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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. 3  
West Service Building  
Review of Observed Structural  
Behavior 1971 to 1972



December 1972



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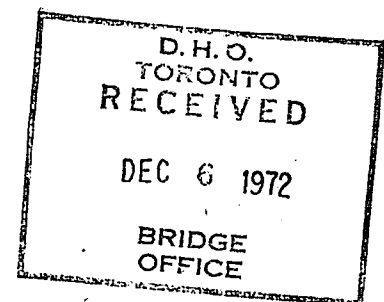
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West Service Building  
Review of Observed Structural  
Behavior 1971 to 1972

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## 1 - INTRODUCTION

### 1.1 - Terms of Reference

At the request of the Ministry of Transportation and Communications in June 1971, Acres Consulting Services Limited initiated investigations into the cause of cracking in the walls of the tunnel, particularly at the west service building. Following submission of reports entitled "Investigations to Determine the Cause of Cracking in the Structure", March 1972, and "Supplementary Report No. 1, West Service Building", May 1972, Acres was authorized to continue monitoring the performance of the service building structures and report on these to the Ministry by the end of 1972.

The results of the monitoring observations at the west service building and an assessment of the results are included in this report. A separate report is being issued for the east service building.

### 1.2 - Scope of the Investigations

By means of continuing monitoring of the deformations occurring in the west service building structure, and visual inspection of cracking patterns, it has been possible to assess the rate of change of deformations and stresses in the structure, and confirm the conclusions reached in the previous Supplementary Report No. 1. Changes in deformation due to rock squeeze have been identified and, following a review of all observations to date, future trends have been predicted. The results have been used to establish criteria for remedial work and allow a comparison to be made of proposed remedial measures.



## 2 - CONCLUSIONS AND RECOMMENDATIONS

The review of field observations made during the period July 1971 to November 1972 has resulted in the following conclusions:

- (a) The conclusions stated in Supplementary Report No. 1, May 1972, have been confirmed. The west service building structure has behaved essentially as predicted under a time-dependent displacement of the rock, referred to as rock squeeze, combined with stresses resulting from changes in temperature.
- (b) The imposed deformation due to rock squeeze applied to the west service building is progressively increasing. The increased compression of the roof of the tunnel between September 1971 and September 1972, shown on Plate 3, amounted to approximately 0.1 inch.
- (c) In addition to increasing progressively the imposed deformation due to rock squeeze has been observed to have an annually fluctuating component, as shown on Plates 3 and 11. This component has a peak to peak magnitude of the order of 0.14 inch, with the maximum compressive effect occurring in October-November of 1971 and 1972.
- (d) The west service building is now sufficiently pre-compressed by the rock squeeze that changes in temperature result only in stress fluctuations, rather than deformations. The stress changes calculated for the reinforcing steel fluctuate over a range of approximately 12,000 psi in the roof struts, with the peak occurring in the months of July and August.
- (e) Stress levels in the outer tunnel walls and the roof structure are generally high, and in some instances are considerably in excess of normal design limits.
- (f) Areas of stress concentration have been delineated in the struts between the air exhaust holes in the tunnel roof at the west service building. Peak stresses in 1972 in the reinforcing steel, due to rock squeeze and thermal changes, have been estimated to be 32,300 psi and 44,900 psi in the north and south roof struts respectively (nominal yield stress of steel is 50,000 psi).



