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Government of Ontario

Ministry of Transportation and Communications

## THOROLD TUNNEL

Investigations to Determine the  
Cause of Cracking in the Structure

Supplementary Report No.1  
West Service Building



May 1972

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Ministry of Transportation and Communications

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## 1 - INTRODUCTION

### 1.1 - Terms of Reference

Following submission of the report "Investigations to Determine the Cause of Cracking in the Structure" in March 1972, the Ministry of Transportation and Communications requested Acres to continue investigations to enable the probable stress conditions in the west service building, during the summer of 1971, to be more accurately assessed, while a contract for remedial work was being prepared. Based on the results of these investigations the Ministry would decide to carry out the remedial work immediately, or to defer it until a later date.

### 1.2 - Scope of the Investigations

The investigations have been aimed primarily at obtaining a better understanding of stress levels and the variation of stress with time in the tunnel roof slab of the west service building and in adjacent tunnel sections. By means of additional stress measurements in steel reinforcing bars and in concrete, and continuing strain observations, the rate of change in stress in the structure has been assessed. The new stress data have also been correlated with a predicted mode of failure.

## 2 - CONCLUSIONS AND RECOMMENDATIONS

Investigations subsequent to those described in the original report of March 1972, and additional information obtained in April 1972, have resulted in the following conclusions:

- (a) - Stress measurements in April 1972 and continuing strain observations have confirmed that the west service building is undergoing deformation resulting from a time-dependent elastic displacement of the bedrock, referred to as rock squeeze.
- (b) - Deformations that occurred prior to 1972 resulted in major deterioration of the south wall at roadway level in the form of excessive cracking of the concrete and yielding of the reinforcing steel.
- (c) - Deformations due to rock squeeze are subject to annual cyclical fluctuations, with peak values occurring in the autumn of each year, apparently following the peak atmospheric temperatures by a time lag of up to 1 or 2 months.
- (d) - The exact amounts of deformation to which the west service building will be subjected in 1972 cannot be determined accurately, but predictions based on the limited indications to date are that stresses will be higher than those estimated to have occurred in 1971.
- (e) - The greatest compressive stresses in the next cyclical loading period will occur in the autumn of 1972 in the south roof strut, and in this member it is estimated that the compressive stress in the steel could reach 37,000 psi. (Nominal yield point 50,000 psi.)
- (f) - Elsewhere in the west service building, compressive stresses are predicted to reach peak values during 1972 close to their design limits.
- (g) - If the deformation applied to the roof strut should prove to be significantly larger than predicted for this year, then excessive strains will occur within the strut. Associated deterioration will then take place in the form of yielding of the steel and internal micro-fracturing within the concrete. Continuing deformation would lead ultimately to collapse, evidenced by progressive spalling of the concrete.
- (h) - Continuing deformation of the rock will also result in higher compressive stresses in the tunnel roof and higher bending stresses in the walls. Such higher

stresses will cause additional cracking with associated deterioration of these structural elements.

- (i) - Continuing high stress levels in the west service building are a matter of serious concern, since they will contribute to accelerated deterioration of the structure through increasing amounts of cracking and seepage.
- (j) - Deformations due to rock squeeze will probably continue for some years, although in gradually decreasing amounts. These will in due course cause failure of the roof strut, and probably also the south wall. Remedial work must, therefore, be carried out at some time.
- (k) - Major remedial work can be deferred until 1973, provided that the stresses in the roof strut are monitored continually during the remainder of 1972, and that steps are taken to arrest the increases in stress if they occur too rapidly, or to a level exceeding 35,000 psi in the reinforcing steel. Emergency measures must be available for immediate implementation to control the stresses, and in the case of the roof strut these should involve the artificial cooling of the strut to reduce the thermal stresses.
- (l) - The best means of preventing the further deformation of the structure by rock squeeze is considered to be the cutting of slots in the rock parallel to the north and south walls and close to them.
- (m) - In view of the high cost of cutting slots in the rock it can be considered sufficient to cut a slot along the south wall only at this time. This will create a condition of non-symmetrical loading of the structure from the rock on the north side, and this will result in a somewhat different pattern of deformation in the structure. While this different pattern will cause flexing of the cracks in the south wall and new cracking in the north wall, it is believed that this will not become serious for several years.

On the basis of the above conclusions it is recommended that:

- (1) - Remedial work to remove the loading due to rock squeeze from the west service building be undertaken not later than the summer of 1973. This remedial work should take the form of a slot in the rock parallel to the south wall; a similar slot parallel to the north wall will probably be required eventually, but it may be safely deferred, possibly for several years.



- (2) - The deformations and stresses in the entire west service building must be monitored on a continuing basis until remedial work is completed.
- (3) - If the remedial work is not undertaken in 1972, then the stress levels in the south roof strut must be carefully monitored and a program must be available for quick implementation whereby the strut can be relieved of the thermal stress, should there be indications that the total compressive stresses reach unacceptable levels. It is recommended that artificial cooling of the strut be employed to control these thermal stresses.
- (4) - Regardless of any deferment of the remedial work, the contract documents for the work should be completed as soon as possible so that they will be available for immediate use if continuing monitoring indicates the advisability of commencing the work.

### 3 - FIELD INSTRUMENTATION

The objective of the additional field instrumentation was to determine present (April 1972) stress levels in the concrete and in the steel reinforcement of the west service building, primarily at the tunnel roof slab level, elevation 549.83.

#### 3.1 - Concrete Stress Measurements

Concrete stress levels were determined using the overcoring technique described in the report of March 1972. Stresses were measured in holes OC32 to OC40, located in the tunnel roof structure at elevation 549.83, and in the east and west walls of the west service building, as shown on Plate 1. These test locations were selected to enable a comparison to be made between stress levels in the north and south parts of the service building, and to ascertain areas where stresses were expected to be a maximum. It was considered imprudent to carry out any more overcoring in the tunnel roof strut area.

#### 3.2 - Steel Stress Measurements

Stresses in reinforcing steel were determined directly by attaching electrical resistance foil strain gauges to exposed bars. Each bar was cut from the structure and the change of stress was calculated from the strain measured in the bar. This type of test was performed at five locations in the tunnel roof slab, at elevation 549.83 feet, and in the roof of the tunnel immediately below the east wall of the west service building, as shown on Plate 1. The test locations in the roof slab were selected to provide stress measurements in the critical tunnel roof strut, and also adjacent to overcoring test locations, to enable a direct comparison of the results of the two test procedures to be made.

In addition to the stress measurements described above, strain and temperature observations, commenced in the previous investigations, were continued.

#### 3.3 - Further Instrumentation

Before remedial work to unload the west service building is commenced, it is considered desirable that certain additional instrumentation be installed to facilitate accurate monitoring of the results of the remedial work. Such instrumentation could take the form of slope indicator devices installed in boreholes in the rock near the structure, and use of a tiltmeter to determine changes in face angle in the outside walls of the tunnel.

#### 4 - FIELD OBSERVATIONS

The results of additional stress measurements carried out in April 1972 are shown on Plate 1. Stress measurements made in October 1971 are shown on Plate 2. Continuing observations on deformations of the west service building and temperature are given on Plates 3 and 4.

##### 4.1 - Stress Measurements

The distribution of stresses measured in concrete and steel is generally consistent with that anticipated as a result of the previous investigations. The effect of a seasonal change in stresses is apparent, with lower compressive stresses now recorded in the north-south direction in the tunnel roof slab, and low compressive or tensile stresses in the east-west direction.

The measurements tended to confirm that maximum stresses are obtained in the tunnel roof strut between the ventilation openings.

##### 4.2 - Comparison of Concrete and Steel Stresses

Comparison of overcoring test results and steel stress measurements resulted in a wide range of possible values for the effective modular ratio. Following consideration of the values obtained and a review of published information, an effective modular ratio of 15 was selected initially for use in assessing long-term behaviour of the west service building. A lower limit of 11 for the effective modular ratio was adopted later, after detailed analysis of results.

##### 4.3 - Observed Deformations

Strain measurements on the tunnel roof slab at the west service building indicated initially a small contraction from September to November 1971, and thereafter a gradually increasing expansion of up to 0.05-inch for the southern half of the building up to April 1972. The trend from November onwards was confirmed by extensometer readings across the structure. This expansion of the building appears consistent with seasonal variations in rock squeeze as observed by Ontario Hydro, see Plate D-1 in the report of March 1972.

#### 4.4 - Wall Extensometers

Due to the small movements recorded, it has proved impossible to establish definite trends for relative movement between the tunnel concrete and the bedrock.

## 5 - ANALYSIS OF RESULTS

Based on the reinforcing steel stress of 16,320 psi measured at the south of the tunnel roof strut in April 1972, the change in temperature and the measured expansion of the building over the winter 1971 to 1972, an attempt has been made to correlate the estimated stress in the reinforcing steel in October 1971 with that measured in the concrete at the same time. From this it proved possible to develop a method of predicting deformation due to rock squeeze, and based on the estimated deformation obtained over the 5-year period from completion of the concrete bulkheads to date, an estimate was made of peak stress levels anticipated in the tunnel roof strut in 1972.

