rock north and south of the service building. This solution is confirmed to be feasible, but it has not yet been investigated in detail to determine the cost or the best method of cutting the slot.

In view of the high stresses measured in the west service building, it may be necessary to carry out the remedial works before the peak temperatures in summer 1972. This need can only be evaluated positively on the basis of trends detected in continuing observations in spring 1972.

The need and the timing for any remedial works at the east service building can be determined only after stresses are measured in that structure.

Table I

TESTS ON CONCRETE

Sample Location	Type of Test	Ultimate Strength psi	Modulus of Elasticity psi	Poisson's Ratio
West Service Building Tunnel Roof Structure	Uniaxial Compression 6-inch sample	6,050*	4.83×10^6	-
West Service Building Tunnel Roof Structure	Impact Test	6,050*	-	-
West Service Building Tunnel Wall South Tube	Uniaxial Compression NX Sample	7,450*	6×10^6	0.22
West Service Building Tunnel Wall South Tube	Impact Test	5,050 to 7,450*	-	-
West Service Building Tunnel Wall North Tube	Impact Test	5,700 to 7,450	-	-
West Service Building Centre Wall in Tunnel	Impact Test	6,300 to 7,450	-	-
Station 68 + 23 Tunnel Wall	Impact Test	5,400 to 7,450	-	-
East Service Building Tunnel Wall	Impact Test	5,050 to 6,700	-	-

*Used to calibrate Impact Tester.

Table II

MAXIMUM STRESSES WEST SERVICE BUILDING
STATION 54 + 65 - ORIGINAL DESIGN LOADS

Load Description	Maximum Stress Roof El 549.8 ft	Maximum Stress Floor	Maximum Stress South Wall
1 - Symmetrical Bulkhead Load	-225	-247	+261
Maximum Equivalent Fluid Pressure	-389	+ 77	-385
2 - Self Weight Load Plus Dead Load	-197	-151	+ 63
Plus Final Earth Loading	-142	+ 70	-182
3 - Active Bentonite Load (2.8 tons/	-687	-431	+245
sq ft) Plus Dead Load Plus Final	-632	+234	-376
Earth Loading			

Stresses in psi: + = tension, - = compression.

For roof and floor, stresses are (top) / (bottom) of member.

For south wall, stresses are (inside) / (outside) surfaces of the tunnel wall.

Table III

MAXIMUM STRESSES WEST SERVICE BUILDING
STATION 54 + 65 - POSSIBLE LOADS

Load Description	Maximum Stress Roof El 549.8 ft	Maximum Stress Floor	Maximum Stress South Wall
1 - Unsymmetrical Bulkhead Load Allowing for Rate of Pour	+ 19 - 54	+120 -105	+ 26 -155
2 - Symmetrical Bulkhead Load Allowing for Rate of Pour	- 91 - 12	- 35 + 17	- 42 - 83
3 - Symmetrical Passive Bentonite Load (6 tons/sq ft) Plus Dead Load Plus Final Earth Loading	-1,414 -1,414	-934 +508	+602 -749
4 - Self Weight Plus Dead Load Plus Final Earth Loading Plus Maximum Point Load at El 532 feet	- 387 - 498	-748 +455	+810* -929

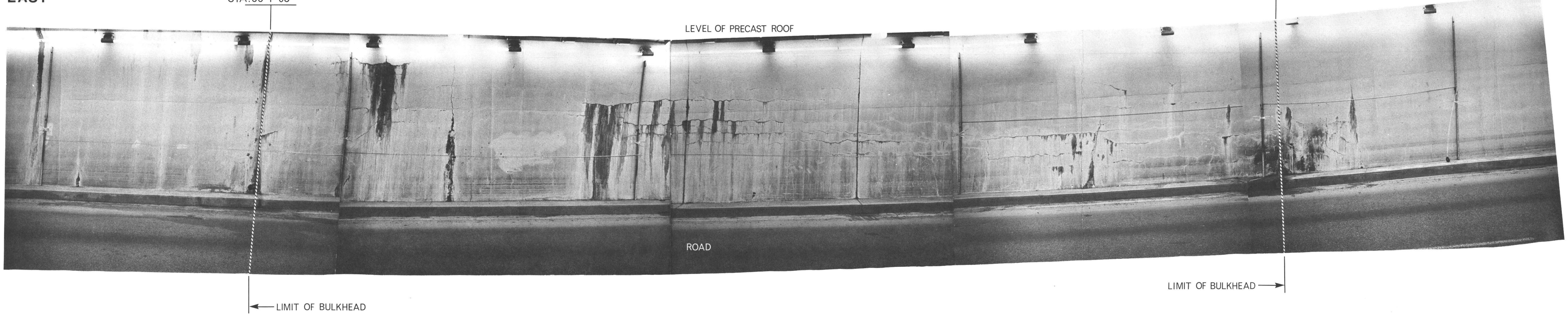
Stresses in psi: + = tension, - = compression.

For roof and floor, stresses are (top) (bottom) of member.

For south wall, stresses are (inside) (outside) surfaces of the tunnel wall.

*Limiting tensile stress at which cracking will occur.

PLATES



PHOTOGRAPHED MAY, 1971

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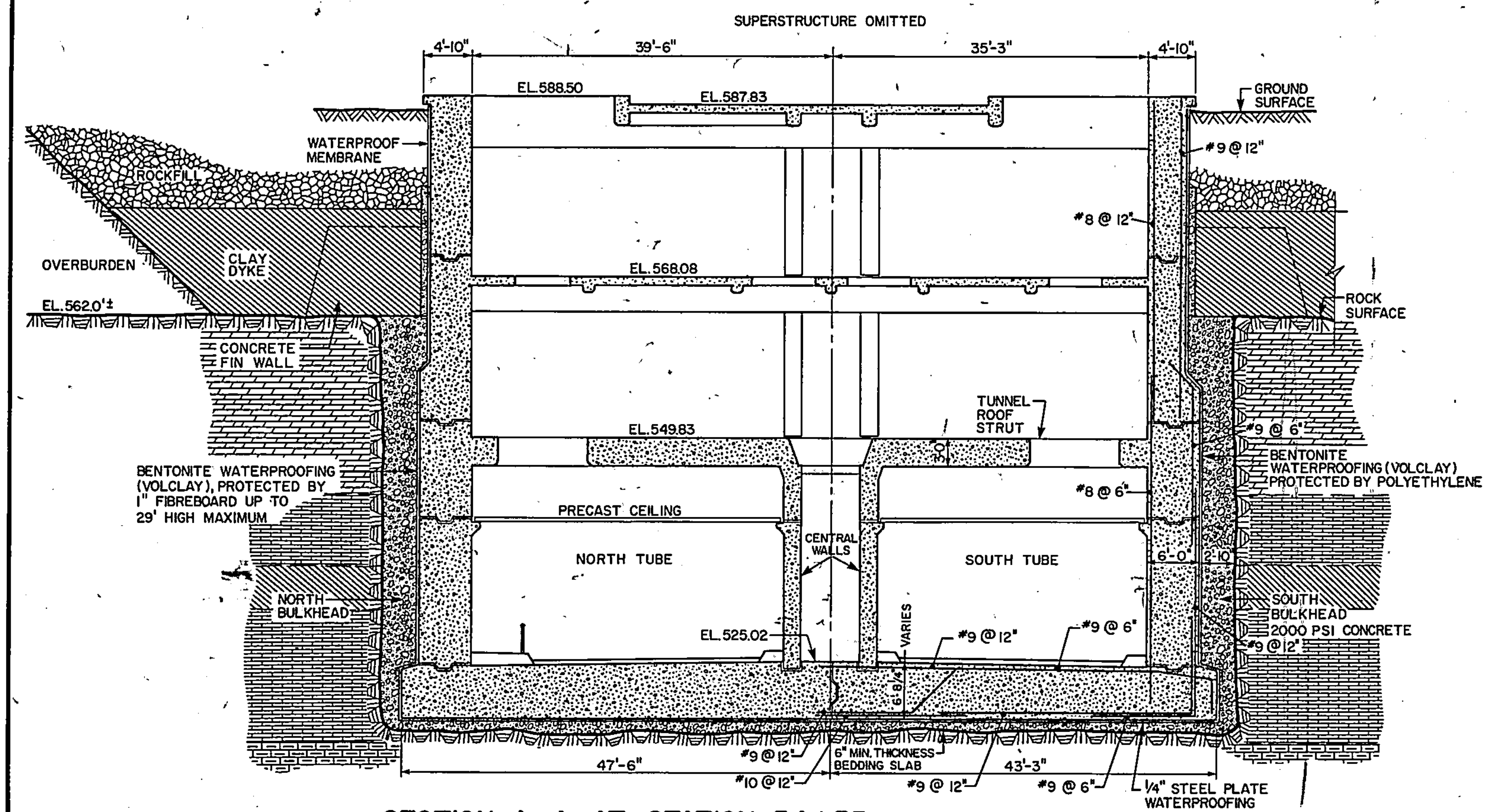
THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING
VIEW OF
SOUTH TUNNEL WALL

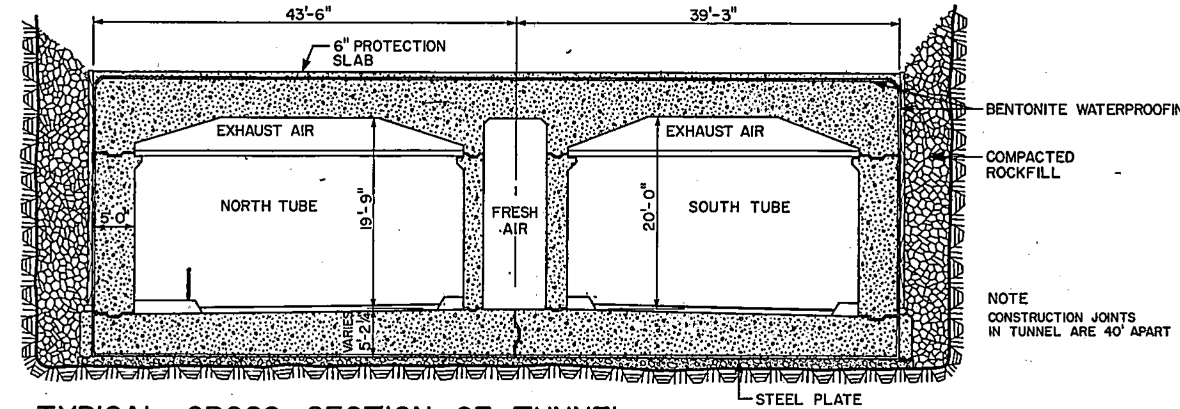
ACRES CONSULTING SERVICES LIMITED

FEBRUARY 1972

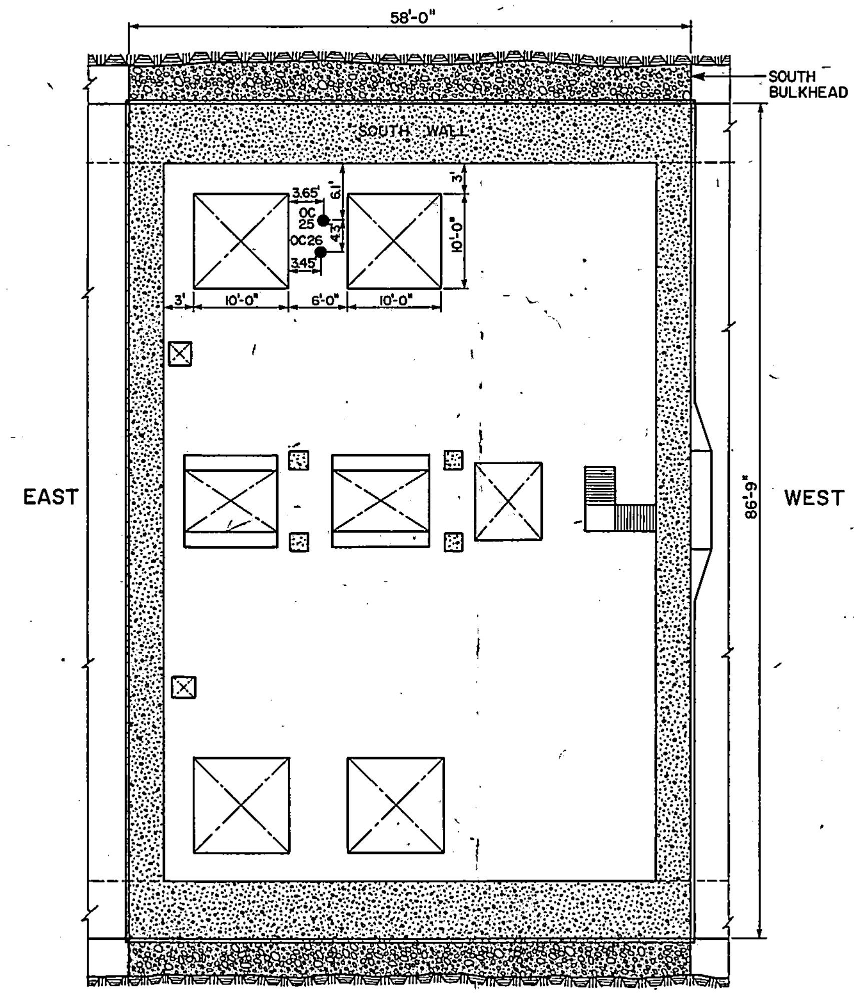
PLATE 1



SECTION A-A AT STATION 54+55

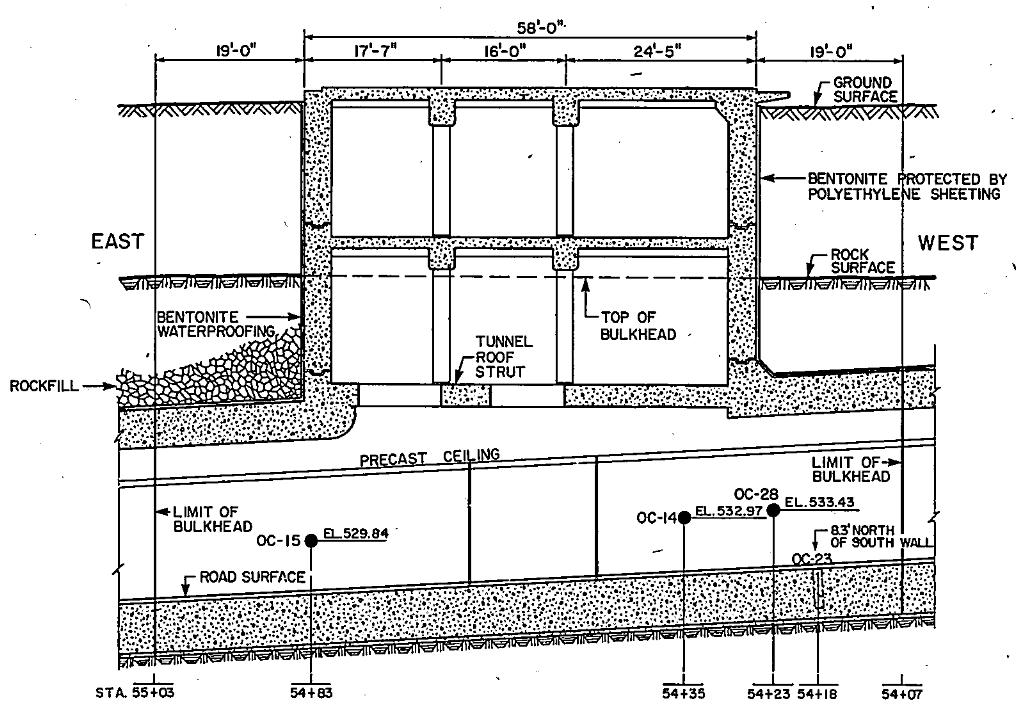


TYPICAL CROSS SECTION OF TUNNEL



PLAN AT EL.549.83

A



SOUTH TUNNEL WALL - ELEVATION

NOTES

OC-14 AND OC-15 OVERCORING TESTS IN SOUTH WALL.
OC-23 OVERCORING TESTS IN ROAD SLAB SOUTH TUBE.
OC-28 OVERCORING TESTS IN NORTH WALL.
OC-25 AND OC-26 OVERCORING TESTS IN TUNNEL ROOF STRUT

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WEST SERVICE BUILDING GENERAL ARRANGEMENT	
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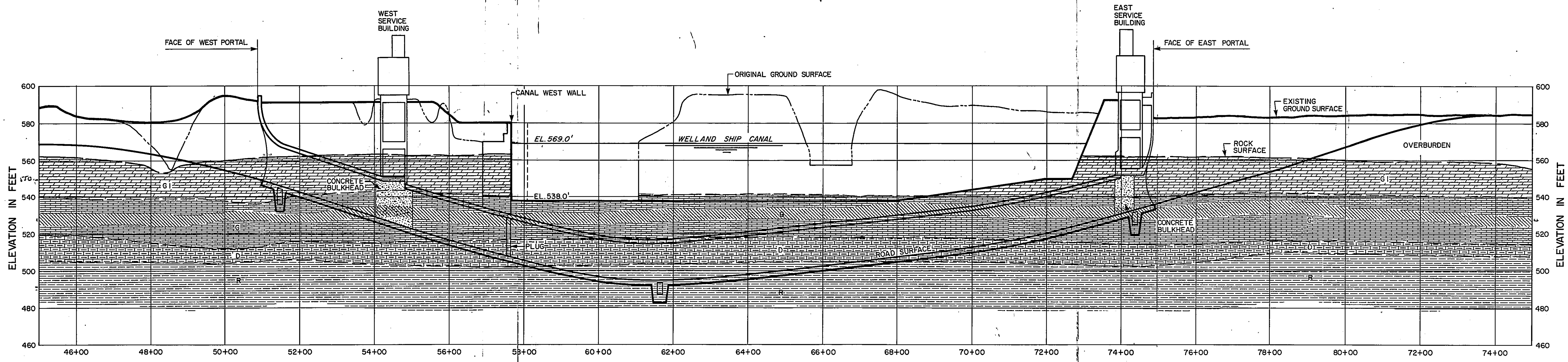
.01
3.1416
360

.01



GEOLOGICAL PROFILE ALONG TUNNEL CENTRE LINE

LEGEND

MIDDLE SILURIAN PERIOD BEDROCK

YOUNGEST	GI	GOAT ISLAND MEMBER (DOLOMITE WITH SOME DOLOMITIC LIMESTONE)
LOCKPORT FORMATION	G	GASPORT MEMBER (DOLOMITIC LIMESTONE WITH SHALE)
	D	DECEW MEMBER (DOLOMITE WITH SOME DOLOMITIC LIMESTONE)
OLDEST	R	ROCHESTER FORMATION (CALCAREOUS SHALE WITH THIN LIMESTONE BEDS)
		SHALY LIMESTONE LAYER IN GASPORT MEMBER PROVIDING INITIAL LOAD ON WEST SERVICE BUILDING

NOTES

1. GEOLOGICAL BOUNDARIES ARE INTERPRETED AS BEING THOSE EXISTING ALONG THE CENTRE LINE.
2. AVERAGE DIP OF BEDDING PLANES 35 FT./MILE TO THE SOUTH.

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THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

TUNNEL AREA
GEOLOGICAL PROFILE

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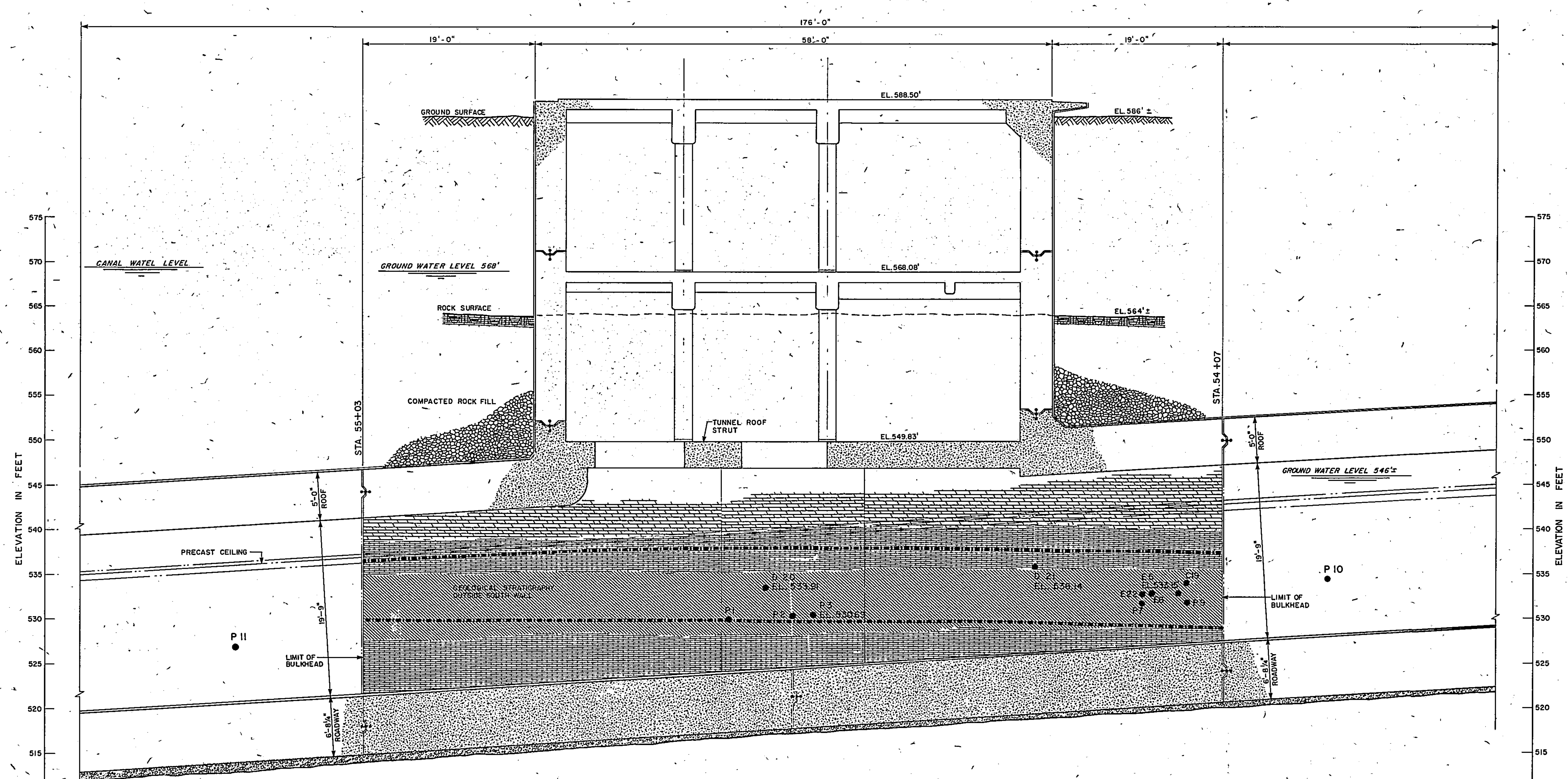
PLATE 3



ELEVATION OF TUNNEL SOUTH WALL

PIEZOMETER	GROUND WATER LEVEL (FT)		FLOW (G/MIN)		INTERFACE MONITORED	STATION	ELEVATION
	27/10/71	13/12/71	27/10/71	13/12/71			
P1	564.75	564.55	0.13	0.13	BENTONITE	54+62	530.15
P2	559.77	0	0.10	0	BENTONITE	54+55	530.57
P3	564.19	565.09	3.3	2.5	ROCK	54+53	530.69
P7	N. T.	553.65	N. T.	2.0	ROCK	54+16	531.95
P8	0	0	0	0	5'-5" INTO WALL	54+13	532.15
P9	553.46	550.66	N. T.	0.05±	BENTONITE	54+11	532.26
P10	547.06	545.66	31.8	3 0	ROCKFILL	53+96	534.16
P11	568.80	568.00	N.T.	7.0	ROCKFILL	55+17	527.0

N.T. - READING NOT TAKEN
CANAL WATER EL.569'±

LEGEND

- GOAT ISLAND DOLOMITE WITH SOME DOLOMITIC LIMESTONE
- GASPORT LIMESTONE (SLIGHTLY DOLOMITIC)
- GASPORT SHALY LIMESTONE (SLIGHTLY DOLOMITIC)
- APPROXIMATE BOUNDARY OF ZONE OF CRACKED CONCRETE