- (g) Compressive strains have continued to increase in the tunnel roof struts, but, as yet, are believed not to be of sufficient magnitude to cause internal microfracturing of the concrete. Strain levels consistent with this advanced stage of structural deterioration could possibly be reached in as short a period as 2 to 3 years, if no remedial work is carried out.
- (h) Should the roof strut be allowed to reach failure level the roof slab will be seriously overstressed. The exact repercussions of this on the overall performance of the west service building is somewhat unpredictable, but major cracking would be inevitable, and failure of other elements in the structure would probably be hastened.
- (i) Cracks in the tunnel walls have continued to widen, and new cracking above tunnel roof level in the west service building has been observed since August 1972, indicating a worsening of the condition of the walls. Extensometer readings in the rock behind the tunnel walls indicate that some dilation of the rock mass is occurring.
- (j) Observed tunnel deformations attributable to rock squeeze for 1971 to 1972 show good correlation with similar observations in the Canadian Niagara Power Company's wheel pit in Niagara Falls. The latter has continued to have a progressive movement with the superimposed annual fluctuation for 70 years, and based on this precedent, there is no reason to believe that the progressive compression of the tunnel will cease in the foreseeable future.
- (k) It is predicted, see Plate 12, that over the next 70 years the inward movement of each wall of the tunnel will be of the order of 0.7 inch. This figure is based on data from the Canadian Niagara Power Company's wheel pit, with which the 1 year's observations in the tunnel are in agreement. Due to the short duration of observations at Thorold, this prediction must be used with considerable caution.
- (l) The only really effective means of preventing 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 as close to them as possible.
- (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 relieve the structure, although a non-symmetrical loading condition



will be created, resulting 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 possibly new cracking in the north wall, these changes will not be sufficient to require the second slot to be installed for several years.

- (n) Observations at the east service building indicate that this structure is not seriously stressed at this time. The duration of observations to date is insufficient to establish if rock squeeze will apply significant restraint to the structure in the future, but a program of observation is continuing to identify the problem.

On the basis of the above conclusions we recommend that:

- (a) Remedial work in the form of slots in the rock to remove the loading due to rock squeeze from the west service building, and thereby stop the deteriorating condition of the cracked tunnel walls, must be completed not later than July 1973.
- (b) This remedial work should include measures for rehabilitating the cracked walls, mainly comprising sealing and grouting cracks. If it is decided that the slot on the north side should be deferred for several years, then rehabilitation of the cracked walls may have to be carried out in stages accordingly.
- (c) The monitoring of deformations and stresses in the west service building should be continued after the remedial work is completed, so that the effectiveness of the measures can be assessed.
- (d) Instrumentation of the rock mass adjacent to the tunnel should be installed to determine changes in rock squeeze during and after cutting of the slot. This instrumentation will provide data to enable a more confident assessment to be made of the need for future remedial measures.



### 3 - FIELD INSTRUMENTATION

The methods used to determine stresses and strains in concrete and reinforcing steel were described in Supplementary Report No. 1 of May 1972. Monitoring observations since May 1972 have utilized instrumentation as follows, see Plate 1:

- (a) Demec mechanical strain gauges across full width of west service building.
- (b) Invar extensometers across full width of west service building.
- (c) Demec mechanical strain gauges on reinforcing steel in top of roof slab.
- (d) Electrical resistance strain gauges on reinforcing steel in top of roof slab, elevation 549.83 feet.
- (e) Demec mechanical strain gauges across cracks in tunnel walls.
- (f) Extensometers embedded into rock through south tunnel wall.
- (g) Thermocouples in south tunnel wall and roof slab.
- (h) Laser extensometers installed by Ministry of Transportation and Communications on top and bottom of roof slab.

In addition to the above, ambient air temperatures were monitored outside the west service building, and related to those taken on the inside surface of the south tunnel wall and above the tunnel roof in the west service building. Water temperature measurements were made in the Welland Canal adjacent to the tunnel and in the west sump of the south tube. Ground water pressures outside the tunnel walls were measured by means of piezometers drilled through the walls.

In September 1972 an additional extensometer was installed across the south tube at road level but no meaningful result was obtained from this at the time of writing this report.



#### 4 - FIELD OBSERVATIONS

The results of the continuing field observations obtained in the period July 1971 to October 1972 are reviewed in detail.

##### 4.1 - Strain Measurements

Strain measurements taken on the tunnel roof are shown on Plate 2. These indicate good agreement between strains measured by various methods, that is, electrical resistance strain gauges on the reinforcing steel, and mechanical and laser strain gauges on the structural concrete. Comparison of strains on the top and bottom of the roof structure indicate that this member is acting primarily as a strut under axial compression.