From Table 1 it can be seen that the stress in the reinforcing steel in the tunnel roof strut in October 1971 is estimated to have been of the order of 28,000 psi, and, based on an effective modular ratio of 15, this is equivalent to a stress of 1,900 psi in the concrete. This is at the low end of the range actually measured - between 1,900 and 2,700 psi, suggesting that possibly the effective modular ratio may be as low as 11. The seasonal reduction in rock squeeze which appears to have occurred, and has been allowed for in calculating the change in stress, is assumed to be as illustrated on Plate 5.

Consideration of the calculated amount of deformation due to rock squeeze applied to date to the west service building, shows that an average annual deformation across one-half of the building of 0.05-inch has occurred to date. As shown on Plate 5 the comparable deformation over the next year, neglecting seasonal fluctuation, will probably be less than 0.05-inch, by an amount which cannot be determined due to present lack of data. If it is assumed that rock squeeze will continue at the same rate for the next year, then the prediction of deformation at the time of peak stress levels in 1972 should be reasonably conservative. The stresses resulting from such an imposed deformation were calculated and modified by thermal stresses in the structure, as shown in Table 2.

On the foregoing basis, peak reinforcing steel stress in the tunnel roof strut in September 1972 is predicted to be of the order of 37,000 psi in compression, and the corresponding concrete stress between 2,500 and 3,300 psi. The corresponding peak stress in reinforcing steel in the tunnel roof slab, other than in the tunnel roof strut, is predicted to be of the order of 20,000 psi.

The annual rate of increase in deformation due to rock squeeze acting on the west service building cannot be determined accurately until at least one complete cycle of observations can be completed. Although using the calculated average trend over the 5 years since completion of construction would appear to be reasonably conservative, it cannot be stated that the exact mechanism of rock squeeze is either understood, or that it is possible to predict the cyclic fluctuations with confidence. In view of this, the rate of increase in rock squeeze may well prove to be even higher than that assumed, with corresponding increases in actual stresses over those now predicted.

## 6 - ASSESSMENT OF STRUCTURAL SAFETY

Further to the assessment of structural safety included in the report of March 1972, the investigations carried out in April 1972 have resulted in a more detailed assessment of safety for certain structural elements.

### 6.1 - Tunnel Roof

It has been confirmed by the April 1972 measurements that there are stress concentrations in the tunnel roof struts caused by the configuration of the ventilation openings.

As a result of these concentrations of stress, maximum compressive stresses occur in the tunnel roof struts, with the stress observed in the south strut being significantly higher than that in the north strut. Stresses in the tunnel roof slab generally are considerably lower than those in the roof struts, so that if the roof struts are not overstressed then stresses elsewhere at roof level will be acceptable.

Comparison of stress measurements in the reinforcing steel (April 1972) with those measured in the concrete (October 1971) have indicated that stresses in the tunnel roof struts are influenced significantly by thermal changes and applied deformation due to rock squeeze. These reinforced concrete struts can be considered similar to laterally tied columns whose structural behaviour can be considered to pass through three phases, as follows:

Phase 1 - Range of strain - 0 to approximately 0.04 per cent; normal behaviour with stress levels not exceeding the design limits. This corresponds to a reinforcing steel stress of approximately 12,000 psi.

Phase 2 - Range of strain approximately 0.04 to approximately 0.5 per cent; stress in excess of design limits for either the reinforcing steel or concrete. When strains in the strut exceed values of the order of 0.2 per cent, distress in the form of plastic flow in steel and progressive micro-fracturing of the concrete will occur. This corresponds to development of a permanent deterioration of this portion of the structure, resulting in reduced capacity for resisting loads, freeze-thaw cycles, etc.

Phase 3 - Range of strain greater than approximately 0.5 per cent. It is expected that the effects of progressive micro-fracturing will become more evident in the form of minor spalling. Continuing application of imposed deformation will result in progressive collapse of the structure.

The maximum reinforcing steel stress of 37,000 psi predicted for 1972 in the south tunnel roof strut will have an associated strain of 0.12 per cent. This member will be well advanced into the Phase 2 portion described above, although some capacity for continuing strain will remain. At present the tunnel roof struts provide support to the north and south walls of the west service building. Due to the uncertainties involved in predicting changes in strain with variation in rock squeeze, it is not possible to predict exactly what will happen if the tunnel roof strut is allowed to reach the Phase 3 condition. The pattern of cracking already observed in the wall in the immediate vicinity of the roof strut indicates that that the wall is already in a Phase 2 condition. The removal of support by the tunnel roof strut could well result in an unacceptable acceleration of the deterioration of the wall in these areas.

If monitoring of the tunnel roof struts during 1972 indicates a more rapid rate of increase in stress than is considered acceptable it will be necessary to undertake temporary measures to reduce the stress. Since a substantial part of the peak stress is due to thermal effects, a reduction in stress could be achieved by artificially cooling these members. This will reduce the stress in the struts, but would not of course substantially reduce the applied deformation of the tunnel walls.

Present indications are that the remainder of the tunnel roof slab will reach the Phase 2 condition in the autumn of 1972.

It is not anticipated that the 1972 temperature variations in the tunnel roof, 19 feet either side of the West Service Building, will be sufficient to cause these members to exceed the Phase 1 condition.

## 6.2 - Tunnel Wall

As stated in the report of March 1972, the south tunnel wall at the West Service Building has already been subjected to considerable strain, causing extensive cracking. The concept of behavioural phases, as outlined previously for compressive members, can also be applied to flexural members with certain adjustments to the defined limits.



Phase 1, representing the normally accepted design range, has permissible average extensional strains up to approximately 0.07 per cent. This will correspond to an average steel stress of approximately 20,000 psi, with the surrounding concrete being subjected to normal tensile cracking.

The upper limit of Phase 2 will be governed by the ratio of the reinforcing steel area to the concrete sectional area, the strain in the latter being of the order of 0.5 per cent.

With increasing strain, the steel stress will continue to increase proportionally to its yielding value. The concrete will also undergo progressive cracking, but the cracking will eventually stabilize at a condition where the spacing of the surface cracks is at approximately twice the concrete cover. For the Thorold Tunnel with its 3 to 4 inches of concrete cover, the ultimate crack spacing will thus be in the range 6 to 8 inches. Further imposed deformation will then be reflected in proportional increases in crack widths.

It is normally considered in design that maximum crack widths should be limited to 0.008 inches in corrosive surroundings, and 0.012 inches in non-corrosive surroundings.<sup>(1)</sup> This corresponds to average surface strains of approximately 0.15 per cent.

Provided that no significant corrosion of the reinforcement has taken place, the extensional strains which can occur on the tensile face can be very large before failure of the structure can occur in the form of failure of the steel or spalling of the concrete.

Transition from Phase 2 to Phase 3 condition is considered to represent unacceptable structural deterioration prior to structural collapse.

Based on the above definitions, the south wall is considered to be in Phase 2 of the structural behaviour scale. The exact position in this phase is difficult to assess, but in areas of the cracked wall the stress in the steel has obviously exceeded the yield point.

For these areas, the strain in the steel is estimated to be of the order of 0.3 per cent, and the measured compressive concrete stress in October 1971 to be of the order of 1500 psi (equivalent to a strain 0.12 per cent). Surface crack widths up to a maximum of 0.025-inch at approximately 8 to 10-inch spacing are evident on the wall, and the depth of

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(1) Levi, F. "Work of the European Concrete Committee"  
ACI Journal, Proceedings V57, No. 9, Mar. 1961,  
pp. 1041-1070

one of these has been observed to be greater than 3 feet 6 inches from the inside face. In view of the severe cracking observed in the south wall, and the obvious deterioration over the last 5 years, it is assumed that this member has advanced well into Phase 2 but can withstand considerable further deformation before reaching Phase 3 and ultimate collapse.

The north wall appears to be in transition between Phases 1 and 2, as evidenced by the smaller amount of cracking and the lower concrete stress measurements of October 1971.

It is considered that both north and south walls are not in imminent danger for the 1972 load cycle, although progressive deterioration will continue.