NOTE

ELEVATIONS FOR DRILL HOLES ARE AT INSIDE FACE OF TUNNEL WALL

INVESTIGATIONS CARRIED OUT IN OCTOBER AND NOVEMBER 1971

HOLE NOTATION:
- E - EXTENSOMETER INSTALLATION
- P - PIEZOMETRIC MEASUREMENT
- D - ROCK EXAMINATION
- C - CRACK EXAMINATION

OTHER HOLE LOCATIONS
- B29 - WEST SERVICE BUILDING, NORTH TUNNEL WALL
- B30 EAST SERVICE BUILDING, SOUTH TUNNEL WALL
- B31 - EAST SERVICE BUILDING, NORTH TUNNEL WALL

OVER CORING HOLE LOCATIONS SHOWN ON PLATE 2

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THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING FIELD INVESTIGATIONS

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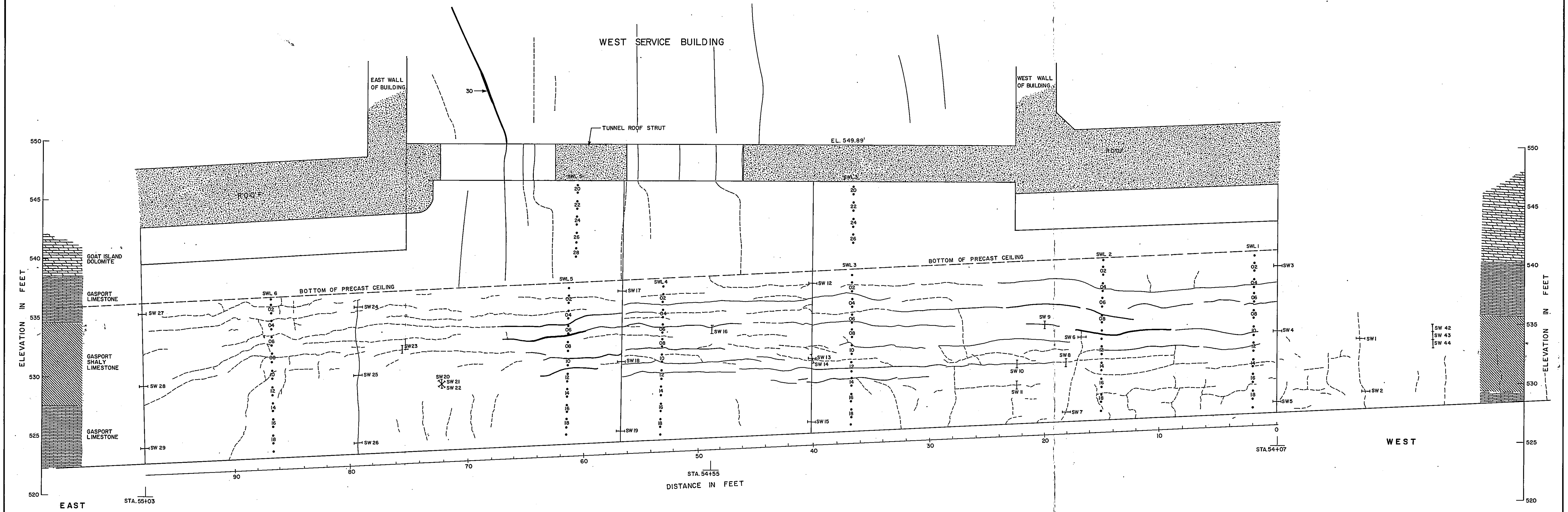
PLATE 4



TUNNEL-SOUTH WALL STRAIN OBSERVATION POSITIONS

NOTES

1. CRACKING BELOW PRECAST CEILING MAPPED MAY 1971, NOT SIGNIFICANTLY CHANGED FEB 1972
2. ABOVE PRECAST CEILING FIRST OBSERVED FEB 1972

LEGEND

MEASUREMENT GAUGE LENGTH

STUDS STANDARD "DEMEC" 8" GAUGE LENGTH

1 - 5
5 - 10
10 - 15
15 - 20
20 - 25

APPROXIMATE CRACK WIDTHS IN THOUSANDS OF AN INCH

SW5 SOUTH WEST WALL INDIVIDUAL GAUGE LENGTH NO. 5

SWL3 02 SOUTH WEST WALL ROW 3 GAUGE LENGTH NO. 2

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THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING PATTERN OF CRACKING SOUTH WALL

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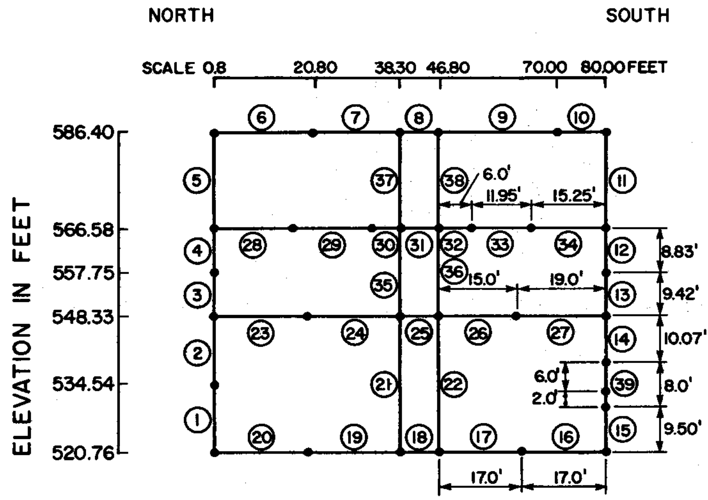
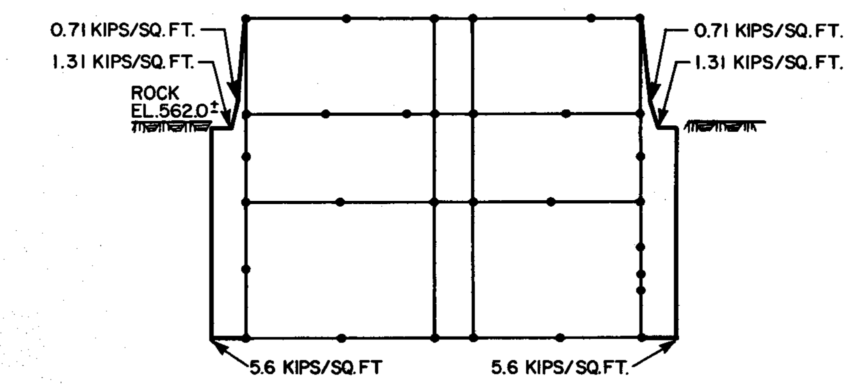
PLATE 5

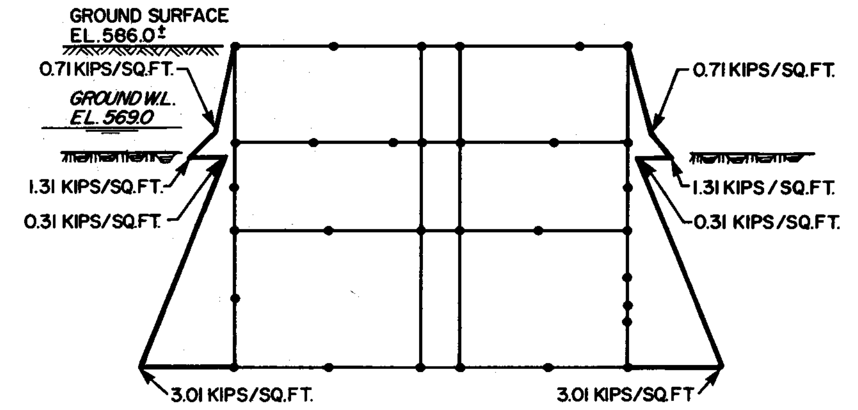
ORIGINAL DESIGN LOADS



REPRESENTATIVE STRUCTURAL FRAMEWORK OF WEST SERVICE BUILDING - STA. 54 + 65

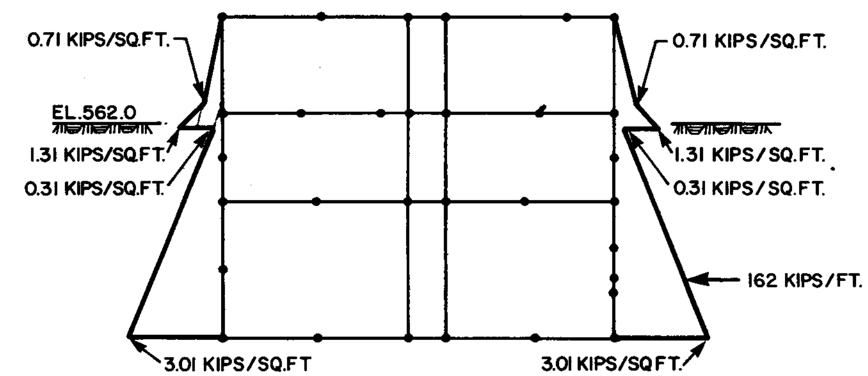


LOADING II-3
SYMMETRICAL ACTIVE BENTONITE LOAD (2.8 T/SQ.FT) PLUS DEAD LOAD PLUS FINAL EARTH LOADING

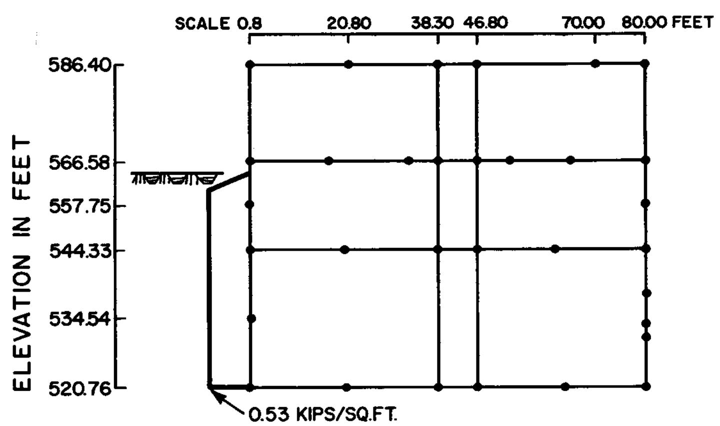


LOADING II-2
SELF WEIGHT PLUS DEAD LOAD PLUS FINAL EARTH LOADING

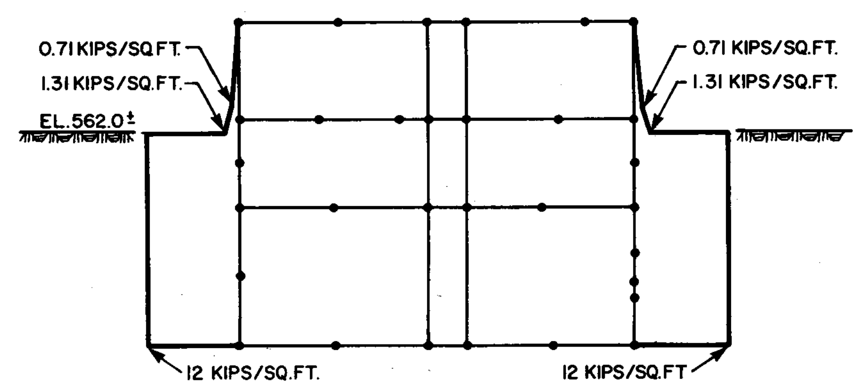
POSSIBLE LOADS



LOADING III-4
SELF WEIGHT PLUS DEAD LOAD PLUS FINAL EARTH LOADING PLUS MAXIMUM POINT LOAD AT ELEVATION 532 FEET



LOADING III-1
UNSYMMETRICAL BULKHEAD LOADING - ALLOWING FOR RATE OF POUR



LOADING III-3
SYMMETRICAL PASSIVE BENTONITE LOAD (6 T/SQ.FT.) PLUS DEAD LOAD PLUS FINAL EARTH LOADING

LEGEND

—•— NODE POINT FOR STRUCTURAL ANALYSIS

(17) MEMBER NUMBERS USED IN STRUCTURAL ANALYSIS

NOTE

LOADING NUMBERS REFER TO CASES DESCRIBED IN TABLE II AND III

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DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

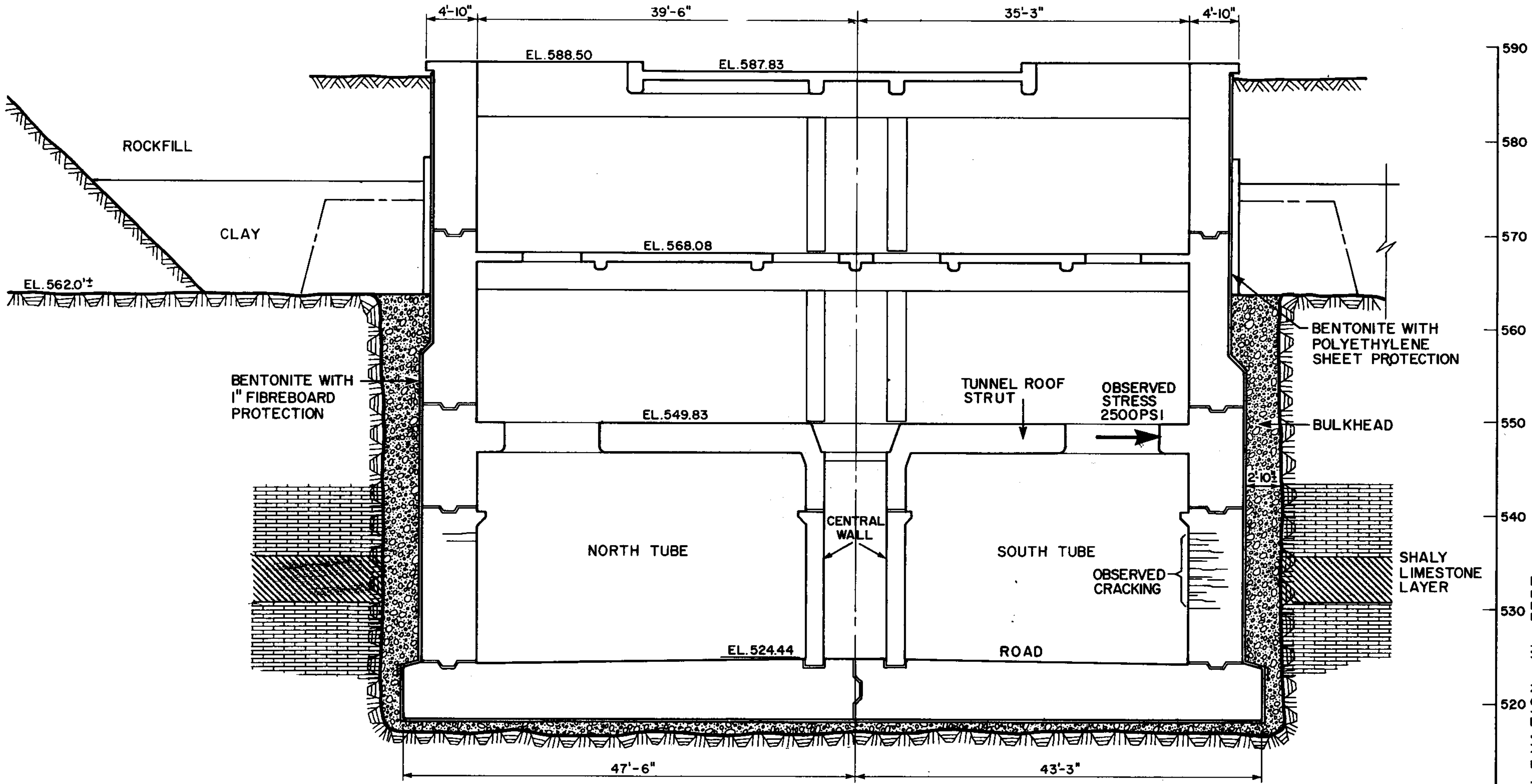
THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING STRUCTURAL ANALYSES

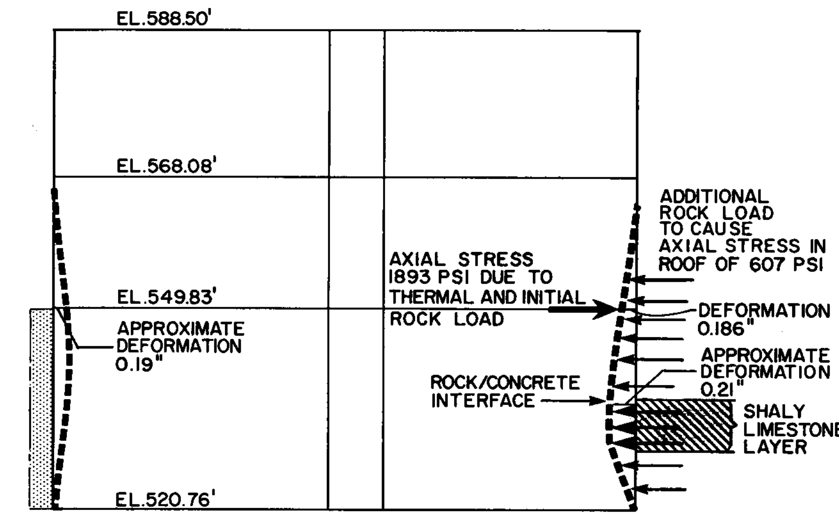
ACRES CONSULTING SERVICES LIMITED

FEBRUARY 1972

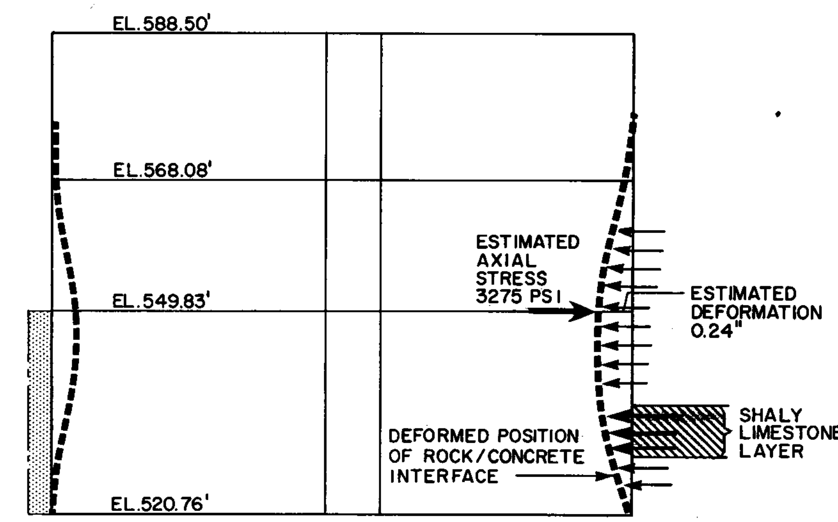
PLATE 6



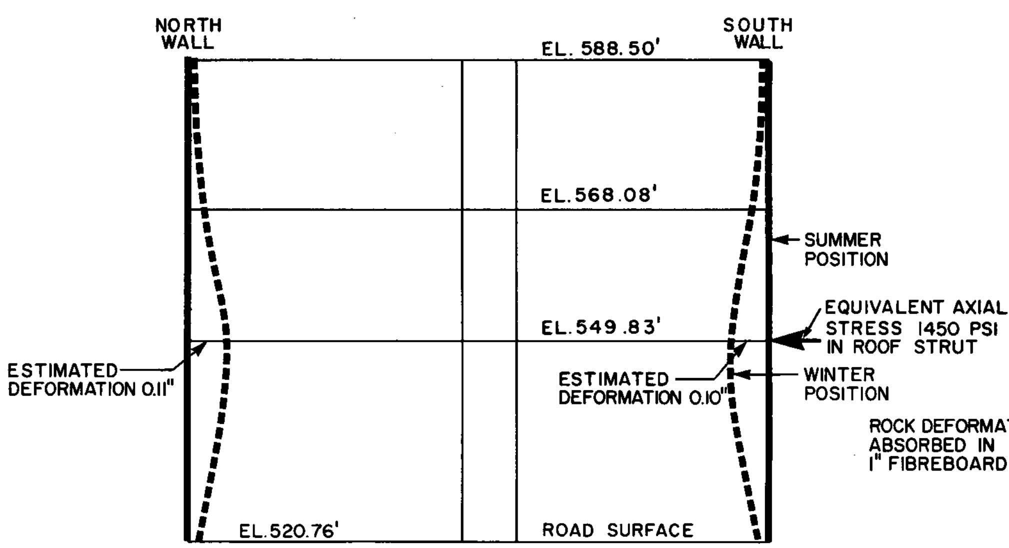
CROSS SECTION OF WEST SERVICE BUILDING - STA. 54 + 65



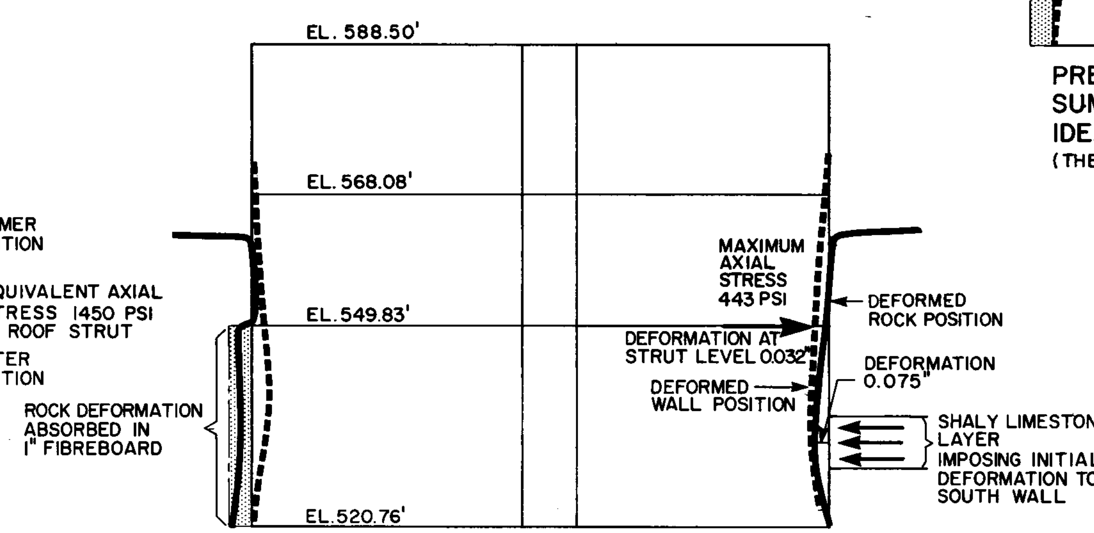
THEORETICAL LOADING MECHANISM FOR OBSERVED CONDITIONS OCTOBER 1971
IDEALISED STRUCTURAL FRAMEWORK



PREDICTED STRESSES IN ROOF
SUMMER 1972
IDEALISED STRUCTURAL FRAMEWORK
(THERMAL STRAIN SUPERPOSED ON ROCK SQUEEZE)



UNRESTRAINED THERMAL MOVEMENT
IDEALISED STRUCTURAL FRAMEWORK



INITIAL ROCK LOADING
ASSUMING LOADS LIMITED BY DEFORMATION REQUIRED TO CRACK WALL
EARTH AND HYDROSTATIC LOADING INCLUDED

NOTE

1. ROCK DEFORMATIONS IN NORTH PRESUMED TO BE ABSORBED BY 1" THICK FIBREBOARD

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DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

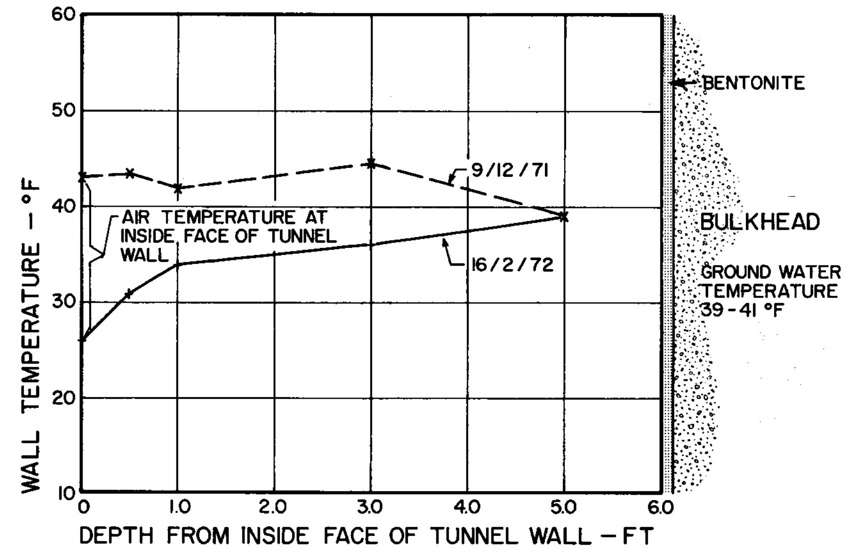
THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING
PROBABLE LOADING CONDITIONS