A reassessment of earlier strain measurements was made in the light of the strain distributions obtained from more comprehensive instrumentation. This resulted in some changes in the deformations determined from individual strain measurements as given in the report of May 1972.

The observed distribution of strains in the roof shows that a ratio of 1:1.8 is obtained between strain in the slab and the strain measured in the struts between the exhaust openings. This observed value is in reasonable agreement with a previously estimated figure of 1:1.5, assessed for an idealized structure under an imposed displacement. The equivalent figure for the structure under an imposed load condition would be of the order of 1:2.4. The ratio of strains measured between May and November 1972, in the struts between exhaust openings in the roof slab over the north and south tubes, and those between air intake holes at the same elevation, 549.83 feet, was 1:0.8.

Artificial cooling of the roof struts between exhaust openings at elevation 549.83 feet achieved a reduction in temperature of 20 degrees F relative to the roof slab. This caused a reduction in the compressive stress in the north and south roof struts with no appreciable measured strain, as shown on Plate 3. The equivalent reduction in compressive stress in the reinforcing steel in the roof struts was approximately 3,000 psi, see Plate 4.

Strain gauge readings on reinforcing steel in the south tunnel roof strut in September appear to be of doubtful accuracy. Based on experience elsewhere, it is queried if



BUT: Plate 5 does not agree with Plate 3?



in fact readings from any of the electrical strain gauges will be reliable longer than a period of 6 months after installation. Future monitoring will rely primarily on extensometers and Demec mechanical strain measurements.

#### 4.2 - Deformation Measurements

Deformation measurements taken with extensometers across the west service building indicate good agreement with those obtained by integration of the 8-inch Demec gauge length measurements. A comparison of the deformation of the tunnel observed by both of these methods is shown on Plate 5, where readings from the extensometer at the false roof level and those obtained from strain measurements on the roof are presented. In both cases the minimum dimension of the structure was observed to be in November 1972, and the yearly maximum in May, apparently with a progressive decrease in dimension over a yearly cycle.

Strain measurements at various locations along the length of the tunnel indicate a variation in the dimensional change of the south tube during the period June 3 to September 1, 1972, as shown in Table 1. To confirm the pattern of deformation an additional extensometer was added across the south tube at road level, with readings commencing in September 1972.

#### 4.3 - Wall Extensometers

Extensometers through the tunnel wall and anchored in the bulkhead concrete and the rock have shown significant movements, as shown on Plate 6. These movements vary with depth with the largest movement for the extensometer anchored deepest in the rock. The progressive movement of the rock indicated by the longest extensometer is approximately 0.026 inch in the past year.

#### 4.4 - Temperature Measurements

Atmospheric air temperature measurements compared to those in the tunnel are shown on Plate 7. Temperatures recorded in the tunnel roof of the west service building indicate a minimum temperature of 23 degrees F in January 1972, and a



maximum of 80 degrees F in July 1972. Wall temperatures, canal and west sump water temperatures, were less variable, but significant fluctuations were recorded.

#### 4.5 - Crack Widths and Visual Observations

Crack widths in the south tunnel wall, shown on Plates 5, 8 and 9, have changed significantly, reducing in the winter, with a minimum in March, and widening in the summer and fall. The crack widths measured in September 1971 were exceeded by August 1972, and have continued to enlarge since that date.

The pattern of cracking in the tunnel wall is essentially the same as that previously surveyed in May 1971, but the cracks have propagated slightly and become more evident. Fresh cracks have occurred in the north and south walls of the west service building above tunnel roof level, as shown on Plate 8. The largest crack in this area was approximately 0.030 inch wide in February 1972, and has increased by 0.012 inch between June and October 1972.

#### 4.6 - Recalibration of Stress Measurements

During compilation of continuing field observations a number of discrepancies were noted between changes in strain measured by electrical and mechanical strain gauges. To resolve the conflict the electrical strain measuring instrument was recalibrated using reinforcing bars cut from the structure.

The recalibration showed that an error had been made in the previous interpretation of strain measurements due to ambiguous operating instructions on the instrument. This resulted in a significant increase in compressive stresses measured in reinforcing steel over the values shown in Supplementary Report No. 1. The revised figures are given on Plate 10.