Table 1

STRESSES IN TUNNEL ROOF STRUT  
OCTOBER 1971

Measured reinforcing steel stress (April 1972 at 46°F)	16,320 psi (compressive)
Thermal contribution (46°F to 67°F)	6,480 psi (compressive)
Measured rock movement 0.05-inch	4,900 psi (compressive)
Estimated steel stress October 1971	27,700 psi say 28,000 " (compressive)
Estimated concrete stress - effective modular ratio,	
m = 15	1,900 psi (compressive)
m = 11	2,700 psi (compressive)
Measured concrete stress	1,900 to 2,700 psi (compressive)

Table 2

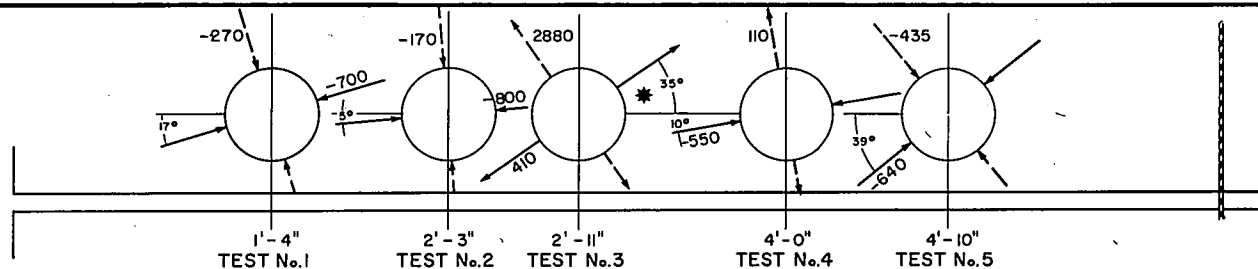
PREDICTED STRESSES  
SEPTEMBER 1972

	<u>Tunnel Roof Strut Axial Compressive Stress (psi)</u>	<u>Tunnel Roof Slab Axial Compressive Stress (psi)</u>
	<u>Position 4</u>	<u>Position 5</u>
Measured reinforcing steel stress (April 1972 - 46°F)	16,320	6,030
Thermal contribution (46°F to 80°F)	10,500	7,400
Predicted rock movement April to September, 1972 0.1-inch	9,810	6,600
Peak steel stress predicted (September 1972)	36,630 say 37,000	20,030 say 20,000
Peak concrete stress (effective modular ratio m = 11 to 15)	2,500 to 3,300	1,400 to 1,800

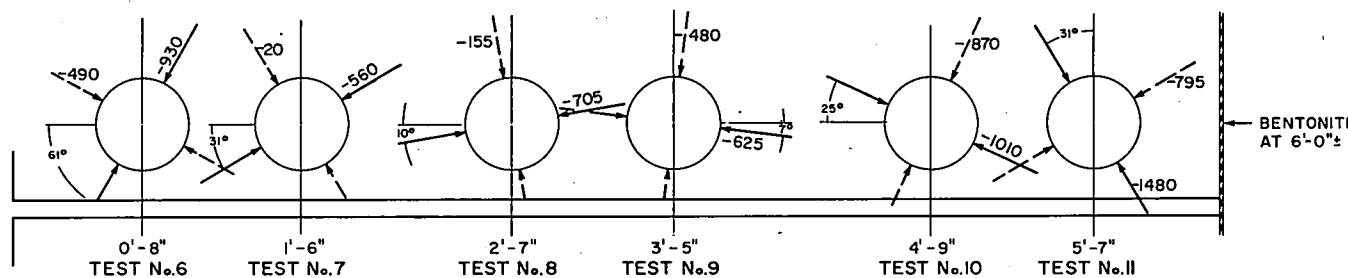
PLATES



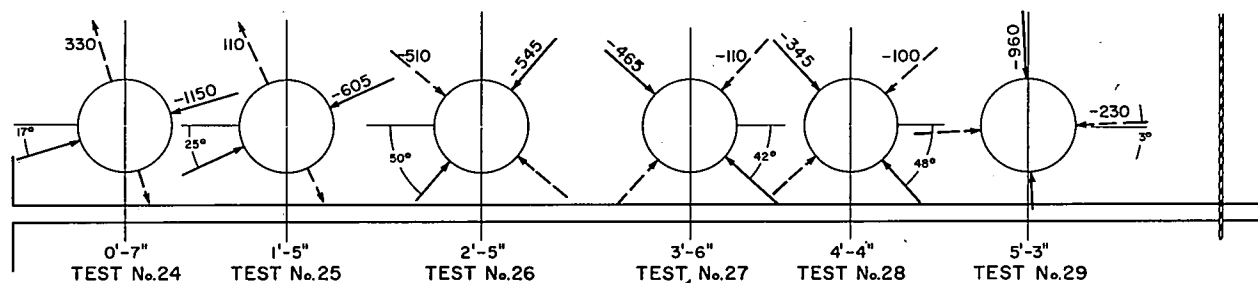
OC No. 14  
SOUTH-WEST WALL



OC No. 15  
SOUTH-WEST WALL



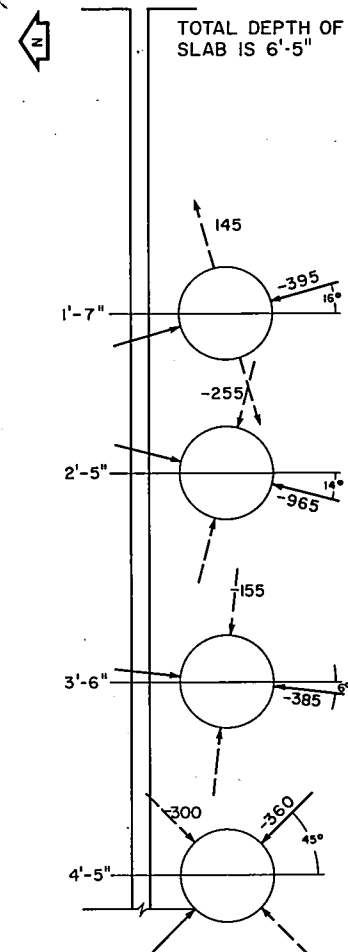
OC No. 28  
NORTH-WEST WALL



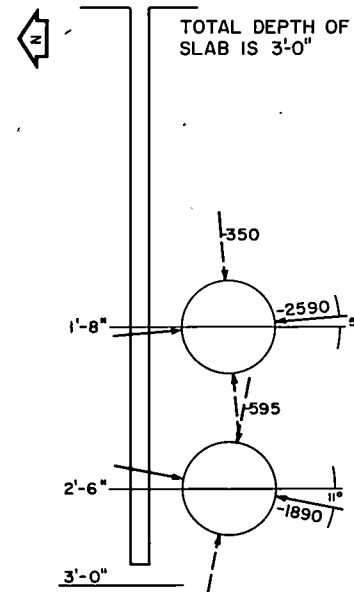
# NOTES

1.  $\sigma_1$  = MAJOR PRINCIPAL STRESS  
 $\sigma_2$  = 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

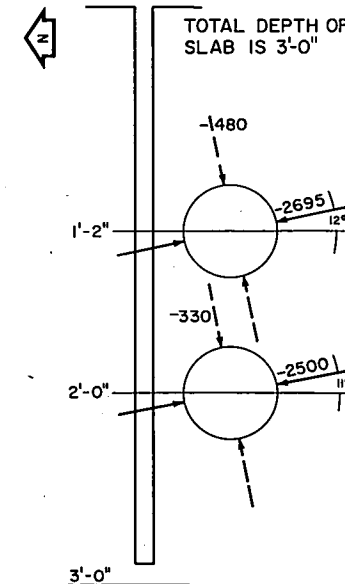
OC No. 23  
ROAD SLAB



OC No. 25  
ROOF SLAB



OC No. 26  
ROOF SLAB



ACRES

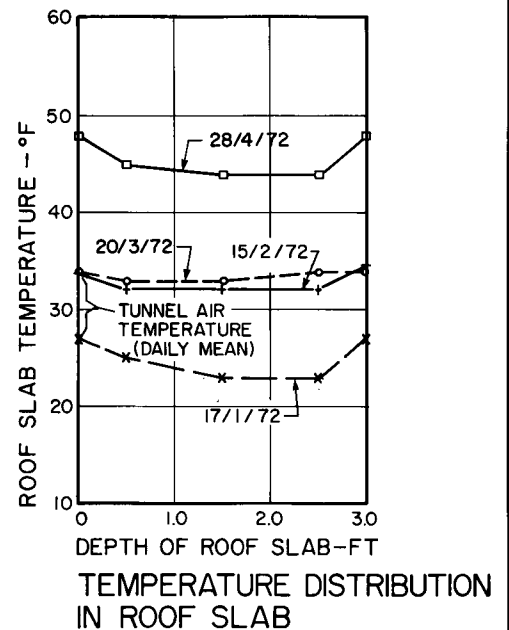
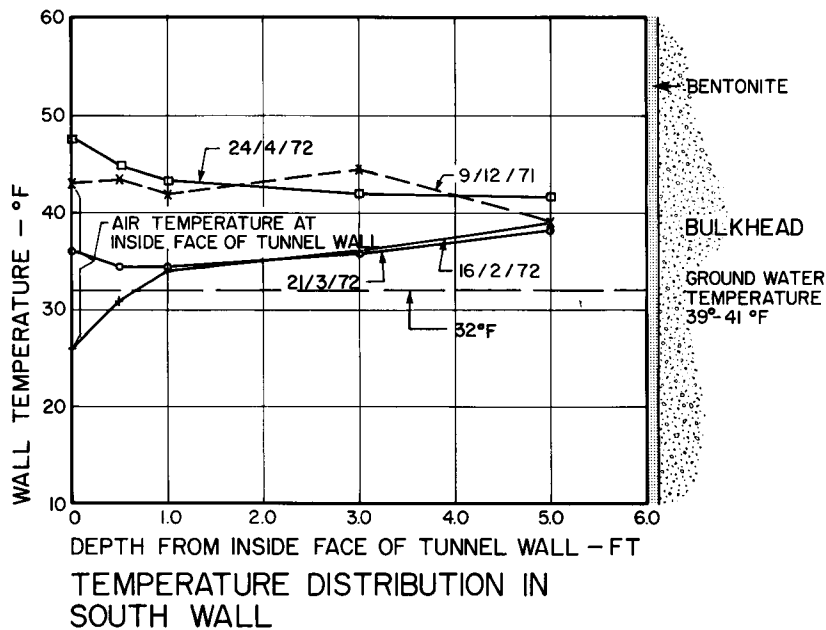
MINISTRY OF TRANSPORTATION  
AND COMMUNICATIONS  
THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING  
OVERCORING TESTS  
RESULTS - OCTOBER 1971