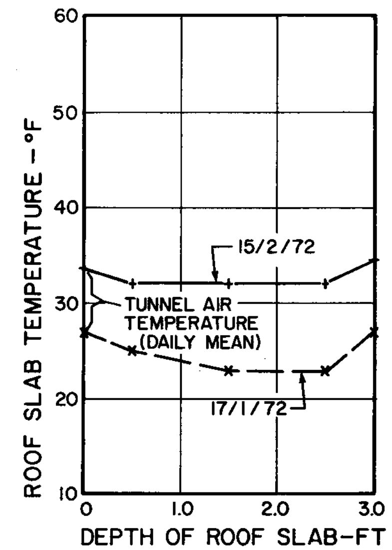
ACRES CONSULTING SERVICES LIMITED

FEBRUARY 1972

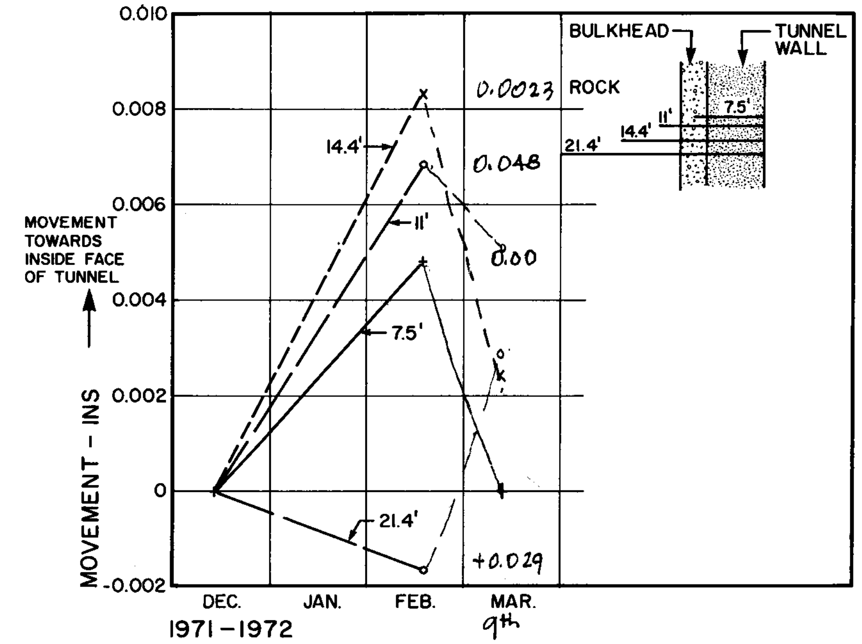
PLATE 7



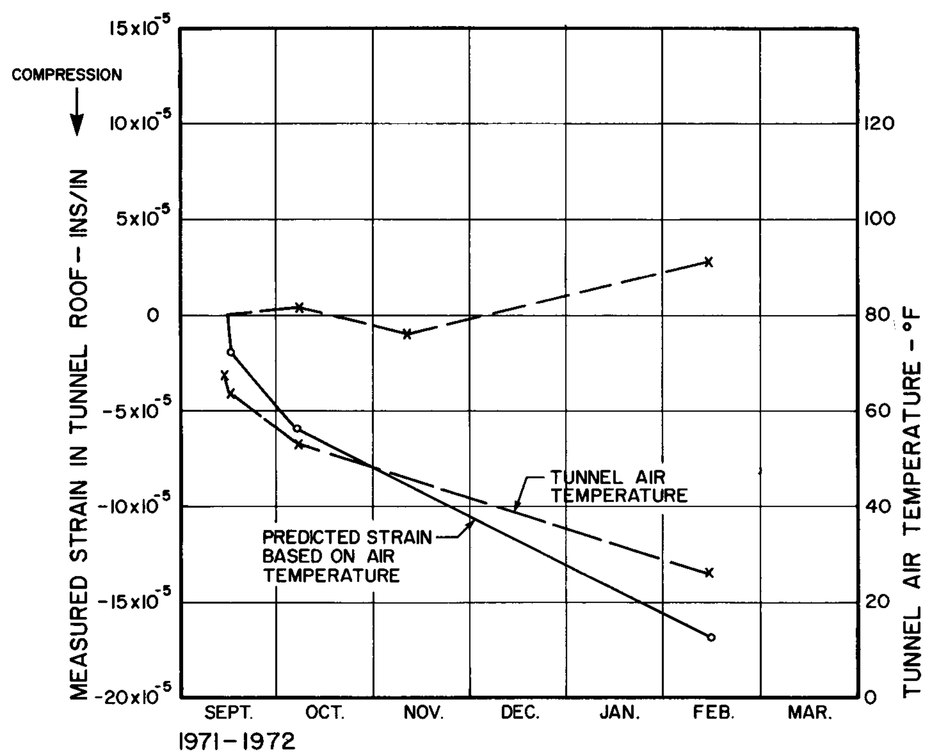
TEMPERATURE DISTRIBUTION IN SOUTH WALL



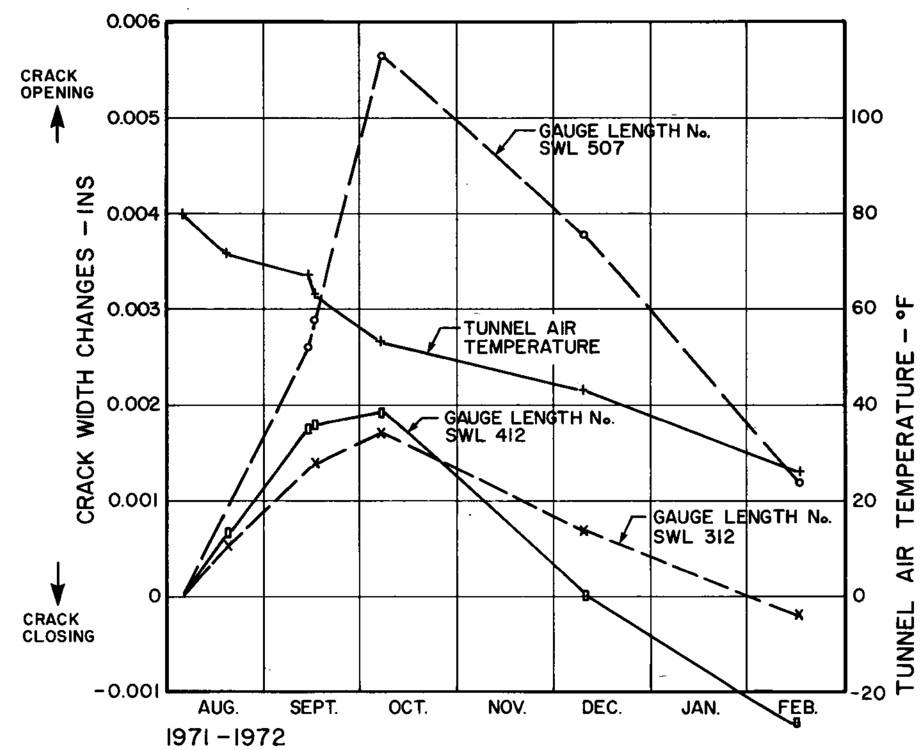
TEMPERATURE DISTRIBUTION IN ROOF SLAB



SOUTH WALL EXTENSOMETER MEASUREMENTS



TUNNEL ROOF STRAIN



CRACK WIDTH AND TUNNEL TEMPERATURE CHANGES WITH TIME SOUTH WALL OF TUNNEL

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS	
THOROLD TUNNEL STRUCTURAL INVESTIGATIONS	
CONTINUING OBSERVATIONS AT WEST SERVICE BUILDING	
ACRES CONSULTING SERVICES LIMITED	FEBRUARY 1972
PLATE 8	

APPENDIXES

APPENDIX A

SITE CONDITIONS AND FIELD INVESTIGATIONS - DETAILS

1 - Existing Conditions June 1971

At the commencement of the investigations in June 1971, extensive horizontal cracking was apparent in the south wall of the south tube under the west service building, between Station 53 + 92 and Station 55 + 07. Some of the cracks in this area were damp with associated deposits which had apparently precipitated from seepage.

Limited horizontal hairline cracking was also visible in the north wall at the west service building and in both external walls at the east service building.

One other horizontal crack exhibiting some seepage was noted at midheight of the south wall of the south tube at Station 68 + 23. This crack may be associated with the contraction joint at the west end of the first stage of construction for the tunnel east of the canal.

Minor diagonal cracks were visible at both portal structures in both the centre and outer walls of the tunnel, but these are considered normal for the type of structure.

Vertical shrinkage cracks occur at the midpoint of most of the tunnel sections between contraction joints. These are not considered to contribute towards any structural problems.

Comments made by the tunnel maintenance personnel and others associated with the tunnel refer to an apparent seasonal variation in the quantities of seepage from both the vertical and horizontal cracks, and from the construction joints. Maximum seepage was particularly noticeable during the spring.

2 - Construction Sequence

The construction sequence for the tunnel was governed by the necessity for maintaining navigation in the canal, except during the period December 15 to March 31 of each year, when the canal was unwatered. This resulted in completion of both the west end of the tunnel and then the east end, during the navigation season, with the central tunnel section constructed in the canal bed during the winter. Construction of the tunnel commenced in September 1965, and was completed in March 1968.

The construction sequence at the west service building was as follows:

Date	Operation
1965	
October 30	Overburden excavation commenced Station 50 + 90.
November 25	Overburden excavation Station 50 + 90 to 56 + 03 complete.
December 2	Rock excavation commenced Station 50 + 90.
December 17 to 21	Canal unwatered.
1966	
January 20	Rock excavation Station 50 + 90 to Station 56 + 03 complete.
March 22	Canal flooded.
July 2	Tunnel section under west service building completed, including roof.
August 24	North bulkhead, bottom 29-foot pour completed.
August 25	North bulkhead, top 15-foot pour completed.
September 1	South bulkhead, bottom 29-foot pour started, then abandoned.
September 2	South bulkhead, bottom 29-foot pour completed.
September 6	South bulkhead, top 15-foot pour completed.
September 10	Consolidation grouting north side of west service building completed.
September 17	Consolidation grouting south side of west service building completed, and seepage control dykes completed.
October 29	Backfilling completed from west and to Unit No. 10.

1967 July	Area around west service building backfilled.
September	Horizontal cracks observed under west service building in south wall.
October 31 - December 2	Painting of walls commenced. Blanket grouting north and south of west service building completed.

Excavation at the east service building was completed in February 1966, and the bulkhead concrete between the tunnel structure and the rock face was poured in November 1966.

3 - Problems Encountered During Construction

No problems of rock movements were reported or observed during excavation of the rock, but an appreciable quantity of seepage from the rock entered the excavation area at the south side of the west service building. In addition, a significant quantity of water flowed along the channel between the tunnel floor slab and the rock face. Pumping from the excavation for the west end of the tunnel was handled by two 8-inch diameter pumps, located at the east end of the excavation, at Station 56 + 03.

The waterproofing specified for the bulkhead areas consisted of bentonite panels 0.2 inch thick, protected by 1-inch fibreboard, and this system was utilized for the first concrete lift in the northwest bulkhead area. However, during pouring of the concrete the fibreboard became damp and tended to sag away from the wall, in spite of the supporting staples. Above 29 feet the fibreboard was replaced by a protective polyethylene sheet, and this design was used at all subsequent bulkheads.

When the south bulkhead at the west service building was first poured, on September 1, 1966, the form at the west end failed. This pour was abandoned and the concrete completely cleaned out. The bentonite panels, which were protected by lapped plastic sheeting, had been placed on the wall of the tunnel just prior to pouring of the concrete, and there are no records of remedial work before completion of the bulkhead the following day. To ensure no build-up of water, pipes were placed through the bulkhead to allow free drainage along the base of the rock cut to the pumps. However, it was noted that some ground water probably seeped from the rock into the concrete as it was poured, and to compensate for this, extra cement was added to the mix. Because of this seepage, plastic sheets were hung over the worst areas of the rock face, primarily to protect the workmen attaching the bentonite panels to the tunnel walls.

The grout curtains in the rock on the north and south sides of the bulkhead were completed a short time after the bulkhead was poured. Most of the holes were injected under gravity head and, during the grouting operation, leakage occurred through the rock face of the excavation. In an attempt to plug the seepage paths, oats were mixed with the grout, which was a bentonite/cement mixture in the ratio of 1:12. After approximately 12 hours the holes were pressure tested to 30 psi without appreciable grout take being recorded.

4 - Stress Measurement

In order to gain some understanding of the stress distribution in the tunnel structure, the overcoring technique was utilized.

This technique involved the placing of a deformation gauge in an EX diameter diamond drill hole (1-1/2-inch diameter), predrilled to the full depth of the test, and then concentrically overcored with a 6-inch diameter industrial coring bit. The relief of the stress in the 6-inch core causes diametrical deformation of the EX hole which was monitored by the gauge. The two-dimensional stress field is then calculated on the basis of these strains, using the appropriate elastic constants determined by laboratory tests and thick-wall cylinder equations. This test was repeated at intervals spaced at approximately 2 to 3 feet along the length of the EX hole. This type of test determines approximate values for the principal stresses acting in a plane perpendicular to the axis of the hole. The theoretical interpretation of the results is based on the behavior of a homogeneous, elastic, and isotropic material. Although concrete may be considered as approximately isotropic, it is not homogeneous with respect to the scale of the gauge probes due to the presence of aggregate particles. The degree to which the necessary assumptions of elasticity are justified is largely dependent upon the stress levels and directions acting in the concrete, and will generally be less valid in tensile zones and in zones of high compression.

A number of such tests were conducted to determine the trend of the stresses within the thickness of the tunnel wall, and to obtain an indication of the stress magnitudes in the tunnel walls, roadway, and roof.

As shown on Plate 4, two test holes were drilled in the south tunnel wall and one in the road slab at the west service building, one in the north tunnel wall at the west service building, and two in the tunnel roof strut at the west service building. The major results of these tests are summarized below.

4.1 - Tunnel Walls

It was initially intended to conduct tests throughout the thickness of both the tunnel wall and the bulkhead concrete. Unfortunately, however, it was not possible to conduct any tests within the bulkhead concrete, as deviation of the central EX hole, due to interference with the reinforcing steel, was sufficient to cause the latter to penetrate outside the 6-inch diameter hole.

Within the south tunnel wall,the results shown on Plate A-1 indicate that the stresses in the wall are consistent with bending taking place in a vertical direction with the member spanning between the roadway and the roof slab. Although there is scatter between individual test results, the trend is from a near vertical principal compressive stress of the order of 1,500 psi at the outside of the wall to an apparent tensile condition at the front face, which has subsequently been relieved by cracking. The scattering of the results in this area was due in part to the cracks intersected by the core. This was particularly evident in hole OC14 where an apparently open crack intersected the core between depths of 2 feet 6 inches and 3 feet 9 inches from the face of the wall. Due to closure of this crack upon overcoring, the quantitative results are questionable in this area.

The range of near vertical principal stress for the north tunnel wall was from approximately 1,000 psi compression to 300 psi tension and, in general, both principal stresses were lower in magnitude than those observed in the south wall.

A near horizontal compressive principal stress of between 200 and 1,000 psi was measured in both walls and no significant variation with depth could be determined.

4.2 - Roadway

Tests in the roadway slab showed that the maximum principal stress was compressive with a magnitude of approximately 400 to 900 psi oriented in a north-south direction, that is, across the tunnel. This result is consistent with the occurrence of bending in the tunnel wall caused by some loading mechanism external to the structure, applied in a north-south direction.

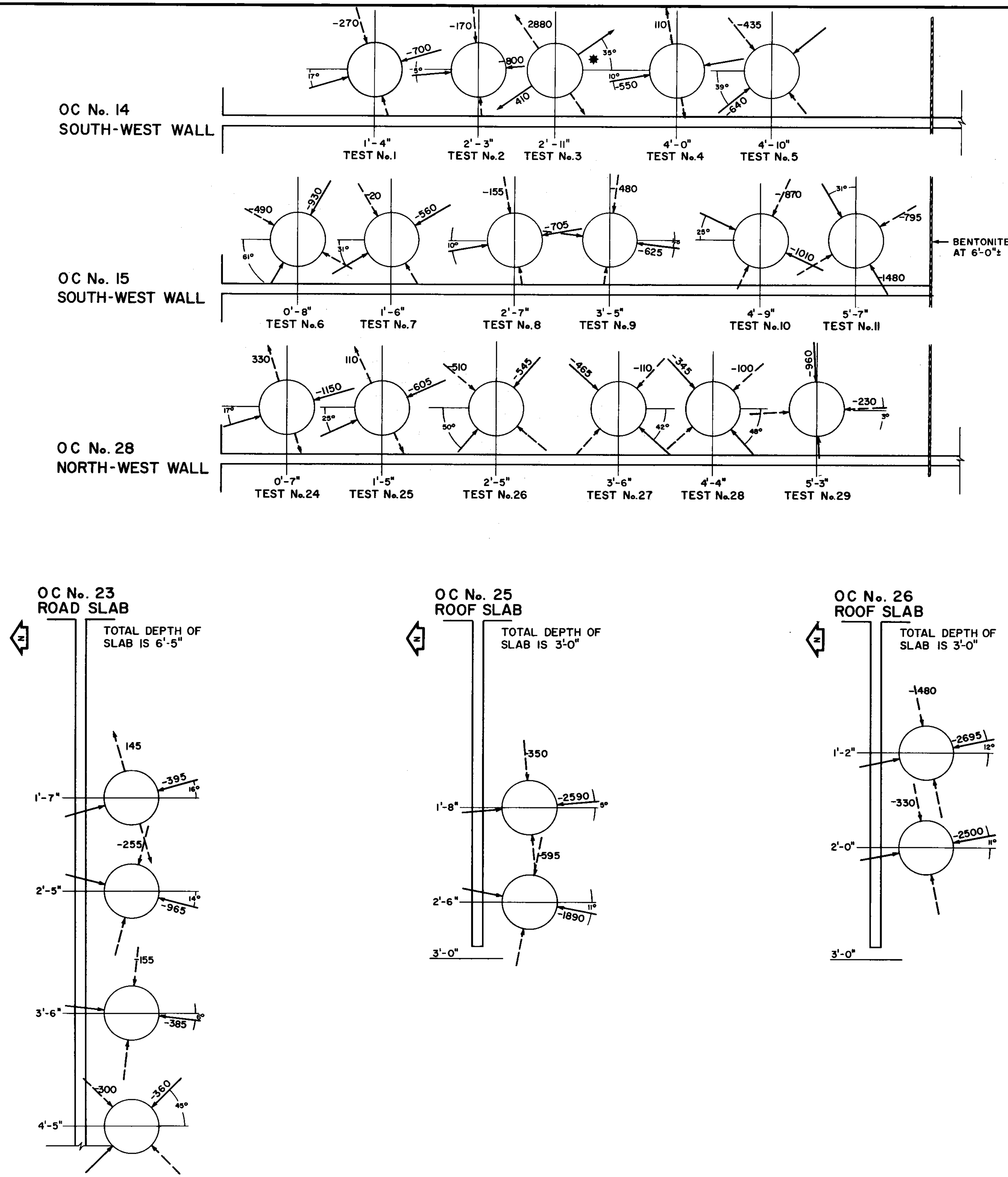
The minor principal stress was determined to be subparallel to the tunnel axis with an approximate compressive value of 200 to 250 psi.

4.3 - Tunnel Roof Slab
Elevation 549.83 feet

Some difficulty was encountered in the testing in the roof slab under the west service building, due to the fact that the 6-inch diameter cores split horizontally at the plane of both the upper and lower reinforcing bars. Thus, only two tests were conducted in each hole. However, the results of these tests showed consistently that the major principal stress is compressive acting in an essentially north-south direction, across the tunnel, and is comparatively high in magnitude, being of the order of 1,900 to 2,700 psi.

A compressive minor principal stress was also measured of magnitude 300 to 600 psi.

In summary, the stress measurements indicated that the tunnel walls in the bulkhead areas are in bending due to the application of some external load, acting across the tunnel. This load is probably transmitted axially through both the roadway and roof slabs.



NOTES

1. ——→ σ_1 = MAJOR PRINCIPAL STRESS
 – – –→ σ_2 = MINOR PRINCIPAL STRESS

2 ✱ READINGS DISTURBED DUE TO CORE SEPARATION ACROSS EXISTING CRACK

3 ALL RESULTS PLOTTED ON SECTIONS LOOKING INTO HOLE

4 POSITIVE DENOTES TENSION

ACRES	DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
	THOROLD TUNNEL STRUCTURAL INVESTIGATIONS
	OVERCORING TESTS RESULTS
ACRES CONSULTING SERVICES LIMITED	FEBRUARY 1972 — PLATE A-1

APPENDIX B

LABORATORY TESTING AND MATERIAL PROPERTIES - DETAILS

1 - Concrete

1.1 - Mixes Used and Strength Properties

The concrete mixes used during construction are shown in Table B-1, the tunnel structure being constructed from concrete with a specified minimum 28-day strength of 3,000 psi, and the bulkheads from similarly specified 2,000 psi concrete. All concrete was of the pre-mix, pre-batched variety. The additives used in the concrete are also listed in Table B-1.

The test cylinder results for concrete placed in the west service building area were examined in detail, together with the inspectors' comments on the conditions at pouring. It was noted that the measured 28-day strengths for concrete in the west service building structure ranged from 3,060 psi to 4,810 psi, with the average well in excess of that specified. Concrete cylinders for the south tunnel wall at the west service building ranged in 28-day strength from 3,740 psi to 3,830 psi.

1.2 - Chemical Content of Concrete

Results of chemical analyses of concrete samples obtained during the recent investigations are given in Table B-2.

2 - Ground Water and Material Deposits

2.1 - Chemical Analyses

Results of chemical analyses of samples of ground water and material deposits obtained during the present program are given in Tables B-3, B-4, and B-5.

Table B-1

THOROLD TUNNEL TYPICAL MIX DESIGNS

	Specified 28-Day Strength	
	2,000 psi	3,000 psi
Batching Constituents		
Cement (Normal Portland)	350 lb/cu yd	481 lb/cu yd
Sand (sp gr = 2.69)	1,520 lb/cu yd	1,320 lb/cu yd
3/4-inch Coarse Aggregate (sp gr = 2.71)	1,850 lb/cu yd	1,930 lb/cu yd
Maximum Water	295 lb/cu yd	250 lb/cu yd
Admixtures		
W.R.D.A.	6-1/2 fl oz/bag of cement (87.5 lb)	
Darex - to provide air entrainment	4 per cent air	6 per cent air
Calculated 28-day strength	2,800 psi	5,140 psi
Calculated density (28 days)	145 lb/cu ft	146.6 lb/cu ft

Notes

1 - The sand came primarily from a natural deposit belonging to Fonthill Sand and Gravel.

2 - The coarse aggregate was a crushed limestone obtained from Walker Brothers Quarries Limited at St. Catharines.

Table B-2

CHEMICAL ANALYSIS OF CONCRETE FROM THOROLD TUNNEL

Test	Per cent by Weight of Dried Sample Sample E-22	Sample OC-28	Sample OC-14
Moisture	2.4	2.1	2.3
Loss on Ignition	28.4	27.0	29.0
Chloride (Cl)	0.0177	0.1	0.0247
Sulphate (SO_4)	0.617	1.4	1.03
Cement Content	13.6	15.1	12.8
R_2O_3 (Al_2O_3 + Fe_2O_3)	7.75	7.5	8.1
Calcium Oxide (CaO)	28.6	25.6	26.7
Magnesium Oxide (MgO)	6.7	4.7	1.5
Free Lime	1.1	0.95	1.3

Tests were performed in Acres Laboratory, Niagara Falls.