Comparison of overcoring test results and steel stress measurements resulted in a range of values for effective modular ratio of between 13 and 20, with an average value of approximately 15.



Comparison of the in situ stresses in April 1972, with the concrete stress measurements in October 1971, is good when account is taken of the observed structural deformation, thermal changes, and shrinkage stresses created in the structure as shown in Table 2.



## 5 - ANALYSIS OF RESULTS

Observations made since May 1972 have confirmed that the structure is deforming essentially as predicted at that time. Cyclical changes in stress due to temperature variation, and imposed deformation due to rock squeeze are occurring, and there is a progressive increase in the compression of the structure. The completion of a 1-year cycle of readings has provided greater confidence in the predictions, and in the data upon which they are based.

Strain measurements in the tunnel roof struts indicate that there has been an inward compression of the west service building, amounting to approximately 0.1 inch between the peak deformations in 1971 and in 1972.

The mode of structural behavior of the strut has been substantiated by strain measurements on the top and bottom of the structural member, and these have been verified by extensometer measurements, as indicated on Plate 5. Extensometer measurements, taken across the tunnel at roof level at different locations, have indicated that the largest deformation is at the east end and the smallest at the west end of the bulkheads. This is particularly evident in the measurements taken in the south tube. These deformations, shown in Table 1, appear to be consistent with a hypothesis that the rock squeeze and associated crack pattern in the tunnel wall is occurring predominantly at the shaly limestone layer.

The observations shown on Plate 5 confirm the conclusion of May 1972, that due to precompression by rock squeeze the width of the west service building does not vary with changes in temperature. This was confirmed previously by in situ stress measurements, and also by the observations at the east service building, the width of which does appear to vary with changes in temperature. It is concluded that the seasonal fluctuations in deformation of the structure which do occur are due to changes in rock squeeze.

The seasonal component of the deformation of the west service building due to rock squeeze is 0.14 inch, which is equivalent to a fluctuation of steel stress in the roof struts of 7,200 psi, see Plate 3.

Because the west service building is subjected to full restraint by the rock, changes in temperature cause a stress change in the structure, without a comparable change in strain. In 1971 to 1972 this has caused a change in steel stress in the roof strut of 12,000 psi, with the peak thermal stress occurring in July-August.



By superposition of the components of the deformation and stress changes in the structure the stress fluctuations for 1971 to 1972 have been derived, and are shown on Plate 3. Peak stress occurred in September, while the peak deformation occurred in November. By integrating the stress fluctuations occurring in September 1972 with the in situ stresses observed in March 1972, peak steel stresses were derived for north and south roof struts, and they were of the order of 32,300 and 44,900 psi respectively, see Table 3.

The fluctuating and progressively increasing squeeze of the west service building has been verified by measurement and the development of additional cracking in the tunnel walls. Observations of crack width changes and movements of extensometers in the rock have shown a net increase over the annual cycle.

The observed deformations at the Canadian Niagara Power Company wheel pit and at Thorold have been used to develop a hypothetical curve for imposed deformation due to rock squeeze, as shown on Plate 11. This curve has been adjusted to take into account the natural shrinkage of the concrete structure. Extrapolation of the deformation curve into the future has been based on observations in the Canadian Niagara Power Company's wheel pit, as given in the original report of March 1972. These observations correlate well with the tunnel measurements when plotted on a semilog scale, with the time scale adjusted with respect to initial excavation, see Plate 12. Based on this curve, for the period November 1972 to November 2022, the north and south tunnel walls will each have an estimated inward closure of the order of 0.7 inch. This figure is based on extremely limited knowledge of the actual mechanism of rock squeeze at Thorold, and hence, its use in predicting future behavior of the west service building must be treated with caution.



In other words, the temperature coefficient and/or the modulus are not constants!



## 6 - REVIEW OF STRUCTURAL SAFETY

It has been determined that the structural safety and operational integrity of the west service building is governed by stress and deformation conditions in specific structural elements, and these are reviewed separately.