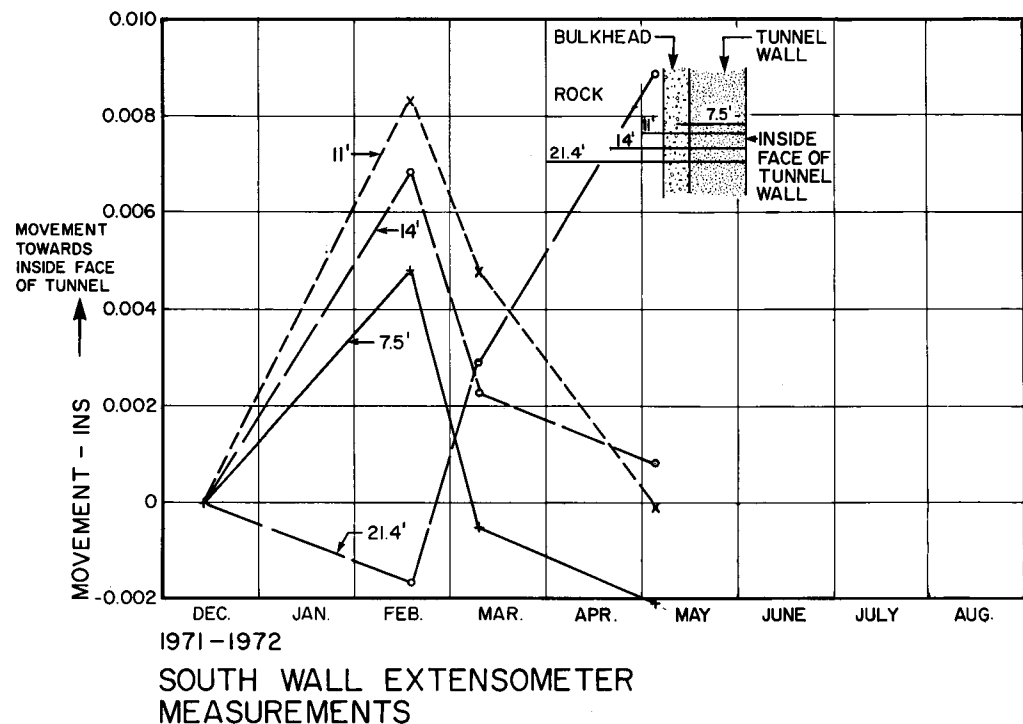
*[Signature]*  
ACRES CONSULTING SERVICES LIMITED

MAY 1972

PLATE  
2



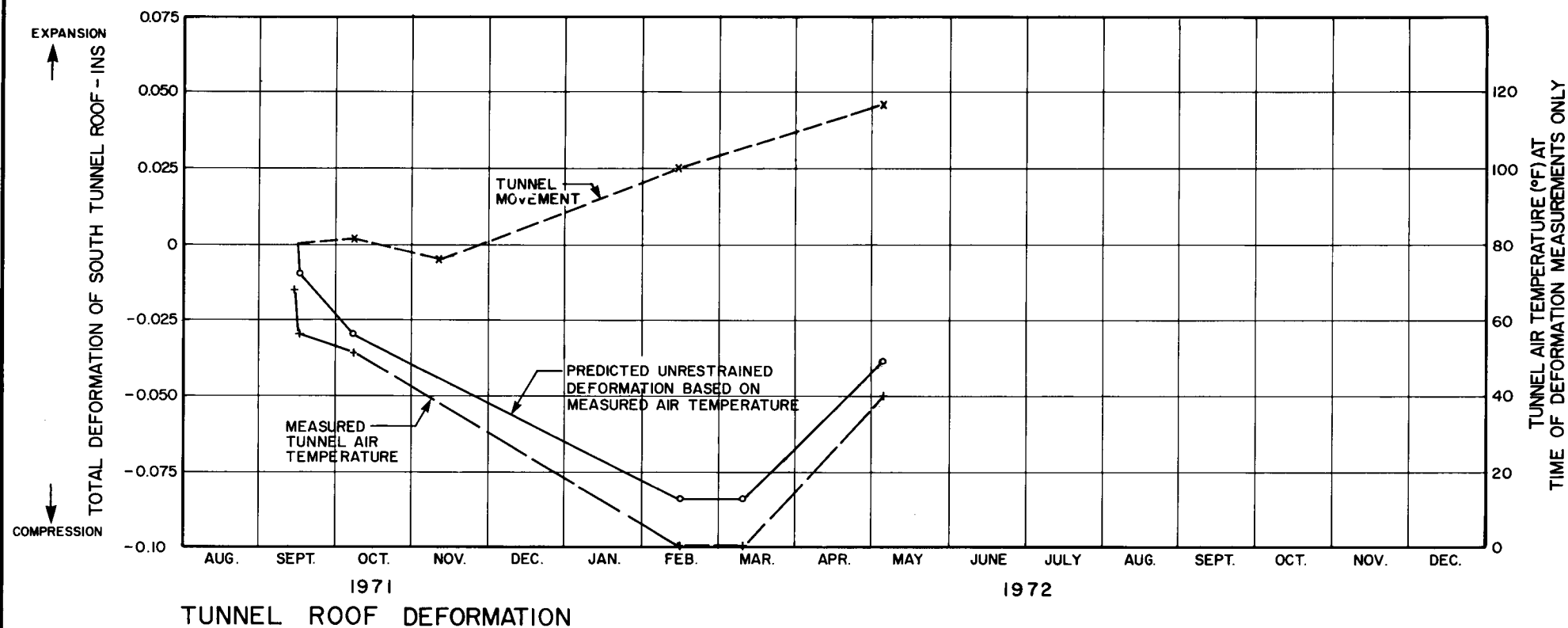
NOTE  
DAILY MEAN AIR TEMPERATURE INDICATED AT TOP (0 FT) AND BOTTOM (0 FT) OF SLAB