Table B-3

CHEMICAL ANALYSIS OF SAMPLE OF GROUND WATER FROM DRILL HOLE D20

Test	Result
pH	6.8
Suspended Solids (Total)	294.0 mg/l
Suspended Solids (Combustibles)	152.0 mg/l
Suspended Solids (Inorganic)	142.0 mg/l
Total Dissolved Solids	310.0 mg/l
$R_2 0_3$ (Total)	155.0 mg/l
$R_2 0_3$ (Soluble)	77.2 mg/l
Chloride (Cl)	42.5 mg/l
Sulphate (SO_3)	42.8 mg/l
Calcium ($CaCO_3$)	155.0 mg/l
Magnesium (as $CaCO_3$)	50.0 mg/l
Nitrate	Traces
Total Alkalinity (MgO) As Calcium Carbonate	145.0 mg/l
Conductivity	420 μMHO

Table B-4

CHEMICAL ANALYSIS OF WHITE PRECIPITATE FROM HOLE D20

Test	Result
Water Extract	Alkaline
Chloride	Traces
Calcium ($CaCO_3$)	25 per cent
Magnesium ($MgCO_3$)	75 per cent
Free Lime	Trace

Mostly carbonate of magnesium and calcium.

Table B-5

CHEMICAL ANALYSIS OF DEPOSIT FROM HOLE OC28

Soft powder removed from cracked core.

Test		Result
R_2O_3 ($Al_2O_3+Fe_2O_3$)	(per cent by weight)	2.3
Calcium Oxide	(per cent by weight)	24.3
Magnesium Oxide	(per cent by weight)	5.3

The yellow streak is oxide of iron.

3 - Bentonite Panels

The bentonite waterproofing consists of Volclay panels as manufactured by the American Colloid Company, Skokie, Illinois. The panels were nominally 0.2 inch thick when placed, and consisted of facing layers of cardboard connected by corrugations, with the voids filled with a high sodium ion Wyoming-type bentonite.

The average moisture content of the panels under normal conditions was found to be between 4 and 6 per cent by weight, but when exposed to unlimited quantities of water, the moisture content of the bentonite increased to 720 per cent and the cardboard to 202 per cent. Under these conditions the thickness had increased by approximately 275 per cent.

Because the swelling characteristics are partially dependent upon the chemical content of the water absorbed, the above properties were determined using water from the south wall of the tunnel at the west service building. The free swell, that is, unconfined, increase in volume for bentonite in this water was approximately 16.5 times compared with 34 for tap water.

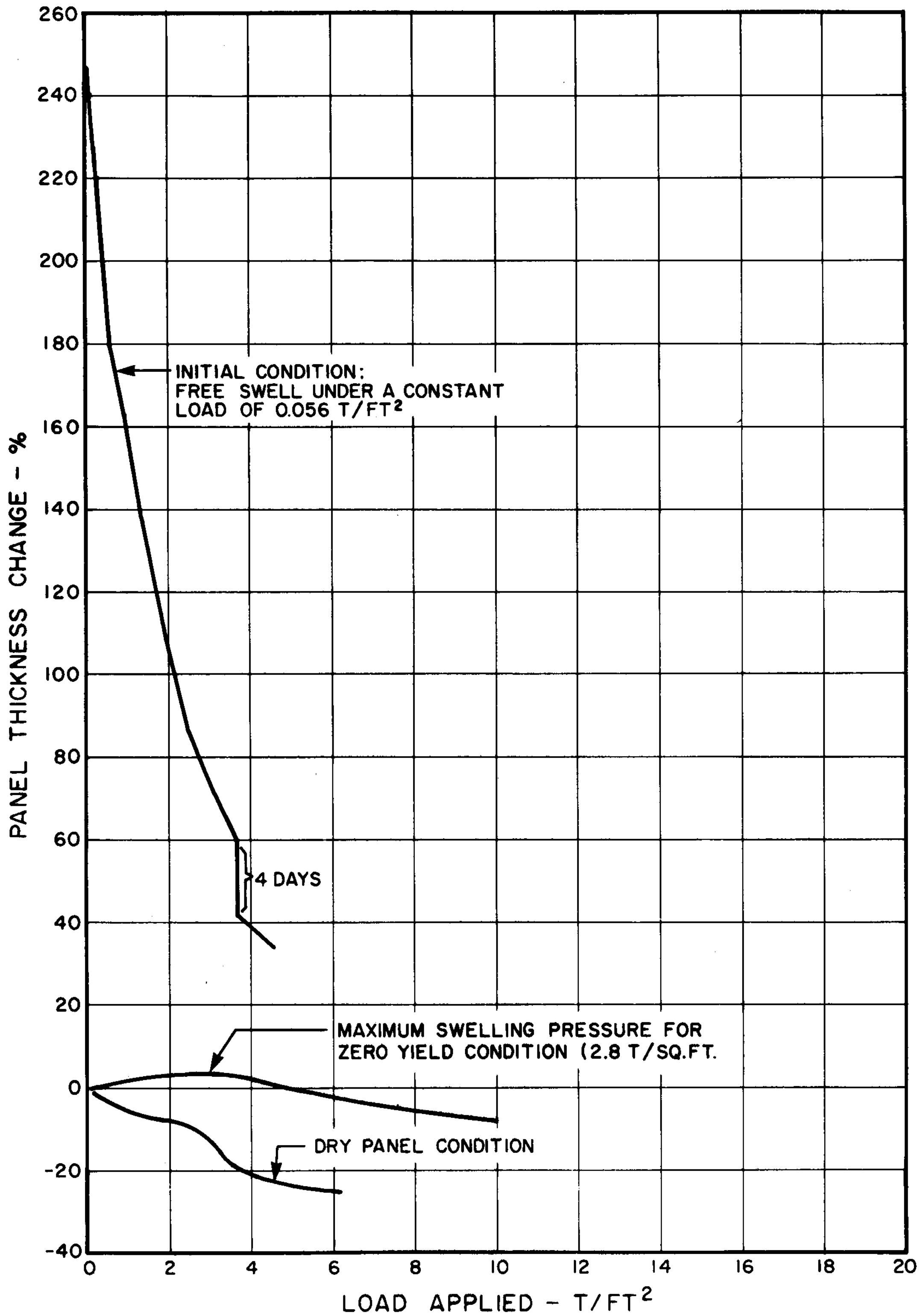
When water is added to a dry panel sample, the panel will expand in a direction normal to its plane. The pressure required to prevent this expansion is the "swelling pressure". The initial load applied to the dry sample prior to exposure to water is referred to as "confining pressure".

Information supplied by the manufacturer on the relationship of the physical swelling and the swelling pressure to the confining pressure, indicates that the maximum swelling pressure occurred under the no-yield condition, and could range from 20 to 80 psi.

The above values were confirmed in part by test results obtained from the Department of Transportation and Communications, which indicated a swelling pressure for the no-yield condition of 28 psi after 24 hours. This testing was carried out using a 2-inch diameter sample in a consolidation apparatus.

Tests performed by Acres, using 3-inch diameter samples, indicate that the maximum swelling pressure is 39 psi at 84 hours. The moisture content of the bentonite, once equilibrium had been reached, was 78 per cent, see Plate B-1.

Based on results of tests performed on a 2-inch sample by H. Q. Golder & Associates Limited for the Department of Transportation and Communications, a dry sample of the bentonite panel will sustain a load of approximately 83 psi, when exposed to water and allowed to swell under a light confining pressure of 1.4 psi. The test load was applied in increments up to 83 psi, once the swelling had reached equilibrium under the confining pressure. At a load in excess of 83 psi, the sample dimension decreased suddenly to less than that of the original dry sample.



NOTES

INITIAL THICKNESS OF DRY PANELS 0.2"

TESTING IN ACRES LAB

ACRES	DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
	THOROLD TUNNEL STRUCTURAL INVESTIGATIONS
	EXPERIMENTAL RESULTS FOR LOAD TESTING OF "VOLCLAY" BENTONITE PANEL

ACRES CONSULTING SERVICES LIMITED	FEBRUARY 1972	PLATE B-1

APPENDIX C

STRUCTURAL INVESTIGATIONS - DETAILS

Structural analyses were carried out to determine the behavior of a two-dimensional framework representing the most critical section in the west service building. This section of the structure is located at Station 54 + 65.

At this section stress conditions were expected to be at their worst, due to the configuration of the ventilation holes in the roof slab at elevation 549.83 feet, which would result in a stress concentration developing in the strut between these holes. In addition, the vertical span of the tunnel walls at the section was larger than that on adjoining tunnel sections, and sufficiently remote from the junction with these not to be significantly influenced by the latter.

Consideration of the distribution of external loading in terms of principal moment stress trajectories indicated that the width of the loaded external surface area of the tunnel wall carried by the above strut was approximately 16 feet. Wall loads and structural parameters used in the analyses of the west service building were derived for this width.

The structural parameters for each member in the two-dimensional framework were calculated assuming an uncracked reinforced concrete section. The dimensions of the framework were based on the position of the neutral axis in each member.

Loadings applied in the framework analyses for the west service building were divided into three types, as follows:

1 - Permanent Design Loads

These are the permanent loads considered in the design of the tunnel.

2 - Possible Loads

These are loads which are not considered to be permanent design loads. They include temporary loads as might have occurred during construction, and loads possibly caused by changes in conditions not predicted during design.

3 - Probable Loads Due to Rock Squeeze

Examination of the analyses for the above loads 1 and 2 in relation to the observed conditions and other information, led to the conclusion that the load actually occurring in the structure was probably caused primarily by rock squeeze.

The loads used, and the results of the analyses, are now described.

1 - Permanent Design Loads

1.1 - Self Weight of the Structure

The self weight of the structure includes all structural components above and below finished ground surface.

1.2 - Self Weight Plus Dead Load

In addition to the self weight, an additional dead load due to permanently installed machinery, that is, ventilation equipment and the steel structure above elevation 588.50 feet, was considered.

1.3 - Self Weight Plus Dead Load Plus Live Load

A live load (0.1 to 0.125 kip/sq ft) was considered on each floor and a snow load (0.06 kip/sq ft) on the roof, to give the maximum possible normally expected vertical loading on the structure without the surrounding rock fill.

1.4 - Self Weight Plus Dead Load Plus Final Earth Loading

On completion of the structure, normal loads include dead load and lateral earth pressure in addition to the self weight. The lateral earth pressure would be due to the surrounding rock fill and a ground water level equal to that of the canal, elevation 569 feet. It was also assumed that the bulkhead shrank away from the tunnel wall, thus subjecting the tunnel to full hydrostatic pressure.

1.5 - Self Weight Plus Dead Load Plus Live Load Plus Final Earth Loading

This is load 1.4 above, with the additional maximum live load (1.3), and represents the maximum total loading for operating conditions.

1.6 - Active Bentonite Loading

The maximum swelling pressure which has been measured for the bentonite waterproofing panels in ground water from the area has been 2.8 tons/sq ft. This was the pressure required to prevent a dry sample of the bentonite panel from expanding when access of water is allowed. This loading was applied in conjunction with the final earth loading above rock surface level, elevation 564 feet.

The stresses calculated for the members of the west service building for the loads described above are summarized in Table C-1.

Stress levels in the major members are less than the maximum allowable design stresses and are also less than those now measured.

It is concluded that the permanent design loads are not large enough to cause the structural problems now observed at the west service building.

2 - Possible Loads

2.1 - Self Weight Plus Fluid Concrete Bulkhead

During construction, after the concrete structure of the west service building up to elevation 588.50 feet was complete, the concrete bulkhead between the tunnel walls and the rock was poured. The concrete was considered to act as a fluid (density 150 lb/cu ft) for the full height of the pour, elevation 520.76 feet to elevation 564 feet, symmetrically on the north and south tunnel walls.

2.2 - Unsymmetrical Bulkhead Loading Allowing for Rate of Pour

The bulkhead on the north side of the tunnel was poured first, thus causing an unsymmetrical load on the structure. This loading might cause a locked-in stress in the bentonite panels which would continue to be applied after the concrete had set, assuming there was no shrinkage. This loading was calculated using procedures outlined in ACI Pub SP-4 for Formwork Design and takes into consideration the rate of pouring and temperature to determine the setting time.

2.3 - Symmetrical Bulkhead Loading Allowing for Rate of Pour

With both north and south bulkheads completed, it was still assumed that no shrinkage had occurred and the locked-in stress in the bentonite panel was still acting. This condition gave a symmetrical loading in the north and south walls of the tunnel.

2.4 - Final Design Loading

This was the final earth loading above the rock surface level, elevation 564 feet, applied in conjunction with the symmetrical bulkhead loading (2.3) above.

2.5 - Passive Bentonite Loading

Based on laboratory testing the maximum load which a bentonite panel in an expanded condition can sustain before being compressed to a thickness dimension less than its original dry dimension, is 6 tons/sq ft. This was obtained for a sample which was allowed to expand, when water was added, under a load of 0.1 tons/sq ft. This load was then increased until the sample thickness was less than the original dry thickness.

For this loading condition it is considered that the tunnel roof strut shrinks sufficiently in winter, due to thermal effects, to allow the bentonite to expand behind the tunnel walls. When rising temperatures occur in summer, the bentonite resists the resulting compression and it is assumed that this maximum load of 6 tons/sq ft is applied. If the resulting compression tends to exceed 6 tons/sq ft then the bentonite will consolidate, allowing the tunnel to expand freely to its original size, with loading behind the wall no higher than that created in the active case considered earlier.

The stresses calculated for the members of the west service building for the loads described above, are summarized in Table C-2.

Stress levels calculated for the major members are less than those now measured. Hence, it is concluded that none of these possible loads is the prime cause of the structural problems now observed at the west service building.

3 - Probable Loads Due to Rock Squeeze

3.1 - Nature of Load

The position of maximum flexure in the south tunnel wall at Station 54 + 65 was determined, by inspection of the cracking pattern, to be at elevation 532 feet, which corresponds closely to the elevation of the shaly limestone stratum. An initial structural analysis was carried out to determine the magnitude of the point load applied to the tunnel wall at this elevation, which would exceed the tensile strength of the concrete and cause the cracking as observed.

The results of this analysis are shown in Table C-2. The point load to cause cracking was calculated to be 162 kips/ft which would be accompanied by a total axial shortening at tunnel roof level of 0.082 inch.

It should be noted that it was assumed in the analysis that rock squeeze applied to the north tunnel wall below roof level was absorbed by the 1-inch thick fibreboard protection over the bentonite waterproofing.

It was apparent that the average axial stress of 443 psi, calculated for the tunnel roof was considerably less than the stress of 2,500 psi measured in October 1971. An upward adjustment of the point of application of the point load did not significantly alter the roof load, and was not consistent with the observed cracking pattern. It was concluded that the cracking in the wall could be caused initially by a rock squeeze deformation originating in the shaly limestone stratum, but that additional axial loads are now being applied direct to the tunnel roof strut.

With the cracking pattern as developed, it can be shown that the load-carrying capacity of the tunnel wall will remain substantially constant for deformations considerably in excess of those obtained to date. Hence the calculated reaction of 443 psi, which forms the part of the load in the roof strut due to the horizontal load applied to the wall, is a maximum value. HORIZONTAL LOAD

To determine the origin of the additional load actually measured in the tunnel roof strut, consideration was given to thermal stresses resulting from constraint of the strut by the rock, either side of the west service building. Field observations showed that the temperature in the roof strut is close to the surrounding air temperature, that is, atmospheric. The effective temperature range for the strut was estimated to be 55 degrees F, which would create a total unrestrained movement of 0.206 inch across the tunnel. To restrain this movement, an axial prestress on the strut, amounting to 1,450 psi, is required. TEMPERATURE

Hence, the maximum stress level in the roof strut in the summer, assuming full restraint of thermal expansion, is 1,893 psi. The actual stress measured in the strut in October 1971 was 2,500 psi, indicating that a greater load must be applied to the strut than that due to changes in temperature.

It has already been established that significant additional load cannot be applied to the tunnel wall below the strut. Hence, it is concluded rock squeeze in the rock stratum at the level of the strut is imposing a significant deformation to the strut, resulting in the higher measured stress.

If this deformation is being applied to the strut, it can be expected that, since the resulting stress is substantially higher than the predicted thermal stress, the strut length should remain unchanged between summer and winter, except for any increased deformation due to rock squeeze. Field measurement of the strut movements to date have, in fact, confirmed that negligible deformation is occurring with change in temperature.

Further analyses were carried out to determine if the south tunnel wall and bulkhead might be acting together as a composite structural member. This member would be appreciably stiffer than the tunnel wall alone and would require a higher load to cause cracking than that necessary to produce the observed stress in the tunnel floor. Hence it was concluded that such composite action is unlikely and that a deformation, due to rock squeeze, must be acting directly on the tunnel roof strut in excess of the reaction to the wall load and thermal stress.

Assuming that deformation due to rock squeeze is applied directly at the roof strut level to create the measured stress condition in the strut, it is possible to predict that the tensile stress in the south face of the ventilation holes each side of the strut will exceed the limit for cracking. This has been confirmed by field observations in February 1972, when a series of vertical cracks was noted, as predicted. Similar cracks are apparent at the north wall of the tunnel confirming that rock squeeze deformation is occurring north and south of the tunnel at roof strut level.

3.2 - Rate of Increase in Deformation

Based on estimates of thermal strain shrinkage and compression in the tunnel roof slab, the total deformation of the concrete/rock interface to date, over a period of 5 years or so, is of the order of 0.554 inch. Assuming this is equally distributed between north and south and has occurred at a constant rate, then the rate of deformation of the roof strut amounts to 0.11 inch per year, which is equivalent to an increase in axial compressive stress in the roof strut of 775 psi per year.

Assuming that the maximum stress in the roof strut in 1971 was 2,500 psi, as measured, then it appears that this stress will probably be increased to about 3,275 psi in the summer of 1972.

It is worth noting that comparable deformations to those estimated above were observed at the Canadian Niagara Power Company wheel pit, as described in Appendix D.

3.3 - Additional Loads

Consideration of the observed widths of cracks in the tunnel wall indicates that the yield stress for the reinforcing steel has been exceeded locally. However, based on the properties of the steel used, specified in accordance with CSA G30.2, the elongation involved is small compared to that at ultimate load. Hence, it was concluded that the cracked area of wall has the capacity to withstand a substantially increased deformation.

Table C-1

MAXIMUM STRESSES WEST SERVICE BUILDING
STATION 54 + 65 - PERMANENT DESIGN LOADS

Load Description	Maximum Stress Roof El 549.8 ft	Maximum Stress Floor	Maximum Stress South Wall
1.1 - Self Weight of the Structure	- 38 + 55	+ 5 - 3	- 63 - 61
1.2 - Self Weight Plus Dead Load	- 35 + 55	+ 7 - 6	- 60 - 67
1.3 - Self Weight Plus Dead Load Plus Live Load	- 42 - 69	+ 9 - 7	- 65 - 73
1.4 - Self Weight Plus Dead Load Plus Final Earth Loading	-197 -142	-151 + 70	+ 63 -182
1.5 - Self Weight Plus Dead Load Plus Live Load Plus Final Earth Loading	-203 -128	-150 + 69	+ 58 -190
1.6 - Self Weight Plus Dead Load Plus Active Bentonite Load (2.8 tons/sq ft) Plus Final Earth Load	-687 -632	-431 +234	+245 -376

Stresses in psi: + = tension, - = compression.

For roof and floor, stresses are (top) (bottom) of member.

For south wall, stresses are (inside) (outside) surfaces of the tunnel wall.

Table C-2

MAXIMUM STRESSES WEST SERVICE BUILDING
STATION 54 + 65 - POSSIBLE LOADS

Load Description	Maximum Stress Roof El 549.8 ft	Maximum Stress Floor	Maximum Stress South Wall
2.1 - Self Weight Plus Fluid Concrete Bulkhead	- 225 + 389	-247 + 77	+261 -385
2.2 - Unsymmetrical Bulkhead Loading	+ 19 - 54	+120 -105	+ 26 -155
2.3 - Self Weight Plus Symmetrical Bulkhead Load	- 92 - 12	- 35 + 17	- 42 - 83
2.4 - Self Weight Plus Symmetrical Bulkhead Plus Dead Load Plus Final Earth Load	- 110 - 13	- 33 + 16	- 47 - 81
2.5 - Self Weight Plus Dead Load Plus Passive Bentonite Load (6 tons/sq ft) Plus Final Earth Load	-1,414 -1,414	-934 +508	+602 -749
2.6 - Self Weight Plus Dead Load Plus Final Earth Loading Plus Maximum Point Load at El 532 feet	- 387 - 498	-748 +455	+810* -929

Stresses in psi: + = tension, - = compression.

For roof and floor, stresses are (top) (bottom) of member.

For south wall, stresses are (inside) (outside) surfaces of the tunnel wall.

*Limiting tensile stress at which cracking will occur.

APPENDIX D

ROCK SQUEEZE DATA

Experience in the construction of tunnels and canals within the Niagara Falls and Northern New York regions has indicated that the phenomenon of rock squeeze does occur.

An early example, where significant movements were observed, was reported by Adams (Ref. 1) to have occurred in wheel pit slots of power plants in the area of Niagara Falls. These wheel pits are open cuts 20 feet wide, 425 feet long, and 182 feet deep. There are three such pits, which were excavated through approximately 10 feet of overburden, overlying 150 feet of dark gray Lockport dolomite, below which was a layer of soft bluish gray argillaceous Rochester shale. It was reported that the observed movements were larger in wheel pit slot number one than in that of Canadian Niagara Power Company, even though each was excavated in similar geological conditions.

Since the time of excavation, observations have been taken at regular intervals at various depths in the wheel pit of the Canadian Niagara Power Company, and the results are shown on Plate D-1. These measurements indicate that for this wheel pit the movements observed are greater at depth than at the top of the excavation, and also that, although the rate of increase in the magnitude of the movement has decreased with time, the movements continued for at least
40 30 years.

Other observations made on Ontario Hydro structures in the Niagara Falls area are reported by Hogg (Ref. 3). The measurements described were taken on open-cut canals and underground tunnels. However, the observations were only taken for a relatively short time during construction. Measurements of closure in the width of one of the power canals are detailed and indicate that a significant portion of the total movement observed occurred within the first 100 days after excavation. The period of observation was probably too short to assess long-term effects. This canal was excavated to a depth of 50 to 60 feet in the limestone cap road, and the maximum closure occurred at the rock surface.

Further information on the rock behavior in this area is contained in an unpublished paper by Rose (Ref. 4), which summarizes all reported surface rock movements in the Niagara Falls and Northern New York regions up to 1951.

A summary of movements relative to open cuts is reproduced from this paper in Table D-1. It is worth noting that example 4 of this table reports movements at Lock 35 of the New York State Barge Canal, where the magnitude at a point 24 feet below the top of the cut exceeded that at the top. It is stated that the top few feet of this excavation are in Lockport dolomite and the lower portion in Rochester shale. This geological stratification is similar to that of the Canadian Niagara Power Company's wheel pit, discussed earlier, in which similar movements were observed.

The essential mechanism for rock squeeze in these open-cut excavations is relaxation, resulting from the release of high horizontal compressive stresses previously existing in the bedrock. Recent work by the United States Army Corps of Engineers on the bed of the Niagara River (Ref. 5), indicates that the in situ horizontal compressive stresses are of the order of 1,000 psi within a depth of 11 to 21 feet below the rock surface.

LIST OF REFERENCES

1 - Adams, E. D. "Niagara Power. History of the Niagara Falls Power Company 1886-1918 - Volume II." p. 48.

2 - Private communication from Hydro Electric Power Commission of Ontario, January 1972.

3 - Hogg, A. D. "Some Engineering Studies of Rock Movement in the Niagara Area." Geological Society of America. Eng. Geol. Case Histories No. 3, May 1959. pp. 1-12.

√ 4 - Rose, C. W. Buffalo District Geologist. United States Army Corps of Engineers 1951. "Preliminary Report on Rock Squeeze Studies, 1951." (Not published). pp. 1-15.

5 - Private communication from United States Army Corps of Engineers, December 1971.