### 6.1 - Tunnel Roof

Continuing observations have confirmed the conclusion of Supplementary Report No. 1 that the tunnel roof is acting primarily as a strut. In addition, areas of stress concentration caused by the configuration of the air intake and exhaust openings have now been clearly defined by strain measurements. These measurements have confirmed that maximum compressive stresses occur in the tunnel roof struts, with the stress 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, but both are in excess of accepted design limits, and sufficiently high that long-term factors of safety will be considerably reduced.

Continuing strain observations have shown that the changes in strain in the roof struts and the total deformation of the north and south tubes at roof level are essentially equal. In situ stress levels determined from strain measurements in reinforcing steel cut from the roof structure, as noted in the previous report, are generally higher over the south tube. This difference, although indicated by a small number of measurements, is considered to be attributable to the compressible nature of the fibreboard installed outside the north wall of the tunnel.

The ratio between strains measured in the north and south struts between air exhaust openings, and in the centre struts between air intake openings, is of the order of 1:0.8, and with that measured in the floor slab is approximately 1.8:1.

Measurements have indicated that the stress in the tunnel roof structure at the west service building, and the adjacent sections of tunnel roof, are significantly influenced by both thermal changes and applied deformation due to rock squeeze. Thermal changes create large stress fluctuations with small strain changes due to redistribution, and thus are not amenable to direct measurement.



The reinforced concrete struts can be considered similar to laterally tied columns, whose structural behavior can be considered to pass through the three phases defined in Supplementary Report No. 1, as follows:

Phase 1 - Range of strain - 0 to approximately 0.04 per cent; normal behavior 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 microfracturing 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 microfracturing 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 compressive stress measured in the concrete of the south roof strut in October 1971 was 2,500 psi, which is comparable to a compressive strain of 0.12 per cent. The net increase in compressive deformation applied to the west service building from September 1971 to September 1972 was observed to be 0.1 inch, which corresponds to a compressive concrete strain in the strut of 0.03 per cent for the past year. This figure has been integrated with the long-term data on rock squeeze which is available, namely from wheel pit observations at the Toronto and Canadian Niagara Power Companies plants at Niagara Falls, the latter being plotted on a semilog scale on Plate 12. From this curve the total deformation of the west service building can be predicted, leading to the conclusion that progressive microfracturing of the concrete in the roof struts, symptomatic of significant permanent deformation of the structure, can be expected to begin in as short a period as 2 to 3 years.



This corresponds to a compressive stress in the reinforcing steel in the struts of 60,000 psi, and in the tunnel roof slab of 33,000 psi. As predicted in May 1972 the tunnel roof slab has already entered the Phase 2 condition.

There are obviously limitations to the applicability of the long-term wheel pit data in predicting rock squeeze at Thorold over a short period, and it could well be that the measured rate of increase of rock squeeze in 1971 to 1972 is not typical. The safety factor of the roof strut is already considerably reduced, and a small unexpected acceleration in the rate of rock squeeze would seriously affect the rate of deterioration of this element of the structure.

Failure of the roof strut is unacceptable since the roof slab will also be seriously overstressed by the time the strut fails. The effect of this on the overall performance of the west service building is somewhat unpredictable, but further major cracking is considered to be inevitable, and failure of other elements in the structure would probably be hastened.

It was necessary in September and October 1972 to cool the north and south tunnel roof struts artificially to maintain stress levels approximately equal to those of 1971. In September 1972 the slab was generally at 70 degrees F, while the struts were maintained at approximately 50 degrees F, and this caused a stress reduction of approximately 3,000 psi in the reinforcing steel.

## 6.2 - Tunnel Wall

The cracking pattern observed in the south tunnel wall at the west service building in November 1972 is shown on Plates 8 and 9. The width of the cracks in the tunnel wall have varied significantly over the past year. These variations have been symptomatic of flexure of the wall, with a peak width occurring in November. Generally, the crack widths of 1972 have exceeded those measured in 1971, indicating that the cracks have penetrated deeper into the wall than the 3 feet 6 inches observed in October 1971. As noted in May 1972 the wall is already well advanced into a Phase 2 condition. It is estimated from changes observed in 1971-1972 that areas of the wall will reach Phase 3 within 8 years or so, if no remedial work is done. At this stage progressive collapse of the structure will be considered to have commenced and be evidenced initially by minor spalling.