NOTES  
FOR LOCATION OF MEASUREMENTS REFER TO PLATE 6

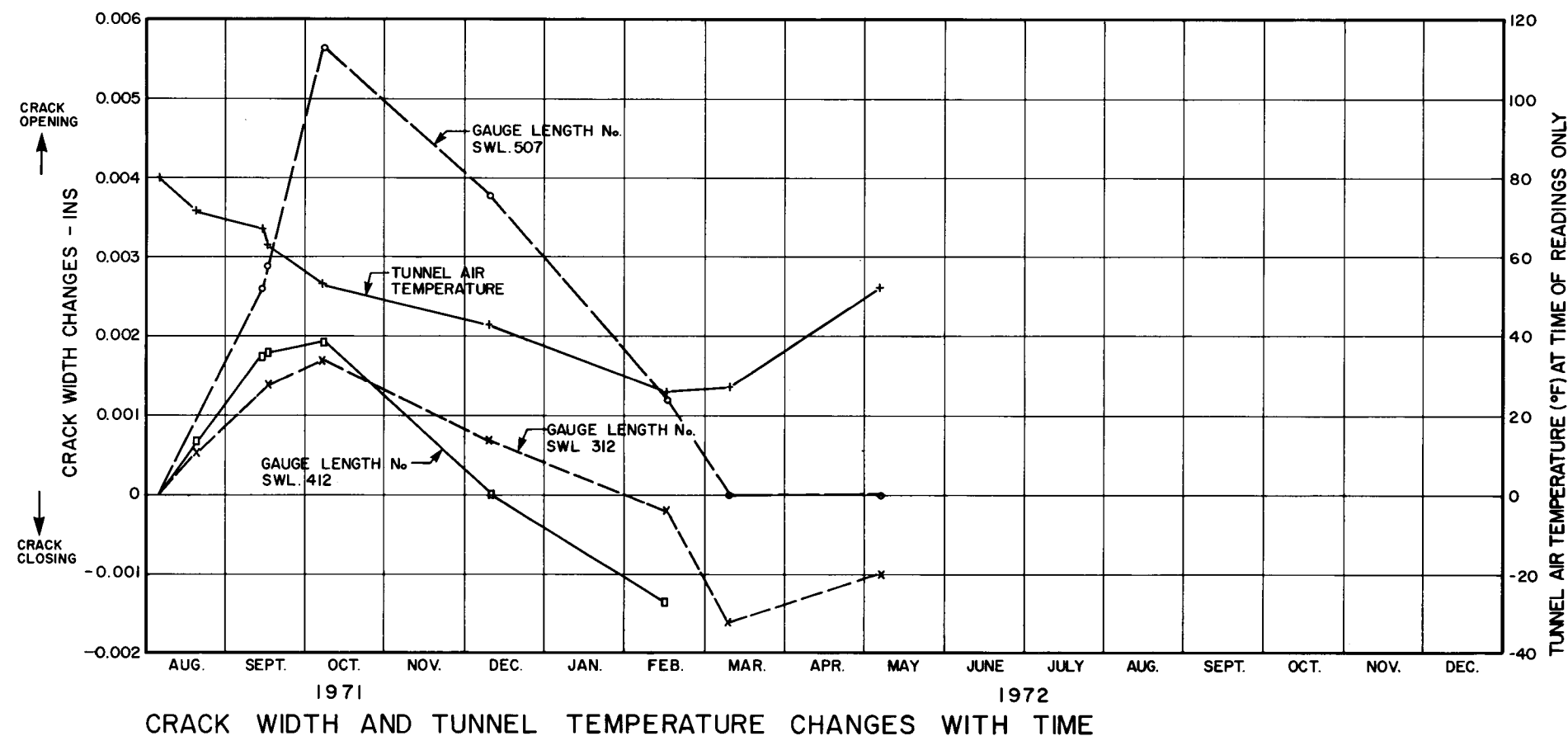
ACRES	MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
	THOROLD TUNNEL STRUCTURAL INVESTIGATIONS
WEST SERVICE BUILDING CONTINUING OBSERVATIONS TEMPERATURE AND WALL EXTENSOMETERS	
 ACRES CONSULTING SERVICES LIMITED	MAY 1972 PLATE <b>3</b>

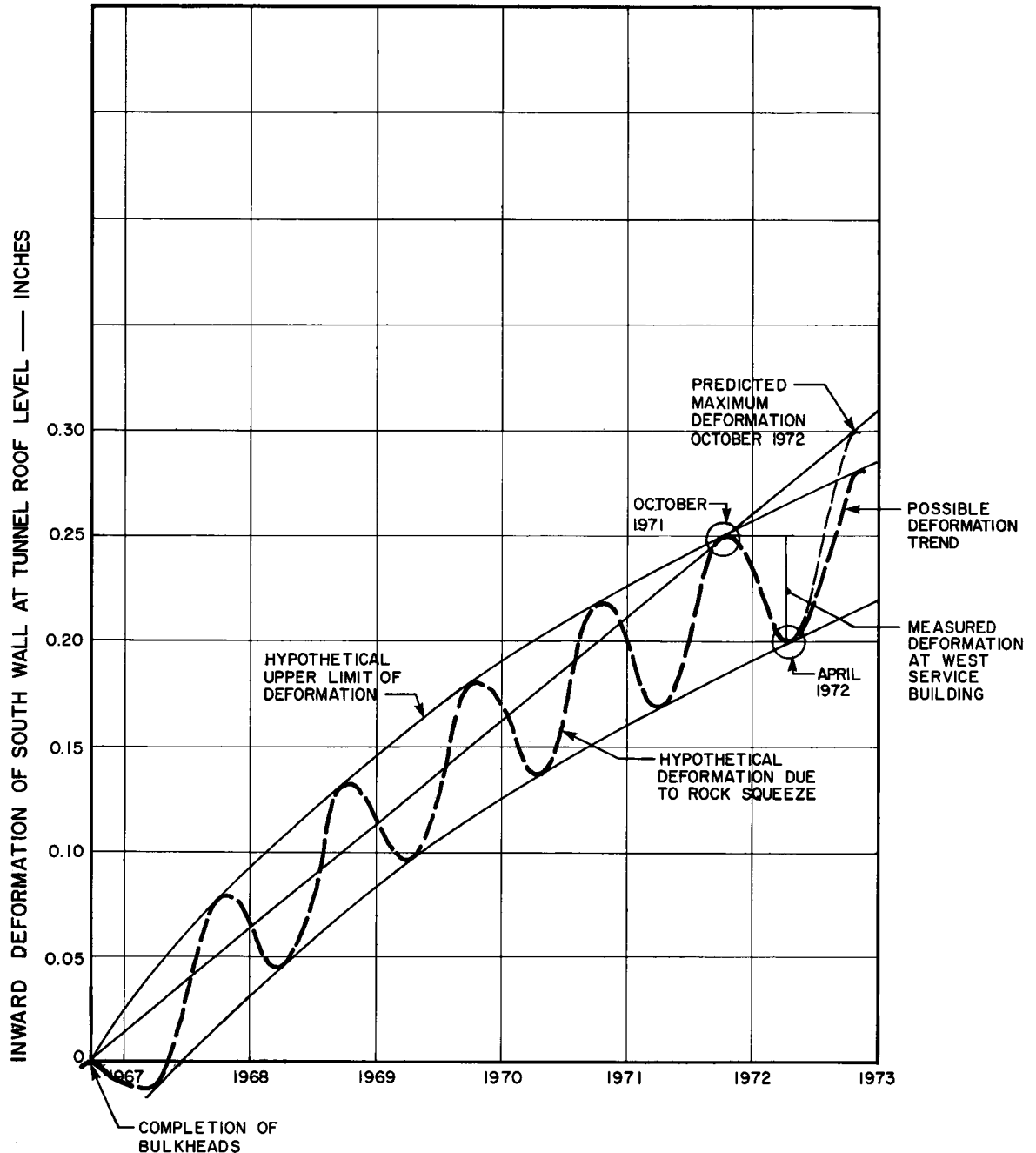




#### NOTES

1. FOR LOCATION OF MEASUREMENTS REFER TO PLATE 6
2. TUNNEL ROOF DEFORMATION IS CALCULATED FROM STRAINS MEASURED BY EXTENSOMETERS SHOWN IN PLATE 6