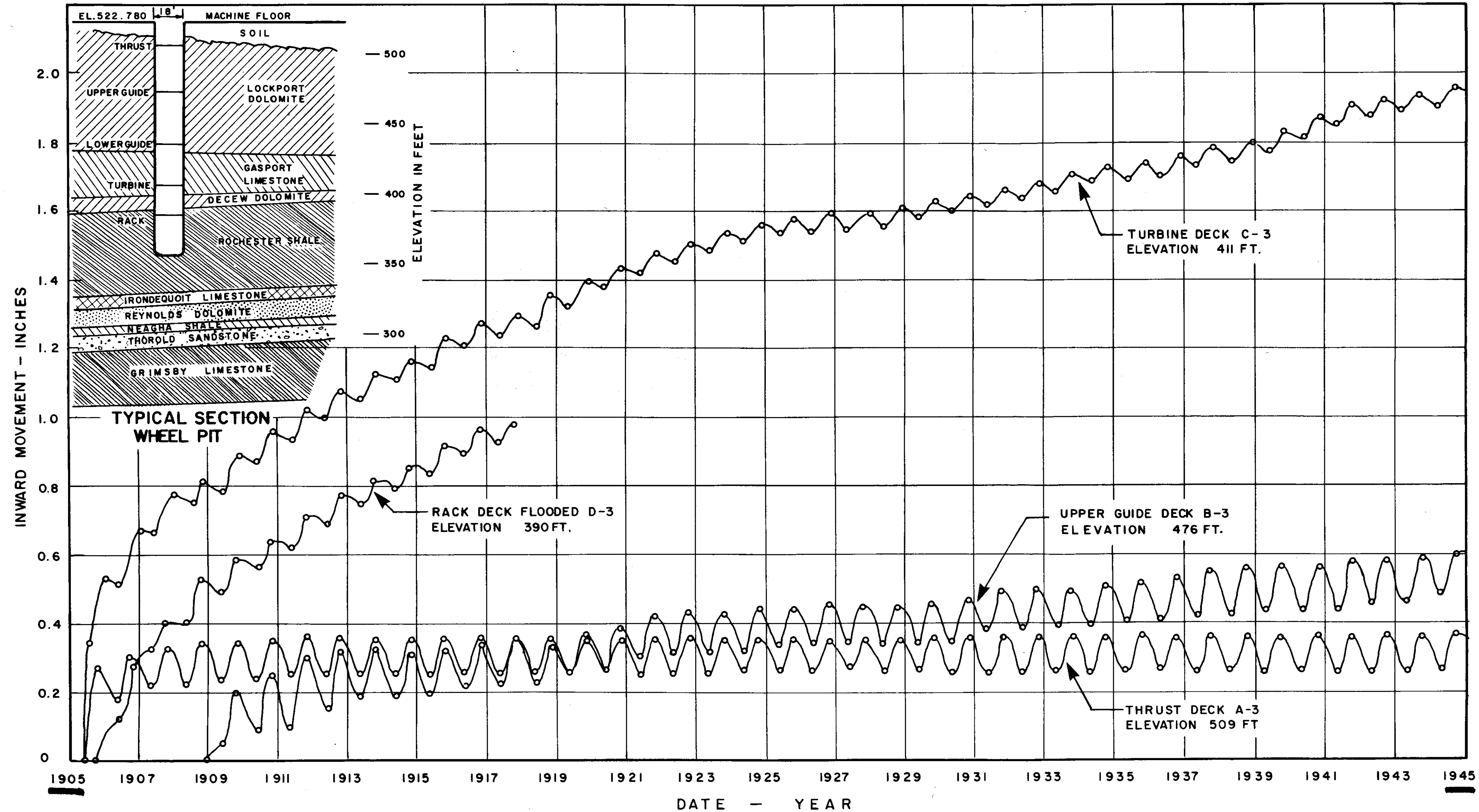
Table D-1

PRECEDENT DATA ON ROCK SQUEEZE

	Location	Reference	Structure	Movement
1	Lockport, New York	4	Main Street Bridge 3-hinged steel arch bridge, span 125 feet, across Barge Canal. Abutments set in Lockport dolomite, probably in Gasport and Decew members.	Abutment closure of 9 inches from 1910 to 1935.
2	Lockport, New York	4	Transit Street Bridge. Prospect Street Bridge. Also across Barge Canal.	Abutment closure reported, but no measurements taken.
3	Lockport, New York	4	New York State Barge Canal Lockport area.	Observed that base of canal, in Rochester shale, rose approximately 2 feet maximum, requiring removal to maintain navigation clearance.
4	Lockport, New York	4	New York State Barge Canal Lock 35 Primarily in Rochester shale. Top "few feet" in Lockport dolomite.	Closure of 6 inches at top of cut and 8 inches at 24 feet below top of cut noted in 1925. From 1925 to 1948 a further "average" closure of 0.18 foot measured.
5	Lockport, New York	4	New York State Barge Canal Lock 34 In Rochester shale.	Closure of unspecified amount noted in 1925. Further closure from 1925 to 1948 of 0.12 foot.
6	Lockport, New York	4	Brodas Avenue Bridge across Barge Canal Abutments set in Lockport dolomite.	Abutment closure approximately 0.6 foot.

	Location	Reference	Structure	Movement
7	Lockport, New York	4	Buffalo Road Bridge.	Expansion end wall was cut back 8 inches in 1922.
8	Lockport, New York	4	Lyell Road Bridge.	Expansion end wall cut back 4 to 5 inches.
9	Rochester, New York	4	Henritta Road Bridge across Barge Canal. Abutments on Lockport dolomite.	Abutment movement of approximately 1 inch.
10	Niagara Falls, Ontario	4	Wheel pit of the Rankine Plant, Niagara Mohawk. 20 feet wide by 425 feet long by 180 feet deep. Top 150 feet in Lockport dolomite.	Closure of 4 inches.
11	Niagara Falls, New York	4	Wheel pits 1 and 2 at Adams Plant.	Slight closure movements.
12	Niagara Falls, Ontario	4	Ontario Power Company Plant. Access tunnel from elevator to generator room. In Rochester shale.	Floor heaved 1/4 inch to 3/4 inch. Minor wall closure approximately 0.24 inch.
			Johnson valve, Unit 15.	Valve buckled after 21 years in service. Compressed approximately 0.075 inch out of true.
13	Niagara Falls, Ontario	4	Queenston Plant. Access tunnel in Grimsby sandstone and Power Glen shale.	Slight heaving of floor. Walls visibly deflected inwards in places.
			Reported that entire plant is slowly tipping towards river, necessitating frequent adjustment of shaft bearings. May be due to movement of Lockport dolomite towards river.	

	Location	Reference	Structure	Movement
			Queenston Canal Control Gate.	3/8-inch removed from gate in 1918 to eliminate jamming. Again jammed in 1950.
14	Niagara Falls, New York	4	Buffalo Avenue Bridge. Erie Avenue Bridge over Canal.	Considerable buckling and distortion reported.
15	Niagara Falls, Ontario	3	Sir Adam Beck Station 2.	Closure of Intake, Canal and tunnels measured. Maximum closure of cuts approximately 1 inch after 15 months. At that time, closure rate approximately 0.005 inch per month.



MEASUREMENTS MADE BETWEEN UNITS No. 3 & No. 4

NOTE
REPRODUCED BY PERMISSION OF
HYDRO ELECTRIC POWER COMMISSION
OF ONTARIO

ACRES	DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
	THOROLD TUNNEL STRUCTURAL INVESTIGATIONS
	ROCK SQUEEZE OBSERVATIONS IN CANADIAN NIAGARA POWER COMPANY WHEEL PIT
ACRES CONSULTING SERVICES LIMITED	FEBRUARY 1972 — PLATE D1

APPENDIX E

BOREHOLE LOGS AND TELEVISION CAMERA RECORDS

H. G. ACRES LIMITED — CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) **JOB No.** P2499.02

PROJECT Thorold Tunnel **HOLE No.** P-1

SITE South Tunnel Wall **SHEET No.** 1 **OF** 1

CONTRACTOR: Canadian Longyear Ltd.

STARTED .M. Oct. 15 19 71

FINISHED .M. Oct. 15 19 71

METHOD OF DRILLING:

- **SOIL**
- **ROCK** Diamond Drill
- **CASING DIAM.**
- **CORE DIAM.** Ax

LOCATION:

- Station
- ~~LATITUDE~~ 54 + 62
- ~~DEPARTURE~~
- **BEARING** South
- **INITIAL DIP** 10°
- **OTHER DIPS**

ELEVATIONS:

- ~~DATUM~~ at wall face: 530.15 ft
- **DRILL PLATFORM**
- **GROUND SURFACE**
- **ROCK SURFACE**
- **BOTTOM OF HOLE**
- **WATER TABLE**

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.2		Tunnel Wall Concrete: The concrete core from the tunnel wall generally is intact, well compacted concrete with no evidence of honey-combing, segregation, or deterioration.	
6.2		Bentonite seal.	
6.22 to 7.1		Bulkhead Concrete: At the bentonite bulkhead interface the core is badly fractured and exhibits chemical leaching of cement. Clean water flowed steadily after the hole was drilled.	98.3%
7.1		End of Borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED [signature]

DATE March 1972

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-1

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
1.8	Fracture, minor chemical weathering	Vert			x			x					
3.4 to 3.5	Core fractured, chemical weathering and leaching of the cement							x					
6.2	Bentonite seal												
6.2 to 7.1	Core fractured, **pronounced** leaching of the cement							x					
7.1	End of Borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. P-2

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 15 1971

FINISHED .M. Oct. 15 1971

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ Station 54 + 55 ELEVATIONS: ~~DATUM~~ at wall face: 530.57 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 10° ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.6		Tunnel Wall Concrete: The concrete core from the tunnel wall is in excellent condition and generally is intact. However, a few fractures in the core exhibit very minor chemical weathering and leaching of the cement.	
6.6 to 8.0		Bulkhead Concrete: At the bentonite bulkhead interface the core exhibits relatively more fractures and pronounced chemical leaching of the cement, confined to the fracture zone. 1/4 inch grout was recovered at 6.6-foot depth. Clean water flowed steadily after the hole was drilled.	100.0%
8.0		End of Borehole.	

INSPECTOR W. B. Pratt

APPROVED

LOGGED BY M. Walia

DATE March 1972

FORM NO. 91-C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-2

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	DESCRIPTION: SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
2.1	Fracture	Vert			x			x					
4.4	Fracture	Vert			x			x					
5.3	Fracture	Vert			x			x					
6.1	Fracture	72^{o}		x			x						
6.15 to 8.0	Core fractured, exhibits chemical leaching of the cement							x					
7.9	Fracture				x			x					
8.0	End of Borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario)   JOB No. P2499.02

PROJECT: Thorold Tunnel   HOLE No. P-3

SITE: South Tunnel Wall   SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.   STARTED .M. Oct. 15 19 71   FINISHED .M. Oct. 15 19 71

METHOD OF DRILLING: SOIL   CASING DIAM.

ROCK Diamond Drill Station   CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ 54 + 53   ELEVATIONS: ~~DATUM~~ at wall face: 530.69 ft

~~DEPARTURE~~   DRILL PLATFORM

BEARING South   GROUND SURFACE

INITIAL DIP 10° ROCK SURFACE 529.21 ft

OTHER DIPS   BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.55		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured, but the fracture faces do not exhibit chemical weathering or leaching of the cement. Steel reinforcing - 0.2 foot	
6.55		Bentonite seal.	
6.57 to 8.75		Bulkhead Concrete: The core at the bentonite bulkhead interface contains more fractures than the tunnel wall concrete and exhibits minor chemical leaching. Clean water flowed steadily after the hole was drilled.	
8.75 to 10.3	Shaly Limestone	Lockport formation (Gasport Member), dark grey, fine-grained, dense, distinct sub-horizontal bedding cleavage, abundant fossils in strained state.	98.0%
10.3		End of Borehole.	

INSPECTOR: W. B. Pratt

LOGGED BY: M. Walia

APPROVED: J. Bradshaw

DATE: March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-3

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING				MUTUAL ANGLE
0.00 to 0.30	Fracture	Vert			x			x								
0.50	Fracture	Vert			x			x								
0.65	Fracture				x		x	x								
1.65 to 2.00	Fracture sub-parallel to the core axis	0°			x		x	x								
2.50	Fracture							x								
6.35	Fracture							x								
6.55	Bentonite seal															
8.10	Fracture, exhibits chemical weathering							x								
8.80	Core fractured exhibits chemical leaching of the cement							x								
10.30	End of Borehole															

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. P-4

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.

STARTED .M. Oct. 18 1971

FINISHED .M. Oct. 18 1971

METHOD OF DRILLING: SOIL

ROCK Diamond Drill

CASING DIAM.

CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ Station 54 + 14

~~DEPARTURE~~

BEARING

INITIAL DIP 10°

OTHER DIPS

ELEVATIONS: ~~DATUM~~ at wall face: 532.02 ft

DRILL PLATFORM

GROUND SURFACE

ROCK SURFACE

BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 5.0		Tunnel Wall Concrete: Due to the deflection in the drill-bearing the borehole was abandoned. Steel reinforcing: 0.5 feet 5.0 feet	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED J. J. Bradshaw

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. E-5

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 18 1971
FINISHED .M. Oct. 18 1971

METHOD OF DRILLING: SOIL CASING DIAM.
ROCK Diamond Drill Station CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ 54 + 12 ELEVATIONS: ~~DATUM~~ at wall face: 533.15 ft
~~DEPARTURE~~ DRILL PLATFORM
BEARING Due south GROUND SURFACE
INITIAL DIP 10^{o} ROCK SURFACE 531.60 ft
OTHER DIPS BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 5.60		Tunnel Wall Concrete: The concrete core from the tunnel wall is generally intact, well-compacted concrete with no evidence of honey-combing, segregation, or deterioration. Steel reinforcing - 0.60 foot Steel reinforcing - 5.25 feet Plastic Sheet - 5.30 feet	
5.60		Bentonite seal.	
5.62 to 9.1		Bulkhead Concrete: The core at the bentonite bulkhead interface is fractured locally and exhibits pronounced chemical leaching.	
9.1		Plastic seal.	
9.1 to 24.0	Shaly Limestone	Lockport Formation (Gasport Member). Dark grey, fine-grained, dense, distinct sub-horizontal cleavage, abundant fossils which show strain flow. The rock exhibits drag folds on a very small scale.	95.6%
24.0		End of Borehole.	

INSPECTOR W. B. Pratt

APPROVED T J Bradshaw

LOGGED BY M. Walia

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO E-5

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
1.20	Fracture							x					
3.40	Fracture							x					
5.60	Bentonite seal												
6.80	Fracture, minor chemical weathering							x					
8.50	Fracture, minor chemical weathering							x					
8.85	Fracture, minor chemical weathering							x					
9.60	Fracture						x	x					
17.0	Joint	5°			x	x							
18.2 to 18.5	Rock fractured due to close jointing												
20.2	Fracture							x					
22.0	Fracture							x					
22.9	Fracture							x					
24.0	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. E-6

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 18 1971
FINISHED .M. Oct. 18 1971

METHOD OF DRILLING: SOIL CASING DIAM.
ROCK Diamond Drill Station CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ 54 + 14 ELEVATIONS: ~~DATUM~~ at wall face: 533.05 ft
~~DEPARTURE~~ DRILL PLATFORM
BEARING Due south GROUND SURFACE
INITIAL DIP 10° ROCK SURFACE
OTHER DIPS BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.00 to 6.30		Tunnel Wall Concrete: The concrete core from the tunnel wall is generally intact. However it is fractured locally exhibiting very minor chemical leaching along the fractures.	
6.30		Bentonite seal.	
6.32 to 9.35		Bulkhead Concrete: The core at the bentonite bulkhead interface is relatively more fractured and chemically leached.	
9.35 to 12.00	Shaly Limestone	Lockport formation (Gasport Member). Dark grey, fine grained, dense, distinct bedding cleavage, abundant fossils in strained state.	
12.0		End of Borehole.	97.2%

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED [signature] Bradshaw

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO E-6

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	DESCRIPTION SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.0 to 0.6	Closed fracture	0°						x					
2.8 to 3.0	Core fractured, minor chemical weathering	Vari						x					
6.3	Bentonite seal												
6.7	Core fractured exhibits minor chemical leaching												
8.7	Fracture exhibits chemical leaching	Vari						x					
11.0	Joint	35°		x		x							
11.5	Fracture	Vari			x		x						
12.0	End of borehole												

Vari = Variable

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. P-7

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.
STARTED .M. Oct. 18 1971
FINISHED .M. Oct. 18 1971

METHOD OF DRILLING: SOIL
ROCK Diamond Drill

CASING DIAM.
CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ Station 54 + 16
~~DEPARTURE~~
BEARING Due south
INITIAL DIP 20°
OTHER DIPS

ELEVATIONS: ~~DATUM~~ at wall face: 531.95 ft
DRILL PLATFORM
GROUND SURFACE
ROCK SURFACE
BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.00 to 6.50		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured locally and exhibits minor chemical leaching. Steel reinforcing - 6.40 feet	
6.50		Bentonite seal.	
6.52 to 10.00		Bulkhead Concrete: The core at the bentonite bulk-head interface exhibits higher frequency of fractures per unit length and pronounced chemical leaching.	93.2%
10.00		End of Borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED H. J. Brosshaw (signature)

DATE March 1972

FORM NO. 91C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-7

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.0 to 0.5	Core fractured, minor chemical weathering	0°			x								
3.9 to 4.0	Core fractured, minor chemical weathering							x					
4.25	Fracture, minor chemical weathering							x					
5.10	Fracture, minor chemical weathering							x					
6.5	Bentonite seal												
6.75	Fracture, chemical weathering							x					
7.35	Fracture, chemical weathering							x					
8.20	Fracture, chemical weathering							x					
9.00	Fracture, minor chemical weathering							x					
10.00	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. P-8

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 18 1971

FINISHED .M. Oct. 18 1971

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ Station 54 + 18 ELEVATIONS: ~~DATUM~~ at wall face: 532.15 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 20^{o} ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 5.4		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured locally and exhibits minor chemical weathering and leaching confined to the fracture face. Steel reinforcing - 0.50 foot When drilled at 5.33-5.50 feet, water started to flow from drill hole P-7.	99.1%
5.4		End of Borehole.	

INSPECTOR W. B. Pratt

APPROVED J Bradshaw

LOGGED BY M. Walia

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-8

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.4 to 0.6	Fracture, runs subparallel to the core axis							x					
2.7	Fracture, exhibits minor chemical weathering	75°			x			x					
3.2	Fracture, chemical leaching, loss of cohesion in the concrete.							x					
3.9	Fracture, minor chemical leaching							x					
5.4	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. P-9

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 19 19 71

FINISHED .M. Oct. 19 19 71

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ Station 54 + 11 ELEVATIONS: ~~DATUM~~ at wall face: 532.26 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 20^{o} ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.2		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured locally and the chemical weathering is confined to the fracture zones only. Steel reinforcing - 0.9 foot 5.5 feet When drilled at 5.0 feet water started to flow from drill holes P-7 and P-8.	
6.2		Bentonite seal.	
6.22 to 7.8		Bulkhead Concrete: The core at the bentonite bulkhead interface exhibits pronounced chemical leaching whenever the core is fractured.	98.0%
7.8		End of Borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED [signature]

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-9

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.35 to 0.45	Core fractured, minor chemical weathering	20°			x	x							
2.9	Fracture, minor chemical weathering	80°			x	x							
3.3	Fracture, minor chemical weathering							x					
3.7 to 4.0	Core fractured, exhibits chemical leaching resulting in loss of cohesion												
5.1	Fracture exhibits chemical weathering	85°			x		x						
5.4	Fracture, chemical weathering							x					
5.6 to 6.0	Core fractured, chemical leaching							x					
6.2	Bentonite seal												
6.4	Core fractured, chemical leaching, slight loss of cohesion							x					
6.8	Fracture, chemical weathering							x					
7.8	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. P-10

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 19 19 71
FINISHED .M. Oct. 19 19 71

METHOD OF DRILLING: SOIL CASING DIAM.
ROCK Diamond Drill CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ Station 53 + 96 ELEVATIONS: ~~DATUM~~ at wall face: 534.16 ft
~~DEPARTURE~~ DRILL PLATFORM
BEARING Due south GROUND SURFACE
INITIAL DIP 10° ROCK SURFACE
OTHER DIPS BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 5.4		Tunnel Wall Concrete: The concrete core from the tunnel wall is badly fractured locally and exhibits chemical leaching. Chemical weathering and leaching is confined to the fracture zones only. Steel reinforcing - 0.50 foot 4.65 feet Substantial flow of water from the borehole. Water black in color flowed for 2 minutes before becoming clear. Strong odour of H_2S in water.	
5.4 to 6.4		Void.	84.3%
6.4		End of Borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED T J Bradshaw

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-10

SITE South Tunnel Wall SHEET NO. 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.20	Fracture, minor chemical leaching				x			x					
0.30	Fracture exhibits chemical weathering	80°			x			x					
0.80	Fracture, minor chemical weathering							x					
1.40	Fracture							x					
2.00 to 2.10	Core fractured, chemical leaching							x					
3.40	Fracture, minor chemical weathering							x					
3.65	Fracture				x			x					
3.90 to 4.00	Core fractured, chemical leaching												
5.0 to 5.4	Core fractured, pronounced chemical weathering				x			x					
5.4 to 6.4	Void												
6.4	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. P-11

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 19 19 71

FINISHED .M. Oct. 19 19 71

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Ax

LOCATION: ~~LATITUDE~~ Station 55 + 17 ELEVATIONS: ~~DATUM~~ at wall face: 527.0 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 10° ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.0		Tunnel Wall Concrete: The concrete core from the tunnel wall is generally intact and exhibits fracturing and chemical leaching locally only. Steel reinforcing - 5.0 feet Substantial flow of water from the borehole. Strong odour of H_2S in water flowing out.	98.9%
6.0		End of Borehole.	

INSPECTOR W. B. Pratt

APPROVED J Bradshaw

LOGGED BY M. Walia

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO P-11

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING					MUTUAL ANGLE
0.5	Fracture	5^{o}			x	x											
2.0 to 2.1	Core fractured, chemical leaching							x									
2.5	Fracture, minor chemical weathering				x	x											
3.2	Fracture, chemical weathering							x									
5.8	Fracture						x	x									
6.0	End of borehole																

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No TC-12

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 20 19 71

FINISHED .M. Oct. 20 19 71

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Ex

LOCATION: ~~LATITUDE~~ Station 54 + 66 ELEVATIONS: ~~DATUM~~ at wall face 530.36 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 5° ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 5.0		Tunnel Wall Concrete: During the core drilling no steel reinforcing was hit and no water leaks were observed from the borehole. The hole was used for installing thermocouples.	

INSPECTOR W. B. Pratt

APPROVED [signature]

LOGGED BY M. Walia

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario) — JOB No. P2499.02

PROJECT: Thorold Tunnel — HOLE No. TC -13

SITE: South Tunnel Wall — SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. — STARTED .M. Oct. 20 19 71 — FINISHED .M. Oct. 20 19 71

METHOD OF DRILLING: SOIL .... CASING DIAM. ....

ROCK: Diamond Drill — CORE DIAM.: Ex

LOCATION: ~~LATITUDE~~ XXXXXXX Station 54 + 27 — ELEVATIONS: ~~DATUM~~ XXXXX at wall face 532.71 ft

~~DEPARTURE~~ — DRILL PLATFORM

BEARING: Due south — GROUND SURFACE

INITIAL DIP: 5^{o} — ROCK SURFACE

OTHER DIPS — BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 5.0		Tunnel Wall Concrete: No water leaks from the bore-hole were observed during the drilling. The hole was used for installing thermocouples. Steel reinforcing: 0.5 feet	

INSPECTOR: W. B. Pratt

LOGGED BY: M. Walia

APPROVED: J H Bradshaw

DATE: March 1972

FORM NO. 91 C

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02
PROJECT Thorold Tunnel HOLE No. OC-14
SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd
STARTED .M. Oct. 20 1971
FINISHED .M. Oct. 21 1971

METHOD OF DRILLING:
SOIL
CASING DIAM. 6.0 in.
ROCK Diamond Drill
CORE DIAM. 5.776 in.

LOCATION: Station
~~LATITUDE~~ 54 + 35
~~DEPARTURE~~
BEARING Due south
INITIAL DIP 0°
OTHER DIPS

ELEVATIONS: ~~DATUM~~ at wall face: 532.97 ft
DRILL PLATFORM
GROUND SURFACE
ROCK SURFACE
BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.1		Tunnel Wall Concrete: The concrete core from the tunnel wall exhibits open fractures intersecting from 2.2 to 3.6 feet. The fractures are partly filled with calcareous and ferrous material.	
6.1		Bentonite seal.	
6.12 to 7.70		Bulkhead Concrete: The core at the bentonite bulkhead interface exhibits a void and pronounced chemical weathering. Vertical steel reinforcing - 0.4 foot. Horizontal steel reinforcing - 0.5 foot.	98.1%
7.7		End of Borehole.	