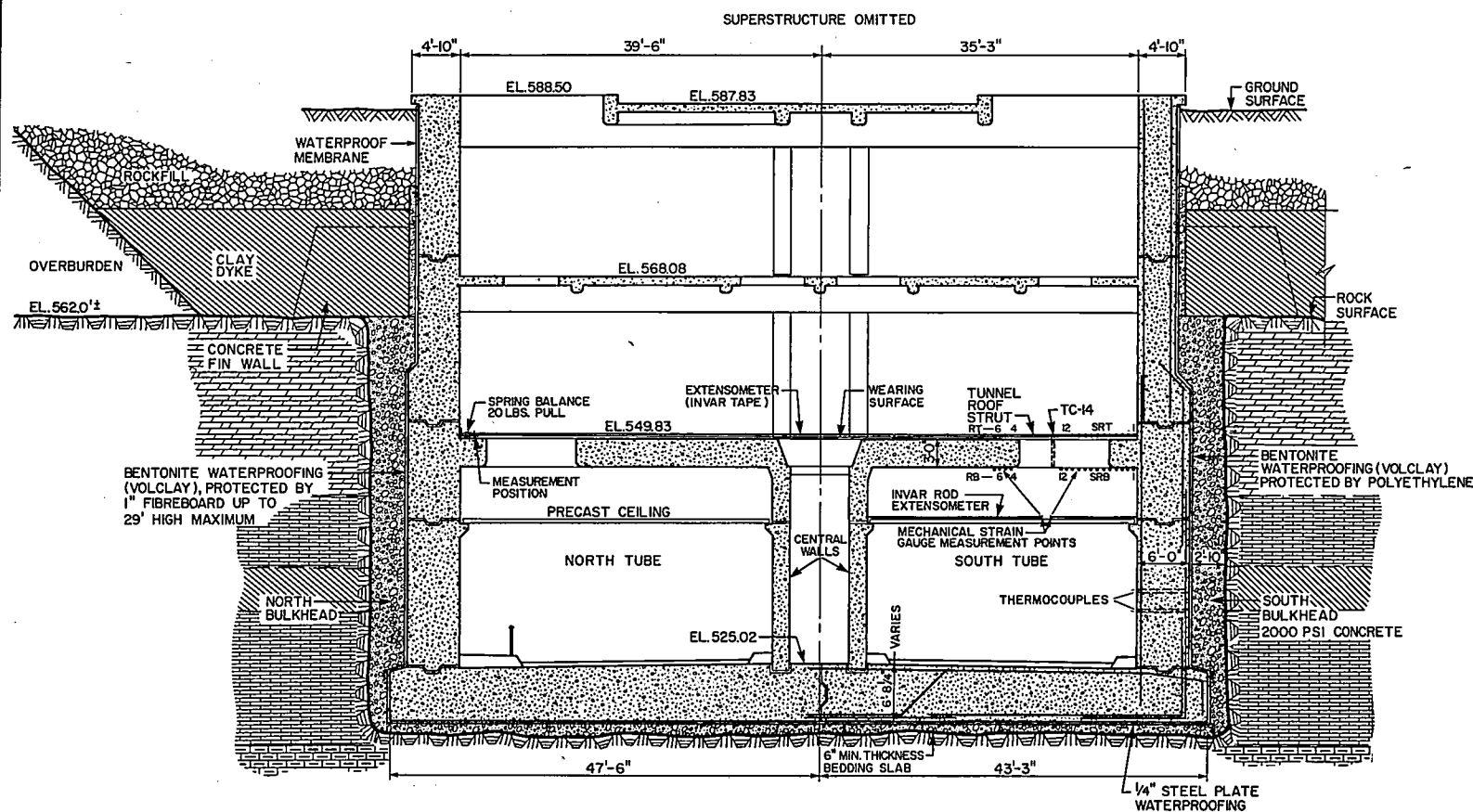


#### NOTES

HYPOTHETICAL PATTERN OF DEFORMATIONS BASED ON MEASUREMENTS OF WHEEL PIT MOVEMENTS MADE BY ONTARIO HYDRO

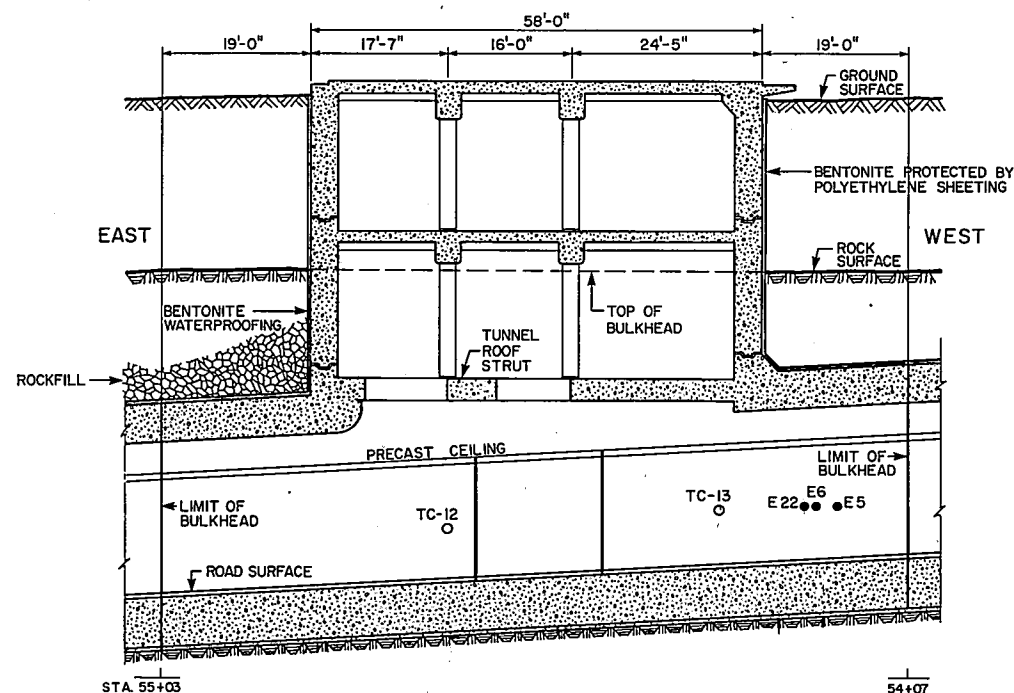
○ POINTS ON CURVE BASED ON FIELD OBSERVATIONS AT THOROLD

ACRES	MINISTRY OF TRANSPORTATION AND COMMUNICATIONS	
	THOROLD TUNNEL STRUCTURAL INVESTIGATIONS	
WEST SERVICE BUILDING IMPOSED DEFORMATION DUE TO ROCK SQUEEZE HYPOTHETICAL CURVE		
 ACRES CONSULTING SERVICES LIMITED		MAY 1972 PLATE <b>5</b>



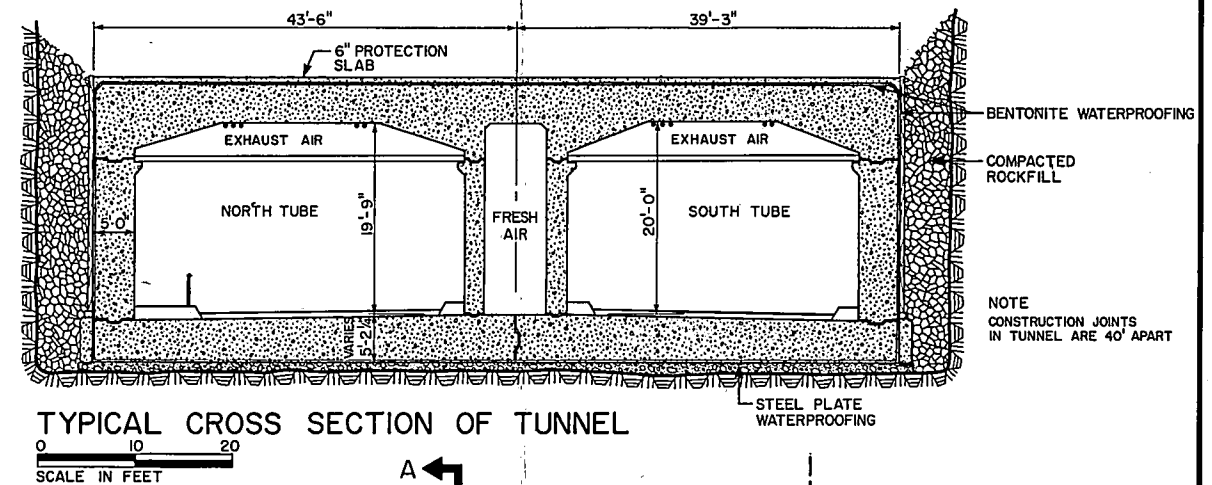
SECTION A-A AT STATION 54+55

SCALE IN FEET



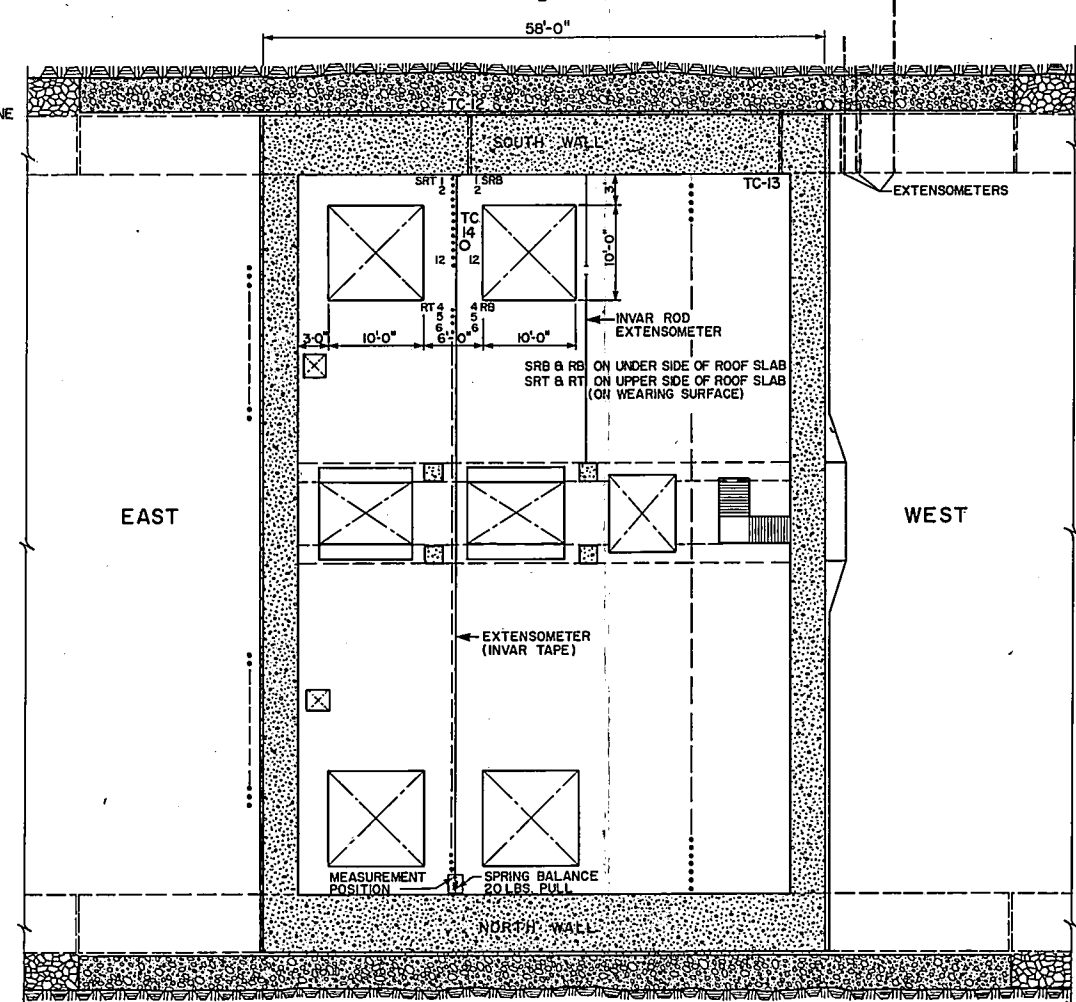
SOUTH TUNNEL WALL - ELEVATION

SCALE IN FEET



TYPICAL CROSS SECTION OF TUNNEL

SCALE IN FEET



PLAN AT EL. 549.83

SCALE IN FEET

#### NOTES

- E 5, 6, AND 22 EXTENSOMETER POSITIONS
- TC 12, 13 AND 14 THERMOCOUPLE INSTALLATIONS
- ..... ADDITIONAL MEASUREMENT POSITIONS INSTALLED MAY 1972

APPENDIXES

## APPENDIX A

### BASIS FOR STRESS CALCULATIONS

#### Reinforcing Steel Properties

Modulus of elasticity	- $30 \times 10^6$ psi
Coefficient of thermal expansion	- $6 \times 10^{-6}$ in/ $^{\circ}$ F
Yield stress	- Nominal 50,000 psi (Measured 57,500 psi)
Ultimate stress	- Nominal 80,000 psi (Measured 99,500 psi)

#### Concrete Properties

Instantaneous modulus of elasticity	- $4.5 \times 10^6$ psi
(Range of measured values	- $3.0-6.0 \times 10^6$ psi)
Ultimate compressive strength	- 5,000 psi
(Range of measured values	- 5,000-7,450 psi)
<u>Effective modular ratio</u>	= 11-15

#### Structural Configuration

Effective length of each tunnel roof strut	- 16 feet
Effective width of tunnel wall carried by strut	- 16 feet
Ratio of stresses in strut to those in slab	- 6:16

As built area of reinforcing steel in strut, top 6 #10, and 5 #11 bars and bottom 11 #10 bars, #5 ties at 12-inch centres.

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

**DRILLING REPORT**

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC32

SITE West Service Building SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear STARTED p .M. April 6, 1972  
Limited FINISHED p .M. April 6, 1972

METHOD OF DRILLING: SOIL Concrete Diamond Drill CASING DIAM. \_\_\_\_\_  
ROCK CORE DIAM. 5.778 inches

LOCATION: WATERS Third Level ELEVATIONS: DATUM Hole El. 553.33  
DEPARTURE El. 549.83 DRILL PLATFORM \_\_\_\_\_  
BEARING East Wall GROUND SURFACE \_\_\_\_\_  
WIND DIRECTION South End ROCK SURFACE \_\_\_\_\_  
OTHER DATA Horizontal BOTTOM OF HOLE \_\_\_\_\_  
Bearing East WATER TABLE \_\_\_\_\_

DEPTH	Concrete ROCK-TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 2'5"	Structural Concrete	0'7" Vertical Bars - No. 8 - 3" centre to centre  0'8" Horizontal Bars - No. 8  0'7.75" to 0'10.25" - Core broken manually by drillers. Diagonal break intersecting horizontal reinforcing bars. - Bond around these bars appears good. - Ex-hole at this point is approximately 1/4 inch off centre toward top of core.  0'11" to 1'9" Corrugation Rings - Indicating rough drilling.  2'5" End of Hole - Core broken by drillers approximately 90 degrees to axis of core. - Ex-hole at this point is approximately 1/4 inch off centre toward top of core.	

INSPECTOR W. P. Pratt

LOGGED BY W. P. Pratt

APPROVED 

DATE May 31, 1972

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

**DRILLING REPORT**

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC33

SITE West Service Building SHEET No. 1 OF 2

CONTRACTOR: Canadian Longyear STARTED a.m. April 7, 1972  
Limited FINISHED a.m. April 7, 1972

METHOD ~~SOIL~~ CASING DIAM. \_\_\_\_\_  
 OF Concrete Diamond Drill  
 DRILLING: ~~ROCK~~ CORE DIAM. 5.778 inches

LOCATION: ~~WATERSIDE~~ Third Level ELEVATIONS: ~~DATUM~~ Floor 549.83  
~~DEPARTURE~~ Floor DRILL PLATFORM  
~~BEARING~~ Southwest area GROUND SURFACE  
~~WINDY~~ Vertical ROCK SURFACE  
~~OTHER DISK~~ BOTTOM OF HOLE  
WATER TABLE

DEPTH	Concrete ROCK-TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 0'2.25"	Industrial Concrete Topping	0'2.25" Core Break - Industrial concrete topping separated from floor concrete during drilling. - Ex-hole well centred.	
0'2.25" to 2'9"	Structural Concrete	0'6.8" East/West Rebars - Two No. 8 bars - Bars run approximately through centre of core.  0'7.3" North/South Rebars - No. 8 bars. - Bars pass through core to the east of core centre line.  1'6.5" - Core broken manually by drillers. Break is approximately 90 degrees to core axis. - Ex-hole off centre approximately .25 inch to the east.  1'6.5" to 2'9" Corrugation Rings - Indicating "chattering" drill.  2'9" Embedded Pattern of North/South Steel Visible - Two inches west of centre line of core.	

INSPECTOR W. P. Pratt

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DATE May 31, 1972

## H. G. ACRES LIMITED - CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

## DRILLING REPORT

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC33

SITE West Service Building SHEET No. 2 OF 2

DEPTH	Concrete ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
		2'9" End of Hole - Core broken manually by drillers. - Break is approximately 90 degrees to axis. - Ex-hole .25 inch to east of centre line.	



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

**DRILLING REPORT**

CLIENT Ministry of Transportation and Communications JOB No. P2499.02  
 PROJECT Thorold Tunnel Field Investigations HOLE No. OC34  
 SITE West Service Building SHEET No. 1 OF 2  
 CONTRACTOR: Canadian Longyear STARTED p .M. April 7, 1972  
                   Limited FINISHED p .M. April 7, 1972  
 METHOD OF DRILLING: SOIL CASING DIAM. \_\_\_\_\_  
                           Concrete Diamond Drill  
                           ROCK CORE DIAM. 5.778 inches  
 LOCATION: DATE Third Level ELEVATIONS: DATUM El. 553.8  
                   DEPARTURE West Wall DRILL PLATFORM \_\_\_\_\_  
                   BEARING South End GROUND SURFACE \_\_\_\_\_  
                   INITIAL DIP Horizontal ROCK SURFACE \_\_\_\_\_  
                   OTHER DIPS West BOTTOM OF HOLE \_\_\_\_\_  
   Bearing WATER TABLE \_\_\_\_\_

DEPTH	Concrete ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 2'9"	Structural Concrete	0'0" Ex-hole off centre towards top of core approximately .25 inch.	
		0'2" Horizontal Bars - No. 8 bars, bond appears good. - Approximately 1.25 inch below centre line of core.	
	Concrete Appears Sound Few Voids	0'3-1/2" Vertical Bars - No. 8 bars. - Slightly north of centre line.	
		0'4-1/2" Second set of horizontal bars - Directly in line with first set of horizontal bars when looking along axis of core.	
		1'2" Core broken manually by drillers - Core break is approximately 90 degrees to axis of core.	
		1'2" to 2'9" Corrugation rings - Indicating "chattering" drill.	
		2'9" End of Hole - Core broken manually by drillers at approximately 90 degrees to axis of core.	

INSPECTOR W. P. Pratt

LOGGED BY W. P. Pratt

APPROVED

*W. P. Pratt*

DATE

May 31, 1972

## H. G. ACRES LIMITED - CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

## DRILLING REPORT

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC34

SITE West Service Building SHEET No. 2 OF 2

DEPTH	Concrete ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
		- Ex-hole off centre toward top of core by approximately .25 inch.	