INSPECTOR W. B. Pratt
LOGGED BY M. Walia
APPROVED J. J. Bradshaw
DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED – NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario)
JOB NO P2499.02

PROJECT Thorold Tunnel
HOLE NO OC-14

SITE South Tunnel Wall
SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING+	STAINING++	BLEACHING	MUTUAL ANGLE
2.2 to 3.6	Conjugate set of fractures	25°								C	Gy Br		
6.1	Bentonite seal												
6.1 to 6.2	Void												
7.7	End of Borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. OC-15

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.

STARTED .M. Oct. 25 1971

FINISHED .M. Oct. 26 1971

METHOD OF DRILLING: SOIL

ROCK Diamond Drill

CASING DIAM. 6 in.

CORE DIAM. 5.776 in.

LOCATION: ~~LATITUDE~~ Station 54 + 83

~~DEPARTURE~~

BEARING Due south

INITIAL DIP 0^{o}

OTHER DIPS

ELEVATIONS: ~~DATUM~~ at wall face: 529.84 ft

DRILL PLATFORM

GROUND SURFACE

ROCK SURFACE

BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.25		Tunnel Wall Concrete: Concrete core from the tunnel wall is intact except for drilling breaks. Vertical steel reinforcing - 0.3 feet and 5.6 feet. Horizontal steel reinforcing - 0.4 feet or 5.4 feet.	99.5%
6.25		End of 6-inch Borehole.	

INSPECTOR W. Pratt

LOGGED BY M. Walia

APPROVED T.J. Bradshaw

DATE March 1972

FORM NO. 91-C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario)

JOB NO P2499.02

PROJECT Thorold Tunnel

HOLE NO OC-15

SITE South Tunnel Wall

SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING				MUTUAL ANGLE
6.0	Bentonite Seal															
6.25	End of 6-inch Borehole															

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario) — JOB No. P2499.02

PROJECT: Thorold Tunnel — HOLE No. C-16

SITE: South Tunnel Wall — SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. — STARTED .M. Oct. 26 1971 — FINISHED .M. Oct. 26 1971

METHOD OF DRILLING: SOIL — CASING DIAM.

ROCK: Diamond Drill — CORE DIAM. Nx

LOCATION: ~~LATITUDE~~ Station 54 + 14 — ELEVATIONS: ~~DATUM~~ at wall face: 533.6 ft

~~DEPARTURE~~ — DRILL PLATFORM

BEARING: Due south — GROUND SURFACE

INITIAL DIP: 0° — ROCK SURFACE

OTHER DIPS — BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.00 to 0.67		Tunnel Wall Concrete: The concrete core is fractured and exhibits minor chemical weathering and leaching along the fracture.	100%
0.67		End of Borehole.	

INSPECTOR: W. B. Pratt

LOGGED BY: M. Walia

APPROVED: J. Bradshaw

DATE: March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO C-16

SITE South Tunnel Wall SHEET NO. 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.0 to 0.3	Fracture subparallel to the core-axis, dips due-north	20° Max			x			x					
0.5 to 0.67	Fracture, dips due-south	15°			x			x					
0.67	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. C-17

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 25 1971

FINISHED .M. Oct. 25 1971

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Nx

LOCATION: ~~LATITUDE~~ Station 54 + 11 ELEVATIONS: ~~DATUM~~ at wall face: 534.95 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 5° ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.00 to 0.67		Tunnel Wall Concrete: The concrete core is fractured, chemical weathering is confined to the fracture faces only.	100%
0.67		End of Borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED J. Bradshaw

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO C-17

SITE South Tunnel Wall SHEET NO. 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	DESCRIPTION: SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING				MUTUAL ANGLE
0.00 to 0.67	Fracture, minor chemical weathering and leaching along the fracture, dips due-south	20°			x			x								
0.67	End of borehole															

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. C-18

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 25 19 71

FINISHED .M. Oct. 25 19 71

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Nx

LOCATION: ~~LATITUDE~~ Station 54 + 12 ELEVATIONS: ~~DATUM~~ at wall face: 534.55 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 2° ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.00 to 1.33		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured and exhibits chemical weathering and leaching along the fracture faces.	100%
1.33		End of Borehole.	

INSPECTOR W. B. Pratt

APPROVED T. J. Bradshaw

LOGGED BY M. Walia

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO C-18

SITE South Tunnel Wall SHEET NO. 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.0 to 0.50	Fracture, dips due-south, chemical weathering along the fracture faces	20°					x	x					
0.75 to 1.20	Fracture, dips due-south, chemical weathering along the fracture faces	20°					x	x					
1.33	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. C-19

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 26 1971
FINISHED .M. Oct. 26 1971

METHOD OF DRILLING: SOIL CASING DIAM.
ROCK Diamond Drill CORE DIAM. Nx

LOCATION: Station 54 + 11 ELEVATIONS: Datum at wall face: 534.31 ft
DEPARTURE DRILL PLATFORM
BEARING Due south GROUND SURFACE
INITIAL DIP 4^{o} ROCK SURFACE
OTHER DIPS BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.0		Tunnel Wall Concrete: The concrete core from the tunnel wall is generally intact, well compacted, with no evidence of deterioration up to the bentonite seal.	
6.0		Bentonite seal.	
6.02 to 8.0		Bulkhead Concrete: The core at the bentonite bulkhead interface is fractured and exhibits chemical leaching locally.	95.0%
8.0		End of Borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED J.J. Bradshaw

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO C-19

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	DESCRIPTION: SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING					MUTUAL ANGLE
3.0 to 3.5	Fracture, dips due-south, minor chemical weathering	20°			x		x	x									
6.0	Bentonite seal																
6.0 to 7.1	Fracture, subparallel to the core axis, chemical weathering and leaching along the fracture							x									
8.0	End of borehole																

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA
DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario) — JOB No. P2499.02

PROJECT: Thorold Tunnel — HOLE No. D-20

SITE: South Tunnel Wall — SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. — STARTED .M. Oct. 26 19 71 — FINISHED .M. Oct. 26 19 71

METHOD OF DRILLING: SOIL — CASING DIAM.

ROCK: Diamond Drill Station — CORE DIAM.

LOCATION: ~~LATITUDE~~ 54 + 58 — ELEVATIONS: ~~DATUM~~ at wall face: 533.91 ft

~~DEPARTURE~~ — DRILL PLATFORM

BEARING: South — GROUND SURFACE

INITIAL DIP: 26° (above horizontal) — ROCK SURFACE 538.70 ft

OTHER DIPS — BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.6		Tunnel Wall Concrete: The concrete core is fractured locally and the chemical weathering and leaching is confined to the fracture faces.	
6.6		Bentonite Seal.	
6.62 to 9.0		Bulkhead Concrete: The concrete core at the bentonite bulkhead interface is badly fractured and the fractures are partly filled with calcareous material. Substantial flow of water from the drill hole was observed.	
9.0 to 16.0	Dolomite	Lockport formation (Goat Island/Gasport Member). Medium grey, fine grained, massive, dense, shaly locally, fractured occasionally.	95.2%
16.0		End of Borehole.	

INSPECTOR

LOGGED BY

APPROVED T J Bradshaw

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO D-20

SITE South Tunnel Wall SHEET NO 1 OF 2

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING				MUTUAL ANGLE
0.0 to 0.4	Fracture, minor chemical leaching along the fracture faces.	15°						x								
1.5 to 1.9	Fracture, exhibits chemical leaching.							x								
2.0 to 2.1	Fracture, exhibits chemical leaching.							x								
2.4	Fracture*							x								
2.8	Fracture, chemical leaching, partly filled with clay.							x								
4.65	Fracture*							x								
6.60	Bentonite Seal* (1/4-inch thick) slightly contorted.															
8.1 to 8.2	Fracture exhibits chemical leaching.							x								
9.0 to 11.0	Rock badly fractured, filled with calcareous material locally.							x	C	Gy						
11.8	Joint, partly filled with clay.	60°		x		x			C	Gy						
12.6	Joint, partly filled with clay.			x		x			C	Gy						

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO D-20

SITE South Tunnel Wall andSHEET NO 2 OF 2

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
13.70 to 14.00	Rock Fractured							x					
14.10	Fracture partly filled with clay.							x					
14.20	Fracture, partly filled with shaly material.							x					
14.55	Fracture, partly filled with shaly material.							x					
14.95	Joint, partly filled with clay.	60°		x		x				CGy			
15.10	Joint, partly filled with clay.	60°		x		x				CGy			
16.00	End of Borehole												
	* Television Camera Study.												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario) — JOB No. P2499.02

PROJECT: Thorold Tunnel — HOLE No. D-21

SITE: South Tunnel Wall — SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.

STARTED .M. Oct. 26 19 71

FINISHED .M. Oct. 27 19 71

METHOD OF DRILLING:
- SOIL — CASING DIAM.
- ROCK: Diamond Drill — CORE DIAM. Nx

LOCATION:
- ~~LATITUDE~~ Station 54 + 28
- ~~DEPARTURE~~
- BEARING: South
- INITIAL DIP: 30° (above horizontal)
- OTHER DIPS

ELEVATIONS: ~~DATUM~~ at wall face: 536.14 ft
- DRILL PLATFORM
- GROUND SURFACE
- ROCK SURFACE
- BOTTOM OF HOLE
- WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.75		Tunnel Wall Concrete: The tunnel wall concrete core is intact. Minor flow of water from the hole after the completion of drilling. Steel reinforcing: 0.40 feet 6.15 feet	
6.75		Bentonite Seal	
6.77 to 11.4		Bulkhead Concrete: The core at the bentonite bulkhead interface is fractured locally.	
11.4		Plastic Seal	
11.4 to 16.0	Dolomite	Lockport Formation (Goat Island/Gasport member). Light grey, fine-grained rock, dense, fractured and jointed locally. Shaly and and silty along the fractures.	
16.0		End of Borehole.	

INSPECTOR: W. B. Pratt

LOGGED BY: M. Walia

APPROVED: J. Bradshaw

DATE: March 1972

FORM NO. 91C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO D-21

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	DESCRIPTION: SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING			MUTUAL ANGLE
2.0	Core fracture							x							
6.75	Bentonite seal slightly contorted														
7.30	Core fractured							x							
11.4	Plastic seal														
13.9	Joint	60°		x		x									
14.2	Joint, partly filled with calcareous material			x		x									
14.5	Joint, exhibits minor chemical weathering	60°		x		x									
14.95	Joint, exhibits minor chemical weathering	60°		x		x									
15.2	Joint, partly filled with silt	60°		x		x									
15.5 to 16.0	Rock fractured							x							
16.0	End of Borehole														

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. E-22

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 27 1971

FINISHED .M. Oct. 27 1971

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Nx

LOCATION: ~~LATITUDE~~ Station 54 + 16 ELEVATIONS: ~~DATUM~~ at wall face: 532.91 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING Due south GROUND SURFACE

INITIAL DIP 10° ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.7		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured locally. The fracture faces do not exhibit chemical weathering or leaching of the concrete mix.	
6.7		Bentonite Seal.	
6.72 to 8.9		Bulkhead Concrete: Core fractured.	
8.9 to 16.0	Shaly Limestone	Lockport formation (Gasport Member). Dark grey, fine grained, dense, distinct sub-horizontal cleavage, abundant fossils in strained state. Core badly fractured locally (15.0 ft to 15.7 ft).	
16.0		End of Borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED T J Bradshaw

DATE March 1972

FORM NO. 91-C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO E-22

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	DESCRIPTION													MUTUAL ANGLE
			SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING				
2.2	Fracture exhibits minor chemical weathering.	70°			x			x								
2.6	Fracture.							x								
4.5	Fracture.						x	x								
5.2	Fracture.	80°			x			x								
5.3	Fracture.	80°			x			x								
6.7	Bentonite Seal.															
7.9 to 8.1	Fracture.							x								
15.0 to 15.7	B. Joint	15°		x		x										
16.0	End of Borehole.															

B. Joint = Bedding Joint

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. OC-23

SITE Road Slab - South Side of South Tube SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Oct. 28 1971

FINISHED .M. Oct. 28 1971

METHOD OF DRILLING: SOIL

CASING DIAM. 6.0 in.

ROCK Diamond Drill

CORE DIAM. 5.776 in.

LOCATION: ~~LATITUDE~~ Station 54 + 18

~~DEPARTURE~~

BEARING

INITIAL DIP

OTHER DIPS

ELEVATIONS: ~~DATUM~~

DRILL PLATFORM

GROUND SURFACE 526.34 ft

ROCK SURFACE

BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 5.5		Road Slab Concrete: The core from the road slab is intact. Steel reinforcing - 0.5 ft 5.5 ft	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED [signature]

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel
Tunnel Roof Strut HOLE No. OC-24

SITE South side of west service building SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Nov. 1 19 71
FINISHED .M. Nov. 1 19 71

METHOD OF DRILLING: SOIL CASING DIAM. 6.0 in
ROCK Diamond Drill CORE DIAM. 5.776 in

LOCATION: ~~LATITUDE~~ Station 54 + 64 ELEVATIONS: ~~DATUM~~ at top of roof strut: 549.83 ft
~~DEPARTURE~~ DRILL PLATFORM
BEARING Vertical GROUND SURFACE
INITIAL DIP 90° ROCK SURFACE
OTHER DIPS BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 2.85		Roof Strut Concrete: Due to the deflection in the drill-bearing the borehole was abandoned and was used for installing thermocouple.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED [signature]

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario) — JOB No. P2499.02

PROJECT: Thorold Tunnel Tunnel Roof Strut — HOLE No. OC-25

SITE: South Side of West Service Building — SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.

STARTED .M. Nov. 2 1971

FINISHED .M. Nov. 2 1971

METHOD OF DRILLING: SOIL — CASING DIAM. 6.0 in

ROCK: Diamond Drill — CORE DIAM. 5.776 in

LOCATION: ~~LATITUDE~~ Station 54 + 64 — ELEVATIONS: ~~DATUM~~ at the roof strut: 549.83 ft

~~DEPARTURE~~ — DRILL PLATFORM

BEARING: Vertical — GROUND SURFACE

INITIAL DIP: 90^{o} — ROCK SURFACE

OTHER DIPS — BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 3.0		Roof Strut Concrete: The core from the roof strut is generally intact. The core is fractured at the following depths where the steel reinforcing is encountered. Steel reinforcing - 0.4 ft 2.7 ft	

INSPECTOR: W. B. Pratt

LOGGED BY: M. Walia

APPROVED: [signature]

DATE: March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO OC-25

SITE Roof Strut SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	DESCRIPTION: SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.4	Fracture	90°			x								
2.7	Fracture	90°			x								

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario)
JOB No. P2499.02

PROJECT: Thorold Tunnel
Tunnel Roof Strut
HOLE No. OC-26

SITE: South Side of West Service Building
SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.
STARTED .M. Nov. 2 1971
FINISHED .M. Nov. 2 1971

METHOD OF DRILLING:
SOIL
CASING DIAM. 6.0 in.
ROCK Diamond Drill
CORE DIAM. 5.776 in.

LOCATION:
~~LATITUDE~~ Station 54 + 64
~~DEPARTURE~~
BEARING Vertical
INITIAL DIP 90°
OTHER DIPS

ELEVATIONS:
~~DATUM~~ at the roof strut: 549.83 ft
DRILL PLATFORM
GROUND SURFACE
ROCK SURFACE
BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 3.0		Roof Strut Concrete: The core from the roof strut is generally intact. The core is fractured at the following depths where the steel reinforcing is encountered. Steel reinforcing - 0.4 ft, 2.7 ft	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED [signature]

DATE March 1972

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario)

JOB NO P2499.02

PROJECT Thorold Tunnel

HOLE NO OC-26

SITE Roof Strut

SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING				MUTUAL ANGLE
0.4	Fracture	90°			x											
2.7	Fracture	90°			x											

C = CARBONATE   H = HEMATITE   K = CHLORITE   ++ Br = BROWN   + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. OC-27

SITE North Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Nov. 3 19 71

FINISHED .M. Nov. 3 19 71

METHOD OF DRILLING: SOIL CASING DIAM.

ROCK Diamond Drill CORE DIAM. Ex

LOCATION: ~~LATITUDE~~ Station 54 + 34 ELEVATIONS: ~~DATUM~~ at wall face: 533.0 ft

~~DEPARTURE~~ DRILL PLATFORM

BEARING GROUND SURFACE

INITIAL DIP ROCK SURFACE

OTHER DIPS BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.0		Tunnel Wall Concrete: Overcoring of Ex-hole was abandoned due to deflection in the bearing of the borehole.	

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED [signature]

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario) — JOB No. P2499.02

PROJECT: Thorold Tunnel — HOLE No. OC-28

SITE: North Tunnel Wall — SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd.

STARTED .M. Nov. 4 19 71

FINISHED .M. Nov. 4 19 71

METHOD OF DRILLING:
- SOIL:
- ROCK: Diamond Drill

CASING DIAM. 6 in.

CORE DIAM. 5.776 in.

LOCATION:
- ~~LATITUDE~~ Station 54 + 23
- ~~DEPARTURE~~
- BEARING: Due north
- INITIAL DIP: 0°
- OTHER DIPS:

ELEVATIONS: ~~DATUM~~ at wall face: 533.43 ft
- DRILL PLATFORM
- GROUND SURFACE
- ROCK SURFACE
- BOTTOM OF HOLE
- WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.10		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured locally and minor chemical weathering is confined to the fracture faces only. Tunnel wall near the bulkhead interface exhibits two irregular fractures.	
6.10 to 6.20		Tentest recovered - approximate thickness 0.75 inch. Vertical steel reinforcing - 0.4 foot. Horizontal steel reinforcing - 0.5 foot and 5.6 feet.	
6.20 to 8.5		Bulkhead Concrete: The core exhibits sub-horizontal fractures. Substantial flow of water was observed during the borehole drilling.	
8.5		End of 6-inch Borehole.	

INSPECTOR: W. Pratt

LOGGED BY: M. Walia

APPROVED: T J Bradshaw

DATE: March 1972

FORM NO. 91-C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO OC-28

SITE North Tunnel Wall SHEET NO 1 OF 2

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
6.1	Fracture	90°			x								
6.2 to 6.5	Two fractures	0°			x			x					
		70°			x			x		C	Gy		
8.5	End of 6-inch Borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. B-29

SITE North Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Nov. 4 1971
FINISHED .M. Nov. 4 1971

METHOD OF DRILLING: SOIL CASING DIAM.
ROCK Diamond Drill CORE DIAM. Nx

LOCATION: ~~LATITUDE~~ Station 54 + 57 ELEVATIONS: ~~DATUM~~ at wall face: 531.41 ft
~~DEPARTURE~~ DRILL PLATFORM
BEARING Due north GROUND SURFACE
INITIAL DIP 0° ROCK SURFACE
OTHER DIPS BOTTOM OF HOLE
WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.2		Tunnel Wall Concrete: The concrete core from the tunnel wall generally is intact, well compacted, and does not show any evidence of deterioration. The core however is fractured locally and exhibits very minor chemical weathering along the fractures.	
6.2		Tentest recovered.	
6.3		End of Borehole.	96.2%

INSPECTOR W. B. Pratt

APPROVED TJ Bradshaw

LOGGED BY M. Walia

DATE March 1972

FORM NO. 91 C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO B-29

SITE North Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.35	Fracture	90°			x								
1.55	Fracture				x			x					
3.85	Fracture				x			x					
5.80	Fracture, minor chemical weathering				x		x	x					
6.3	End of borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++Br = BROWN +Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Transportation and Communications (Ontario) JOB No. P2499.02

PROJECT Thorold Tunnel HOLE No. B-30

SITE South Tunnel Wall SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. STARTED .M. Nov. 5 19 71

FINISHED .M. Nov. 5 19 71

METHOD OF DRILLING: SOIL

CASING DIAM.

ROCK Diamond Drill

CORE DIAM. Nx

LOCATION: Station 74 + 08

~~LATITUDE~~

~~DEPARTURE~~

BEARING Due south

INITIAL DIP 0^{o}

OTHER DIPS

ELEVATIONS: ~~DATUM~~ at wall face: 538.40 ft

DRILL PLATFORM

GROUND SURFACE

ROCK SURFACE

BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.0		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured locally and exhibits minor chemical weathering and leaching of the concrete mix, confined to the fracture faces.	
6.0		Bentonite Seal.	
6.02		End of Borehole.	98.6%

INSPECTOR W. B. Pratt

LOGGED BY M. Walia

APPROVED T J Bradshaw

DATE March 1972

FORM NO. 91-C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO B-30

SITE South Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.7 to 0.8	Fracture						x	x					
3.1 to 3.4	Core Fractured, chemical weathering.				x		x	x					
6.0	Fracture, minor chemical leaching.	90°			x								
6.0	Bentonite Seal												
6.02	End of Borehole												

C = CARBONATE H = HEMATITE K = CHLORITE ++ Br = BROWN + Gy = GRAY

H. G. ACRES LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Transportation and Communications (Ontario) — JOB No. P2499.02

PROJECT: Thorold Tunnel — HOLE No. B-31

SITE: South Tunnel Wall — SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Ltd. — STARTED .M. Nov. 5 1971 — FINISHED .M. Nov. 5 1971

METHOD OF DRILLING: SOIL — CASING DIAM.

ROCK: Diamond Drill — CORE DIAM. Nx

LOCATION: XXXXXXX Station 74 + 08 — ELEVATIONS: ~~DATUM~~ at wall face: 538.62 ft

XXXXXXXX — DRILL PLATFORM

BEARING: Due north — GROUND SURFACE

INITIAL DIP: 0° — ROCK SURFACE

OTHER DIPS — BOTTOM OF HOLE

WATER TABLE

DEPTH Feet	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0.0 to 6.0		Tunnel Wall Concrete: The concrete core from the tunnel wall is fractured locally and the fracture faces exhibit minor chemical weathering and leaching of the cement.	
6.0		Bentonite seal.	
6.02		End of Borehole.	97.1%

INSPECTOR: W. B. Pratt

LOGGED BY: M. Walia

APPROVED: [signature]

DATE: March 1972

FORM NO. 91-C

H. G. ACRES LIMITED - NIAGARA FALLS

DRILLING REPORT
(CORE DETAILS)

CLIENT Department of Transportation and Communications (Ontario) JOB NO P2499.02

PROJECT Thorold Tunnel HOLE NO B-31

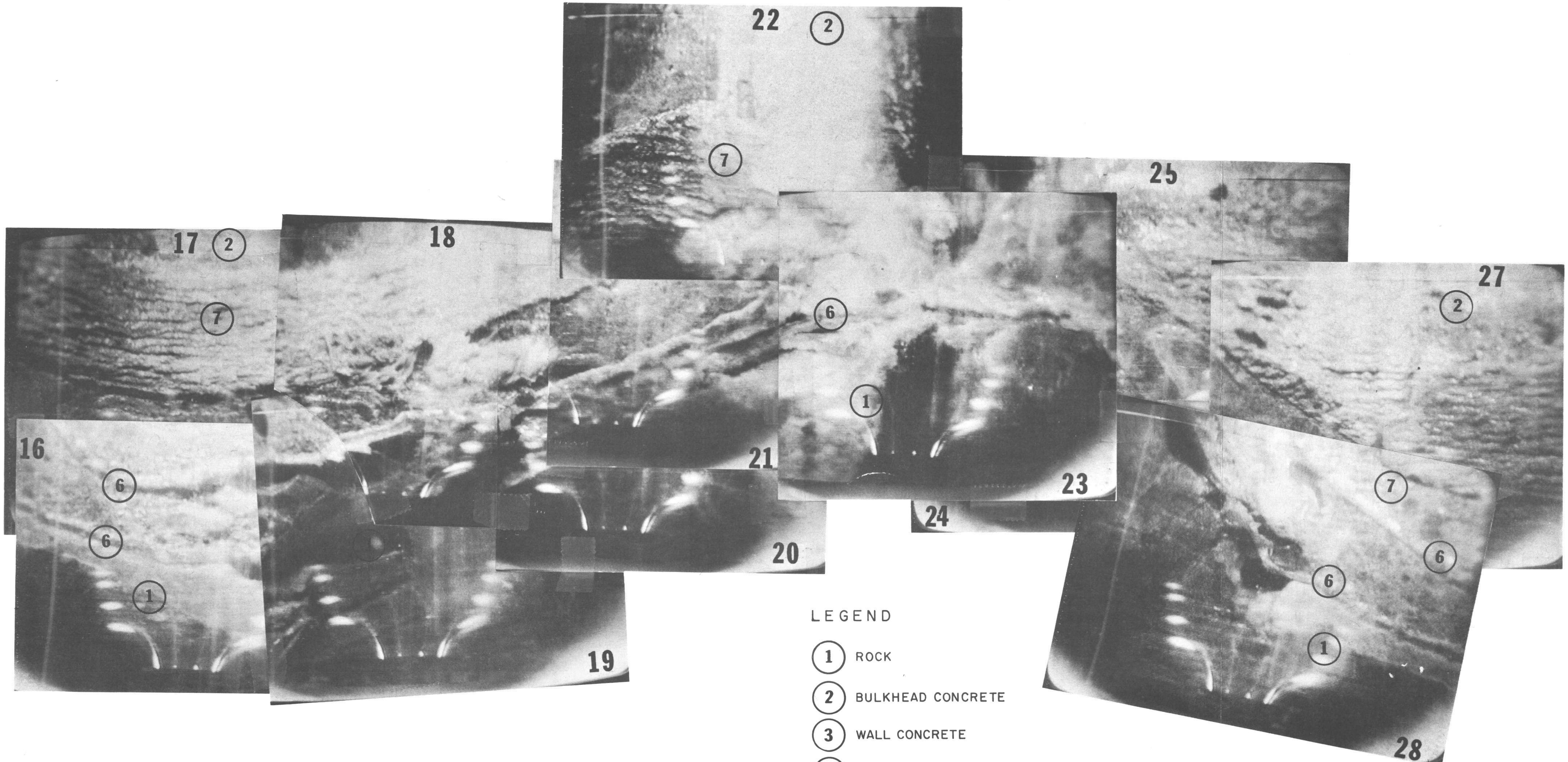
SITE North Tunnel Wall SHEET NO 1 OF 1

DEPTH (FT.)	DISCONTINUITY (JOINT, FAULT, BEDDING PLANES, CLEAVAGE, LINEATION)	ANGLE WITH CORE AXIS	SLICK	SMOOTH	ROUGH	PLANE	CURVED	IRREGULAR	SLICKENSIDED	FILLING +	STAINING ++	BLEACHING	MUTUAL ANGLE
0.3 to 0.4	Core Fractured, chemical weathering.				x		x	x					
1.2	Fracture, minor chemical weathering.							x					
2.7	Fracture, minor chemical weathering.				x			x					
3.5	Fracture, minor chemical leaching.	90°			x			x					
4.3	Fracture, chemical weathering.	90°			x			x					
4.9	Fracture, minor chemical weathering.	90°			x			x					
6.0	Bentonite Seal												
6.02	End of Borehole.												