## H. G. ACRES LIMITED — CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

## DRILLING REPORT

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC35

SITE West Service Building SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear LIMITED STARTED p.m. April 11, 1972  
FINISHED p.m. April 11, 1972METHOD OF DRILLING: SOIL Concrete Diamond Drill CASING DIAM. CORE DIAM. 5.778 inches  
ROCKLOCATION: ~~NATURAL~~ Third Level ELEVATIONS: ~~NATURAL~~ Floor 549.83  
~~DEPARTURE~~ Floor  
~~BEARING~~ Northwest Area  
~~INTAKE~~ Vertical  
~~OTHERS~~DRILL PLATFORM  
GROUND SURFACE  
ROCK SURFACE  
BOTTOM OF HOLE  
WATER TABLE

DEPTH	Concrete -ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 0'3.5"	Industrial Topping	0'3.5" - Core break, topping separated from floor during drilling. Break is 90 degrees to axis of core. - Ex-hole well centred.	
0'3.5" to 2'9"	Structural Concrete	0'6" - Sliver of north/south steel recovered on extreme east side of core.	
	Concrete Appears Compact and Sound	1'4" - Core broken manually by drillers, break is approximately 90 degrees to axis of core.	
		1'4" to 2'1" - Corrugation rings indicating "chattering" drilling.	
		2'9" - Sliver of east/west steel on extreme south side of core.	
		2'9" - End of hole. Core broken manually by drillers (approximately 90 degrees to axis). - Ex-hole still good.	

INSPECTOR W. P. Pratt

APPROVED

LOGGED BY W. P. Pratt

DATE

May 31, 1972

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

**DRILLING REPORT**

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC36

SITE West Service Building SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear STARTED p.m. April 11, 1972  
Limited FINISHED p.m. April 11, 1972

METHOD ~~SOX~~ CASING DIAM. \_\_\_\_\_  
 OF Concrete Diamond Drill  
 DRILLING: ~~ROCK~~ CORE DIAM. 5.778 inches

LOCATION: ~~EASTING~~ Third Level ELEVATIONS: DATUM El. 553.75  
~~DEPARTURE~~ West Wall DRILL PLATFORM \_\_\_\_\_  
~~BEARING~~ North End GROUND SURFACE \_\_\_\_\_  
~~INCLINATION~~ Horizontal ROCK SURFACE \_\_\_\_\_  
~~OTHER DATA~~ BOTTOM OF HOLE \_\_\_\_\_  
Bearing West WATER TABLE \_\_\_\_\_

DEPTH	ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
		Hole abandoned due to excessive eccentricity of Ex-hole.	

INSPECTOR W. P. Pratt

APPROVED

LOGGED BY W. P. Pratt

DATE

May 31, 1972

## H. G. ACRES LIMITED — CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

## DRILLING REPORT

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC37

SITE West Service Building SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Limited STARTED p.m. April 12 1972  
FINISHED p.m. April 12 1972METHOD OF DRILLING: ~~SOIL~~ Concrete Diamond Drill CASING DIAM. 5.778 inches  
~~ROCK~~ Third Level CORE DIAM.LOCATION: ~~DATUM~~ West Wall ELEVATIONS: DATUM El. 553.10  
~~DEPARTURE~~ North End DRILL PLATFORM  
BEARING West GROUND SURFACE  
~~INITIAL/DIR~~ Horizontal ROCK SURFACE  
~~OTHER DATA~~ BOTTOM OF HOLE  
WATER TABLE

DEPTH	Concrete ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 2'9"	Structural Concrete	0'0" to 0'0.5"	- Six-inch diameter hole badly off centre. - Recentred - Ex-hole well centred.
	Compact and Sound	0'2.5" Vertical Steel	- No. 8 bars. - Run to the south of centre line. - Bond appears good.
		2'9"	- Ex-hole off centre- toward south of core by approximately 3/8 inch.
		End of Hole	- Core broken manually by drillers (approx- imately 90 degrees to axis).

INSPECTOR W. P. Pratt

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LOGGED BY W. P. Pratt

DATE

May 31, 1972

## H. G. ACRES LIMITED — CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

## DRILLING REPORT

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC38

SITE West Service Building SHEET No. 1 OF 2

CONTRACTOR: Canadian Longyear Limited STARTED p .M. April 12, 1972  
FINISHED p .M. April 12, 1972METHOD OF DRILLING: ~~SOAK~~ Concrete Diamond Drill CASING DIAM. CORE DIAM. 5.778 inchesLOCATION: ~~DATUM~~ Third Level East Wall ELEVATIONS: DATUM El. 553.33  
~~DEPARTURE~~ Northend  
BEARING East  
~~INITIAL DIP~~ Horizontal  
~~OTHER DIPS~~  
DRILL PLATFORM  
GROUND SURFACE  
ROCK SURFACE  
BOTTOM OF HOLE  
WATER TABLE

DEPTH	Concrete ROCK TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 2'10"	Structural Concrete  Appears Compact and Sound	<p>0'0" Ex-hole well centred.</p> <p>0'0" to 0'3.25" Crack, runs longitudinally along core; in cross section view runs from top of core (1" north of centre line) downwards and to the south at approximately 45 degrees to horizontal.</p> <p>Horizontal - No. 8 bars. Steel - Runs below Ex-hole - Two slivers of steel on extreme edges of core. (Believed to be more horizontal steel).</p> <p>0'8.5" Core Break - Core broke while drillers attempted to remove barrel; caused by sliver of steel wedging between concrete core and barrel wall.</p> <p>0'8.5" to 1'2" Corrugation- Indicating "chattering" drill.</p> <p>2'10" End of Hole- Core broken by drillers at approximately 90 degrees to axis of core.</p>	

INSPECTOR W. P. Pratt

LOGGED BY W. P. Pratt

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DATE

May 31, 1972

# DRILLING REPORT

DEPTH	Concrete ROCK-TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
		<p>- Ex-hole offset 0.5 inch toward south of core.</p>	

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

CLIENT Ministry of Transportation and Communications JOB No P2499.02  
 PROJECT Thorold Tunnel Field Investigations HOLE No. OC39  
 SITE West Service Building SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear STARTED a.m. April 13 1972  
Limited FINISHED a.m. April 13 1972

METHOD ~~SOX~~ CASING DIAM. \_\_\_\_\_  
 OF Concrete Diamond Drill  
 DRILLING: ~~ROCK~~ CORE DIAM. 5.778 inches

LOCATION: ~~LATITUDE~~ Third Level ELEVATIONS: ~~BATH~~ Floor 549.83  
~~DEPARTURE~~ Floor DRILL PLATFORM  
~~BEARING~~ North East Area GROUND SURFACE  
~~HEADING~~ Vertical ROCK SURFACE  
~~OTHER DATA~~ BOTTOM OF HOLE  
WATER TABLE

DEPTH	Concrete <del>ROCK</del> TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 0'3.25"	Industrial Topping	0'3.25" Core Break - Core split during drilling at interface between industrial topping and concrete slab. - Ex-hole well centred.	
0'3.25" to 2'10"	Structural Concrete		
	Intact but Slightly Honeycombed	0'5.5" to 0'6.5" - Conglomeration of bars, tightly spaced, estimate 3 bars.  1'4.5" to 2'2.5" - Corragation indicating "chattering" drill  2'10" End of Hole - Embedded pattern of steel visible - Core broken manually by drillers at approximately 90 degrees to axis of core - Ex-hole well centred.	

INSPECTOR W. P. Pratt

LOGGED BY W. P. Pratt

APPROVED

DATE

May 31, 1972



## H. G. ACRES LIMITED — CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

## DRILLING REPORT

CLIENT Ministry of Transportation and Communications JOB No. P2499.02

PROJECT Thorold Tunnel Field Investigations HOLE No. OC40

SITE West Service Building SHEET No. 1 OF 1

CONTRACTOR: Canadian Longyear Limited STARTED p.m. April 13 1972  
FINISHED p.m. April 13 1972METHOD OF DRILLING: ~~SOIL~~ Concrete: Diamond Drill CASING DIAM. \_\_\_\_\_  
ROCK CORE DIAM. \_\_\_\_\_LOCATION: ~~LATITUDE~~ Third Level ELEVATIONS: ~~NATURAL~~ Floor 549.83  
~~DEPARTURE~~ Floor DRILL PLATFORM \_\_\_\_\_  
~~BEARING~~ South East Area GROUND SURFACE \_\_\_\_\_  
~~INITIAL DIP~~ Vertical ROCK SURFACE \_\_\_\_\_  
~~OTHER DATA~~ BOTTOM OF HOLE \_\_\_\_\_  
WATER TABLE \_\_\_\_\_

DEPTH	Concrete <del>ROCK</del> TYPE	DESCRIPTION: COLOUR, TEXTURE, FOLIATION, JOINTING, FRACTURING, FAULTING, ALTERATION, WATER LOSS OR GAIN, CAVING, LOST CORE, CEMENTING, ETC.	% CORE
0'0" to 0'3"	Industrial Topping	0'3" Core Break - Core split during drilling at interface between industrial topping and concrete slab - Ex-hole well centred.	
0'3" to 2'2½"	Structural Concrete	0'5" East/West Steel - No. 8 bars - Bond appears good - Bars run just south of centre line of core  0'6.5" North/South Steel - No. 8 bars - Bars intersect centre line of core  0'11" to 1'5" Corrugation Rings - Indicating "chattering" drill  2'2.5" End of Hole - Core broken manually by drillers at approximately 90 degrees to core axis - Ex-hole well centred.	

INSPECTOR W. P. Pratt

APPROVED

LOGGED BY W. P. Pratt

DATE

May 31, 1972