C = CARBONATE H = HEMATITE K = CHLORITE ++Br = BROWN +Gy = GRAY

LIST OF TELEVISION BOREHOLE CAMERA OBSERVATIONS

Number	Title
1	Rock Interface of D20 Inclination + 26^{o}
2	Bentonite Panel of D20 Inclination + 26^{o}
3	Observed Cracks in D20 Inclination + 26^{o}
4	Rock Interface of D21 Inclination + 30^{o}
5	Bentonite Panel of D21 Inclination + 30^{o}
6	Bentonite Panel of OC14 Inclination 0^{o}
7	Observed Cracks in OC14
8	Tentest - Bentonite Panels OC28 Inclination 0^{o}
9	Observed Cracks in OC28
10	Observed Cracks in OC28
11	Scale Factors for Borehole Observations



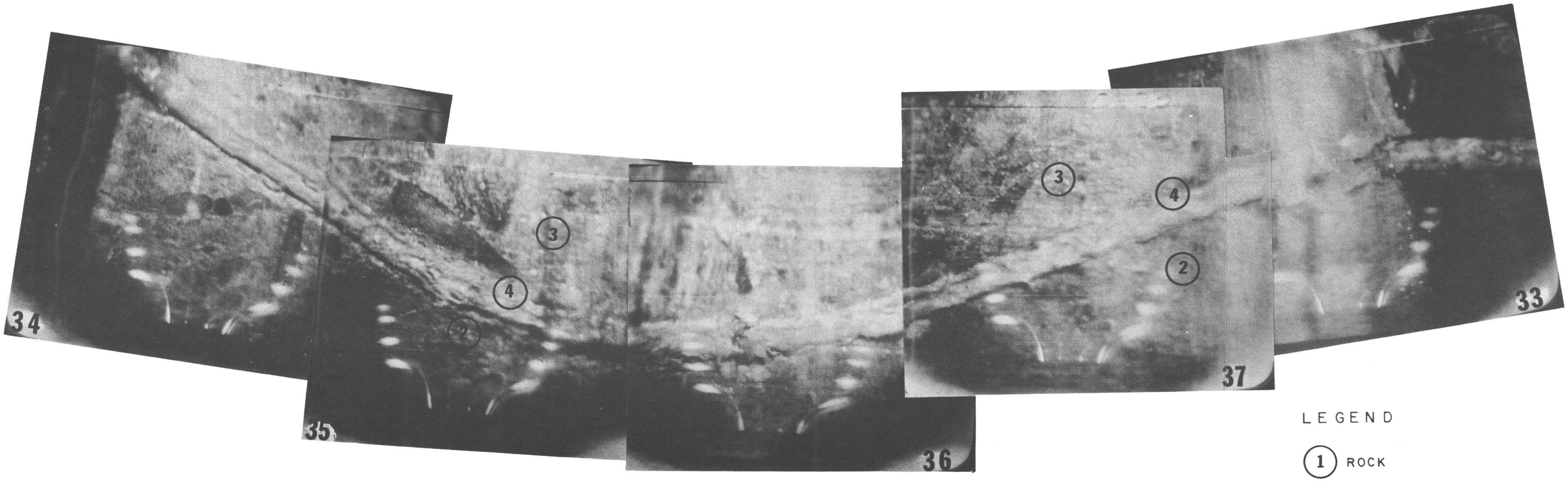
LEGEND

1. ROCK
2. BULKHEAD CONCRETE
3. WALL CONCRETE
4. BENTONITE
5. TENTEST
6. PLASTIC
7. WOOD

ROCK INTERFACE OF D 20
INCLINATION + 26°

PHOTO I

ACRES



LEGEND

1 ROCK

2 BULKHEAD CONCRETE

3 WALL CONCRETE

4 BENTONITE

5 TENTEST

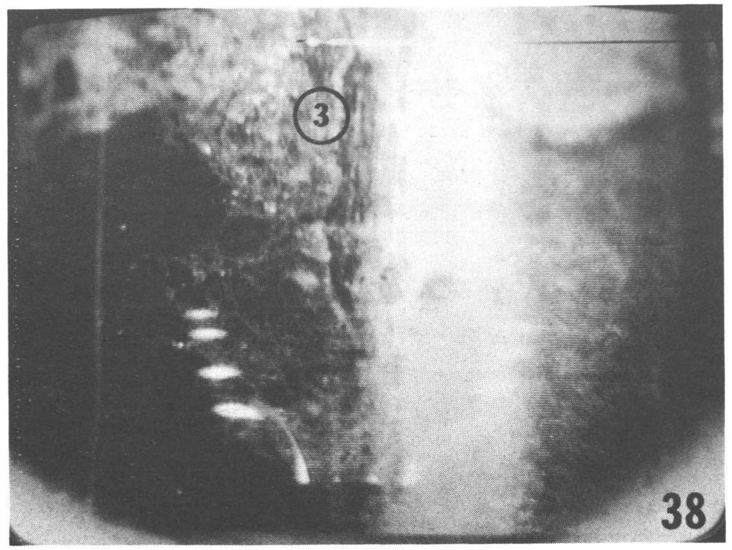
6 PLASTIC

7 WOOD

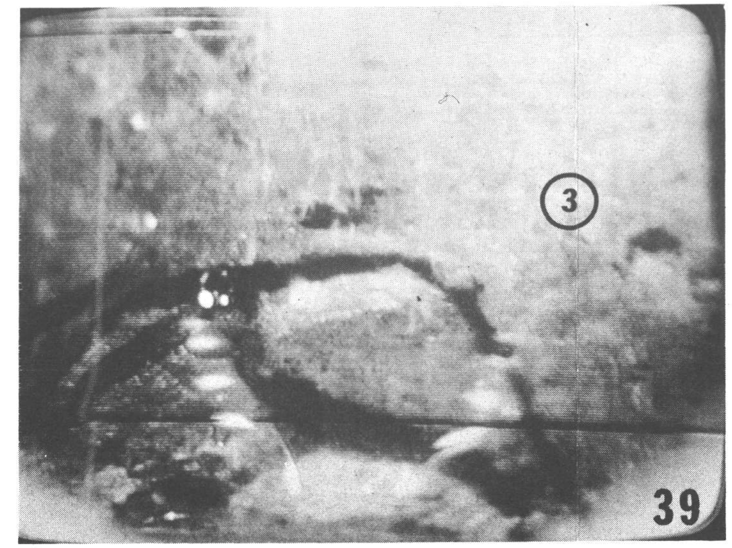
BENTONITE PANEL OF D20
INCLINATION + 26°

PHOTO 2

ACRES



CRACK I IN D20: 4'-8"



CRACK II IN D20 AT SOFFIT: 18"

LEGEND

1 ROCK

2 BULKHEAD CONCRETE

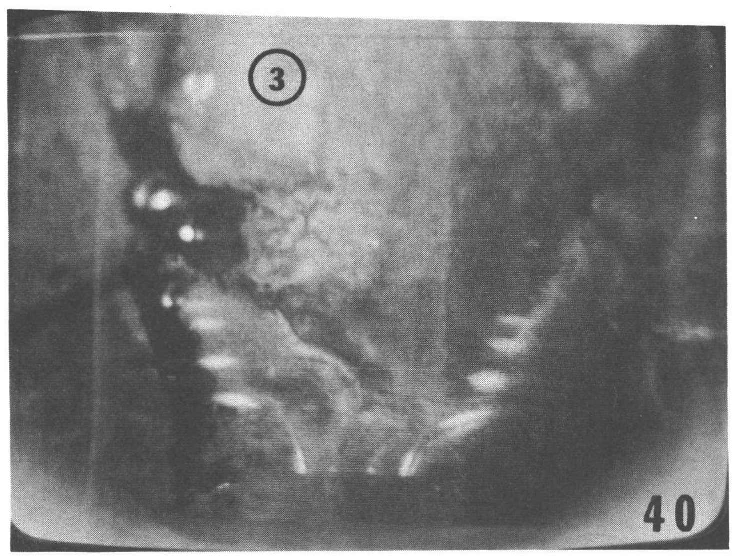
3 WALL CONCRETE

4 BENTONITE

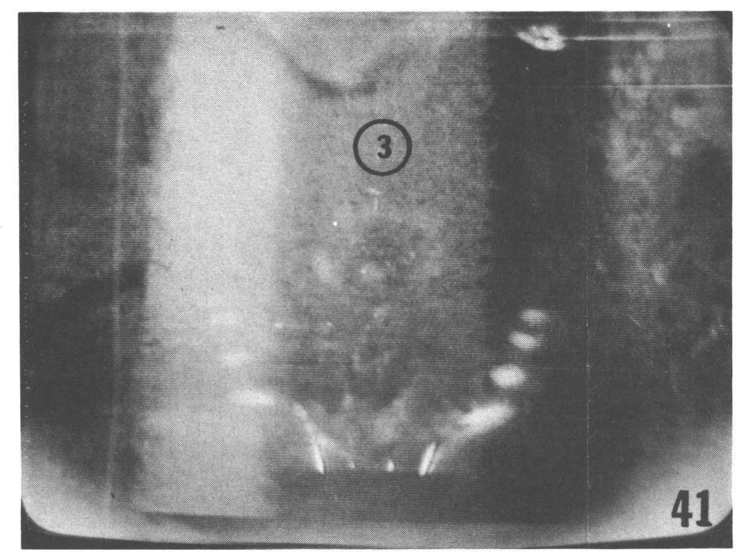
5 TENTEST

6 PLASTIC

7 WOOD



CRACK II IN D20 AT INVERT: 2'-5"



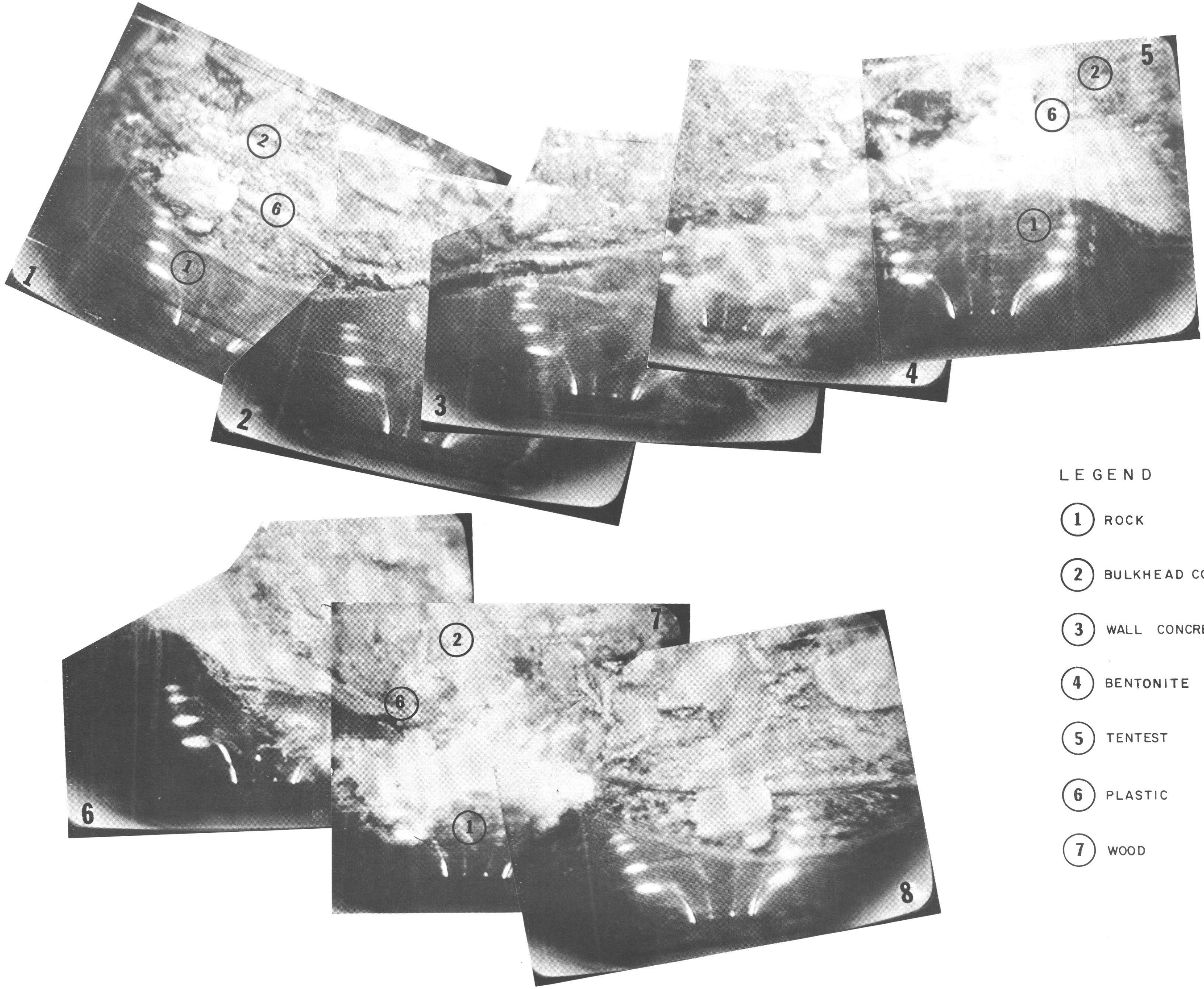
SEDIMENTATION IN D 20

OBSERVED CRACKS IN D20

INCLINATION +26°

PHOTO 3

ACRES



LEGEND

1 ROCK

2 BULKHEAD CONCRETE

3 WALL CONCRETE

4 BENTONITE

5 TENTEST

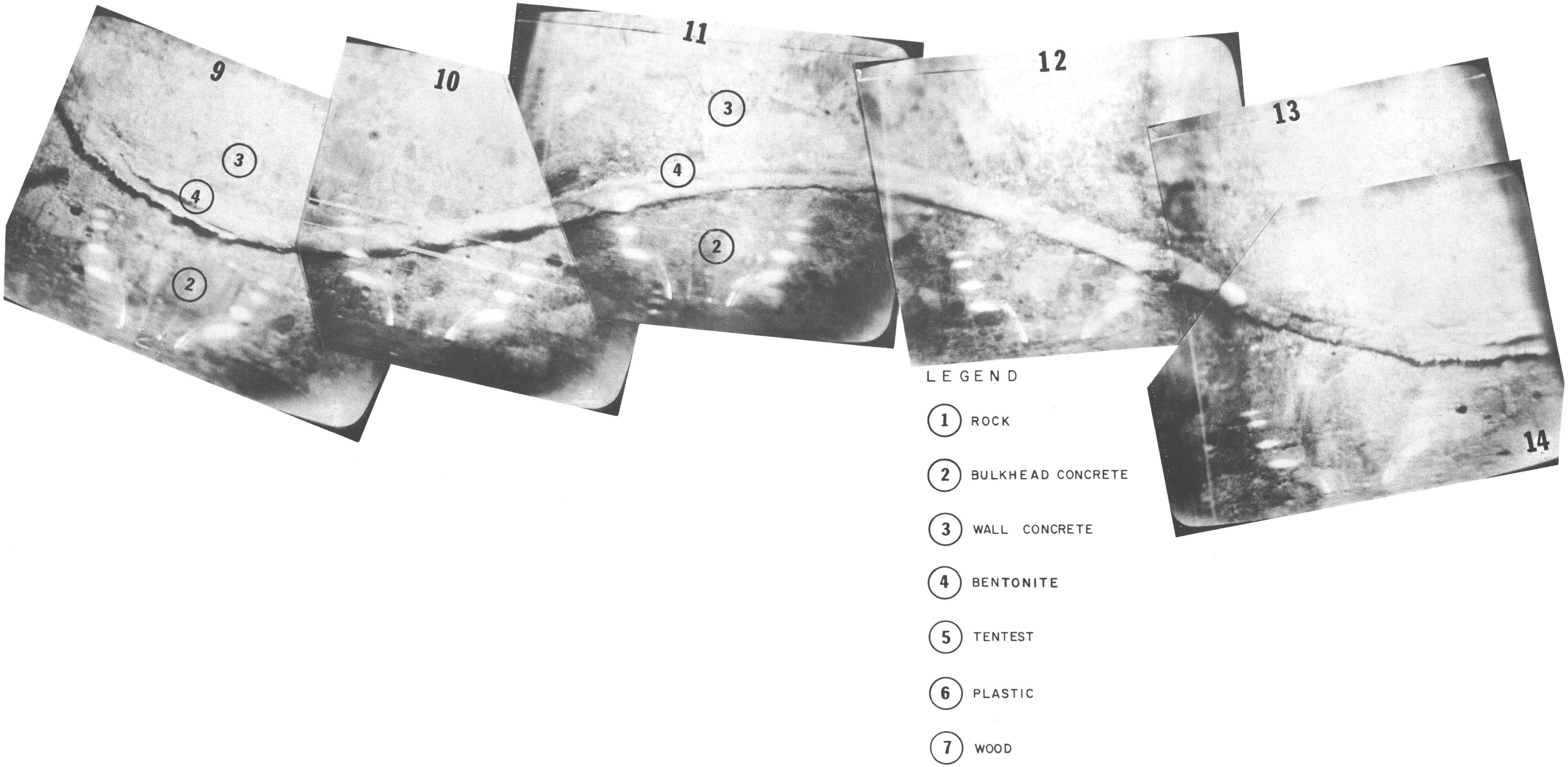
6 PLASTIC

7 WOOD

ROCK INTERFACE OF D21
INCLINATION +30°

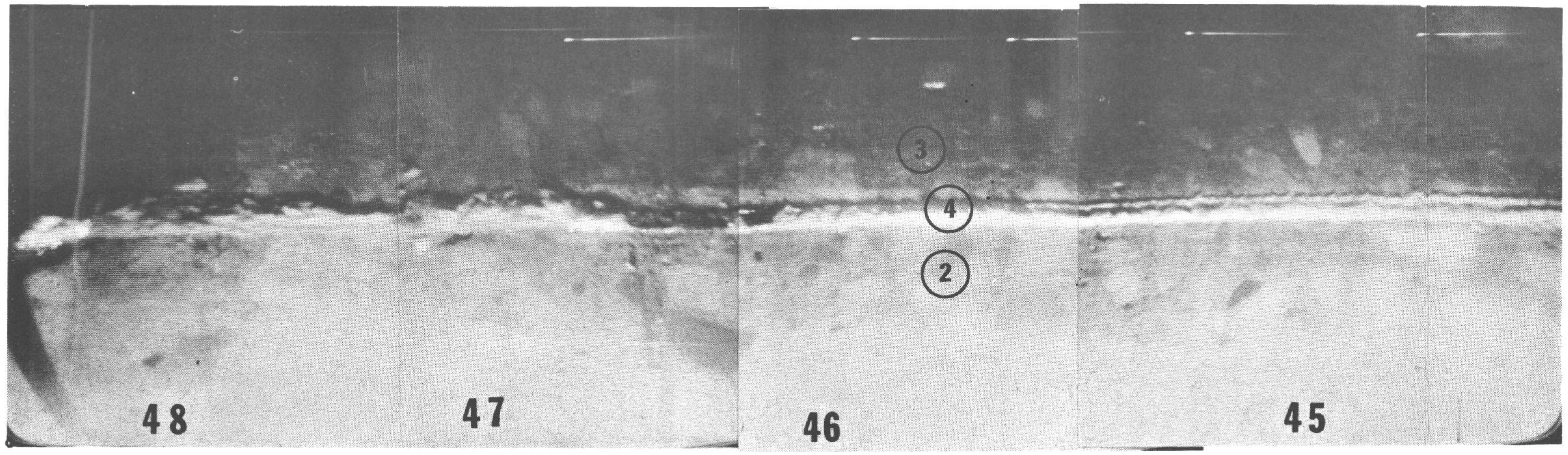
PHOTO 4

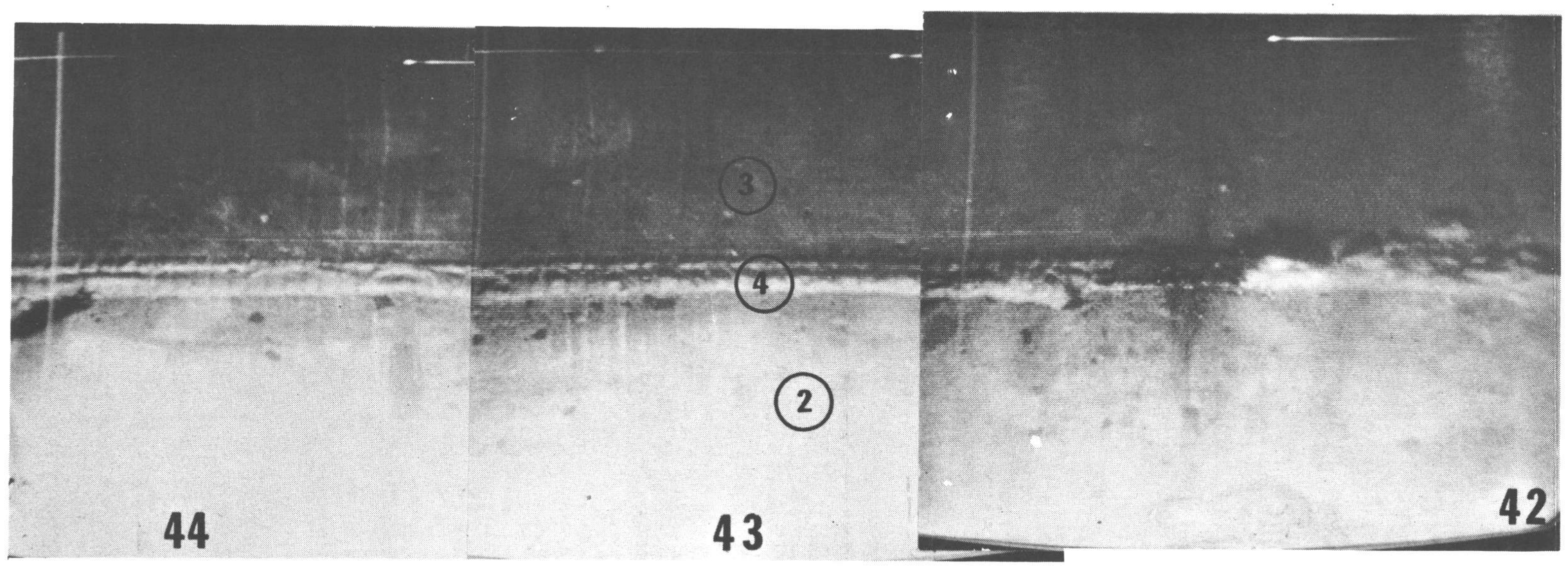
ACRES



BENTONITE PANEL OF D21
INCLINATION + 30°

PHOTO 5





LEGEND

1 ROCK

2 BULKHEAD CONCRETE

3 WALL CONCRETE

4 BENTONITE

5 TENTEST

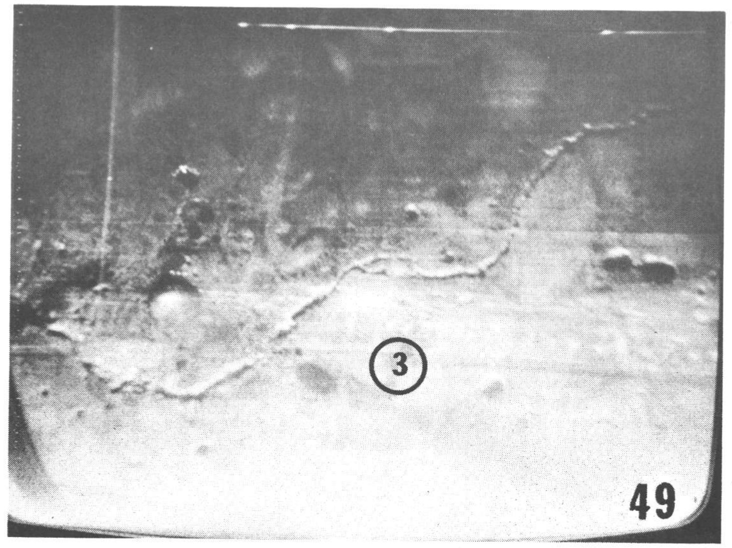
6 PLASTIC

7 WOOD

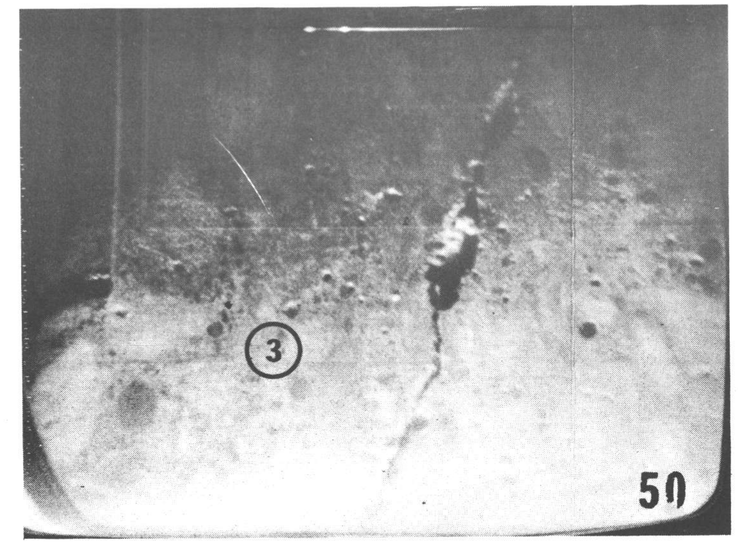
BENTONITE PANEL OF OC14
INCLINATION 0°

PHOTO 6

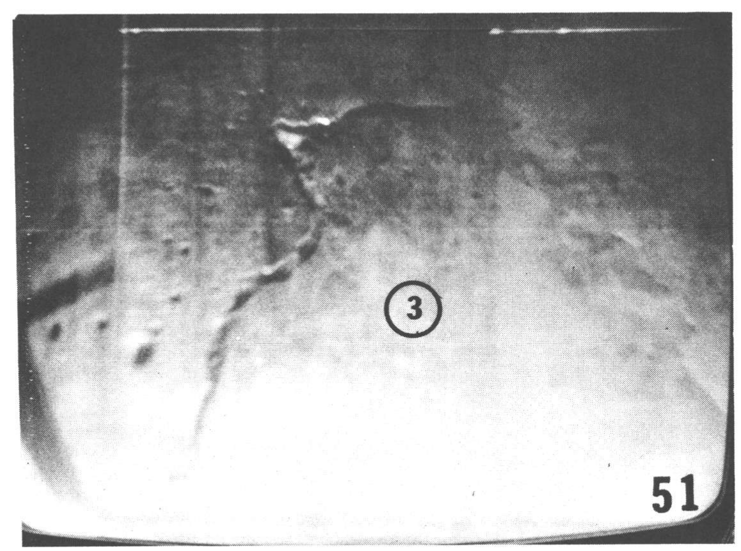
ACRES



CRACK I AT SOFFIT OF OC 14 : 3'-9"



CRACK I GENERAL OF OC 14



CRACK I AT INVERT OF 14 : 2'-10"

LEGEND

1 ROCK

2 BULKHEAD CONCRETE

3 WALL CONCRETE

4 BENTONITE

5 TENTEST

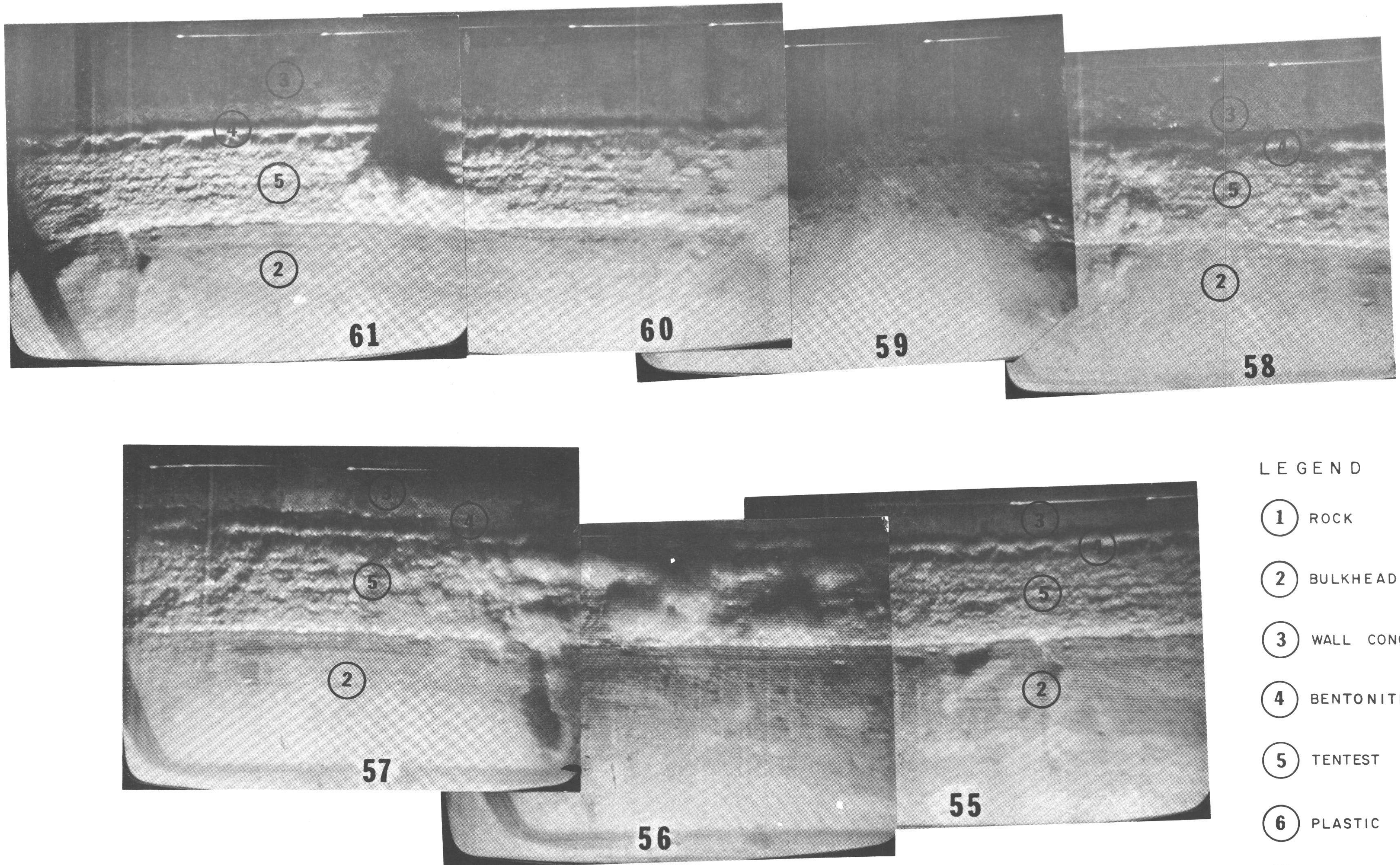
6 PLASTIC

7 WOOD

OBSERVED CRACKS IN OC 14

PHOTO 7

ACRES



LEGEND

1 ROCK

2 BULKHEAD CONCRETE

3 WALL CONCRETE

4 BENTONITE

5 TENTEST

6 PLASTIC

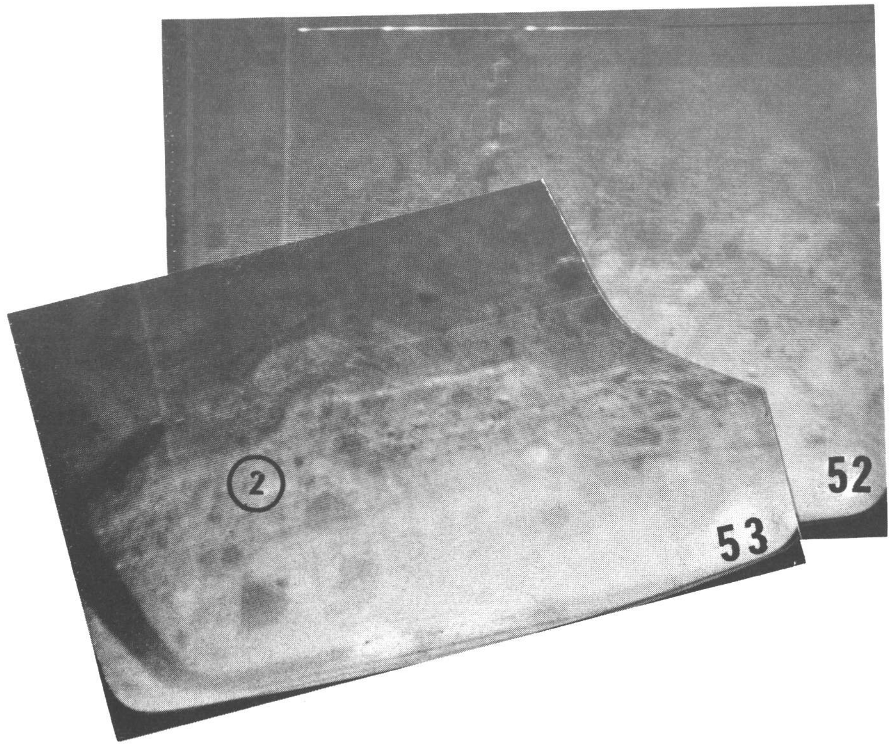
7 WOOD

TENTEST - BENTONITE PANELS OC 28

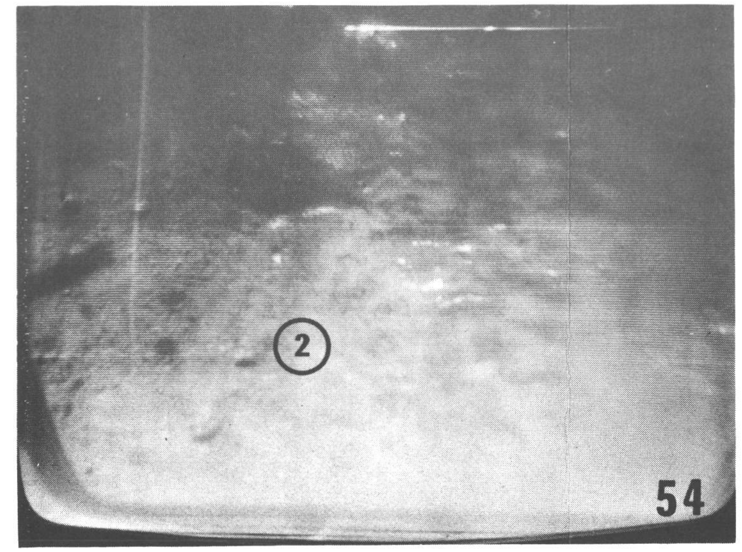
INCLINATION 0°

PHOTO 8

ACRES



CRACK JUNCTION OC 28 : 7'-10"



INVERT OF OC 28 : 7'-10"

LEGEND

1 ROCK

2 BULKHEAD CONCRETE

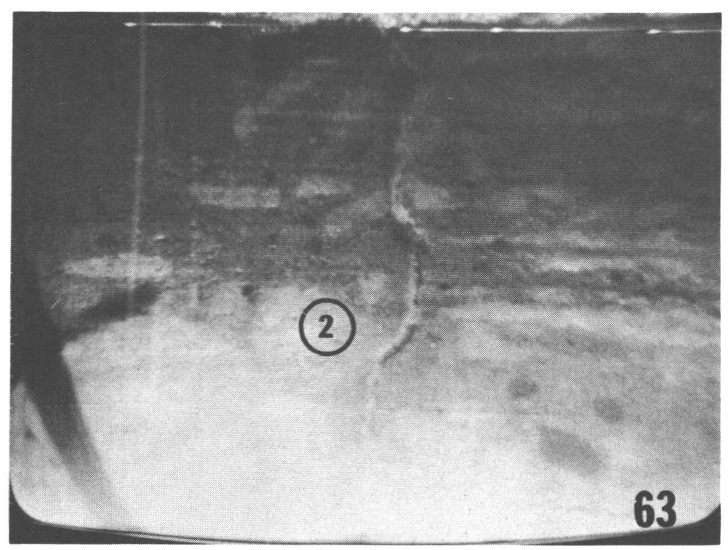
3 WALL CONCRETE

4 BENTONITE

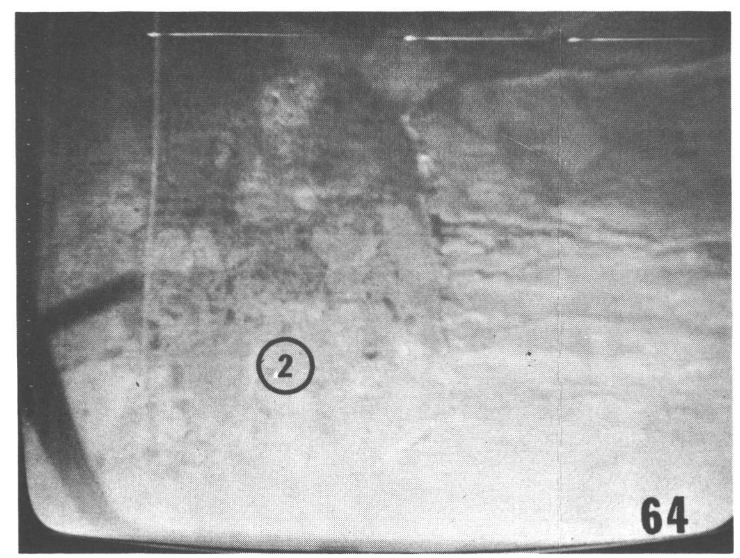
5 TENTEST

6 PLASTIC

7 WOOD



CONTINUATION OF CRACK I SHOWN IN TENTEST-BENTONITE PANEL OF OC 28

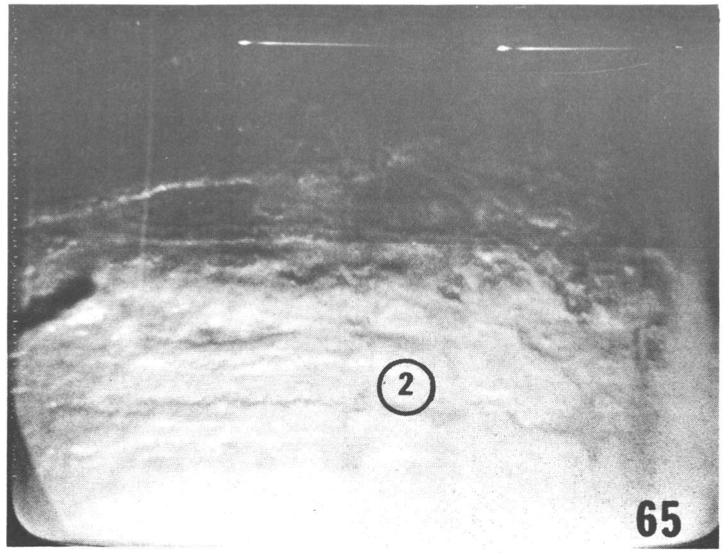


CONTINUATION CRACK I SHOWING TRIBUTARIES: 6'-8": OC 28

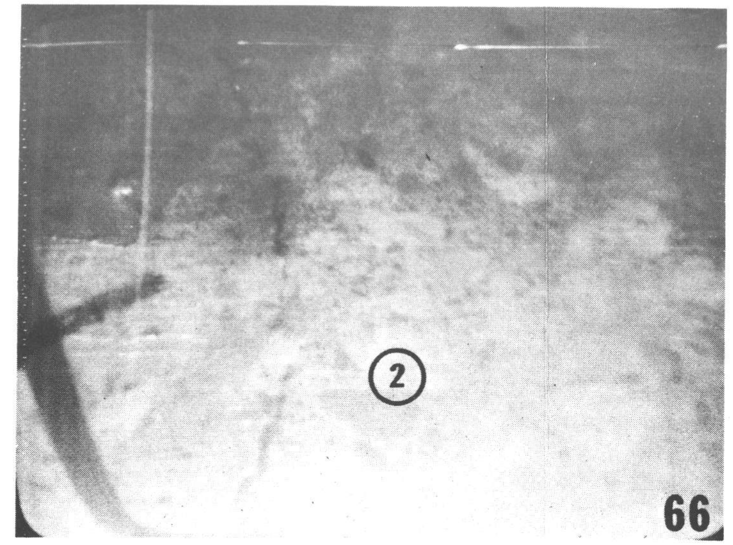
OBSERVED CRACKS IN OC 28

PHOTO 9

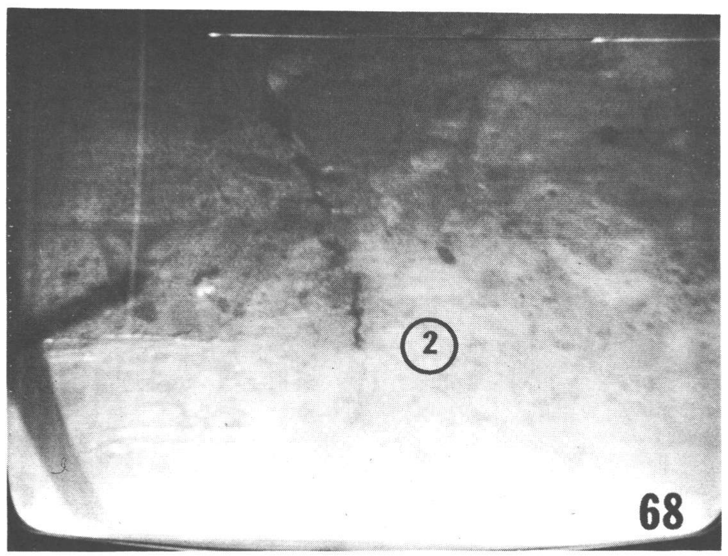
ACRES



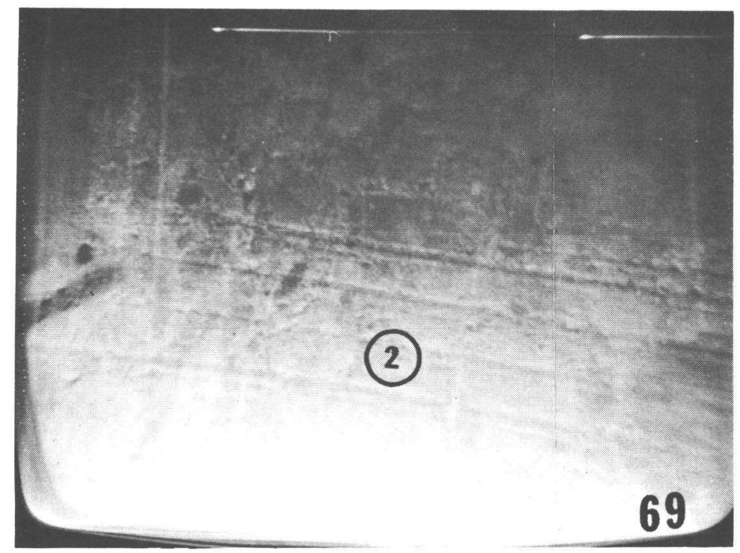
TRIBUTARIES OF CRACK I
NEAR INVERT : 6'-8" : OC 28



OPPOSITE TRACE OF CRACK I
350° : 6'-7" : OC 28



FOLLOWING TRACE OF PHOTO 66 :
350° : 6'-8" : OC28



MINUTE HAIRLINE
360° : 5'-1" : OC28

LEGEND

1 ROCK

2 BULKHEAD CONCRETE

3 WALL CONCRETE

4 BENTONITE

5 TENTEST

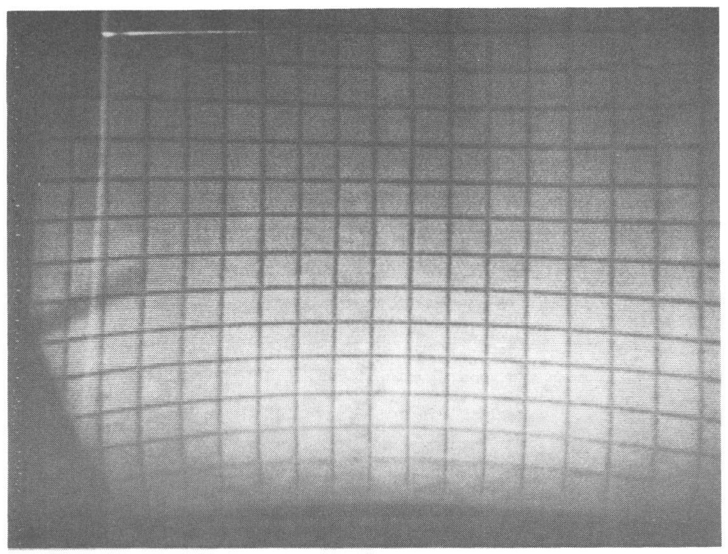
6 PLASTIC

7 WOOD

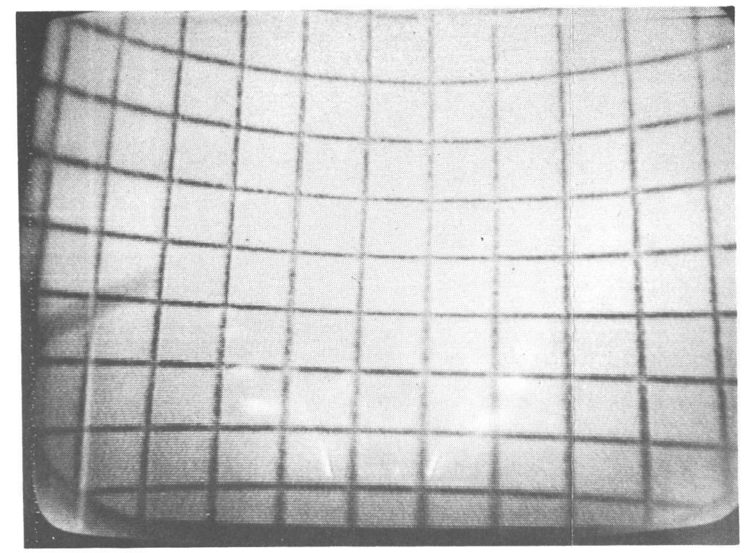
OBSERVED CRACKS IN OC 28

PHOTO 10

ACRES



1/4" GRID FOR 6" BOREHOLE



1/4" GRID FOR NX BOREHOLE

SCALE FACTORS FOR BOREHOLE OBSERVATIONS

PHOTO 11