Above tunnel roof level on the west service building new cracking has occurred in the north and south walls, as shown on Plate 8. Also, cracks have continued to enlarge at a significant rate. The major crack in this area was approximately 0.030 inch wide in October 1971, with an increase of about 0.012 inch measured between June and October 1972. It is considered that corrosion of the steel exposed at the cracks may reduce the time required to reach the Phase 3 condition.

Observations in November 1972 indicate that water is seeping through a number of cracks in both north and south outer walls of the tunnel at the west service building, and a continuing leaching of material is apparent from these cracks. Although corrosion of reinforcing steel exposed in the cracks has not been confirmed, the amount of rust which is readily visible on steel exposed in open drill holes is substantial.

It has been concluded that the only logical explanation for cracking at each end of the south wall under the west service building is that it is caused by failure of concrete in shear. This is substantiated by stress measurements in the tunnel floor in October 1971, which indicate that shear stress of 370 psi exists in the base of the wall. There is also a horizontal crack approximately 0.020 inch wide running into the haunch at the east end of the south tunnel element, monolithic with the west service building, and a crack at this location can only be caused by a shear failure of the concrete.

### 6.3 - Concrete Bulkhead

The concrete bulkheads, between the tunnel structure and the vertical surface of the rock, form a dam to prevent canal water from entering the west portal of the tunnel. There is evidence from the drill hole records reported previously that there is substantial cracking already in the south bulkhead. If the amount of cracking in the bulkhead is allowed to increase, then the clay dike founded on the bulkheads may become exposed to the risk of subsurface erosion at the base. Warning of this should be obtained if movement of material is observed in seepage water entering the west pump sump. None has been noted yet.



Try slot on south only and hope tunnel moves south before pressure on north side becomes critical. (But keep monitoring!).



## 7 - REMEDIAL MEASURES

Any remedial measures adopted must ensure that any structural failure within the west service building during the life of the tunnel neither endangers life and property, nor disrupts normal traffic use of the facility. To accomplish this objective it is necessary to guarantee that two requirements are fulfilled, namely, that stresses in the tunnel roof slab and walls are reduced to acceptable levels, and that the imposed deformation due to rock squeeze is eliminated from the walls of the structure.

Two basic types of remedial measures have been considered, namely those that apply no restraint to the rock, so removing or minimizing the rock squeeze applied to the structure, and those based on restraint of the rock. Schemes developed for each of these are as follows:

### A - No Restraint of Rock

- 1 - Remove rock deformation from structure.
- 2 - Modify structure to reduce internal strain to acceptable level.
- 3 - Cooling of structural members.

### B - Restraint of Rock

- 1 - Post-tensioned tendons in rock.
- 2 - Stiffening the structure.
- 3 - Strutting across structure.

These six schemes are illustrated on Plate 13, together with a brief summary of pertinent comments for each.

As concluded in the earlier reports, it appears that the two requirements described above can only be completely fulfilled by physically isolating the west service building from the effects of rock squeeze. This is best accomplished by means of slots cut in the rock north and south of the structure. This will ensure the maximum reduction in stress in the structure, while ensuring that no future build-up of stress can occur due to rock squeeze.

Based on the earlier comments on the rate of deterioration in the west service building, it is concluded that the remedial measures must be implemented before July 1973.



It is assumed that, following completion of the remedial work as described, it will be necessary to seal the cracks to prevent further corrosion of the reinforcing steel, and to allow restoration of the structural surface in the tunnel.

*Suggest delaying cosmetic treatment of wall for at least a year in case further deformations do occur.*



Table 1

DIMENSIONAL CHANGES AT VARIOUS  
LOCATIONS IN TUNNEL  
JUNE 3, 1972 TO SEPTEMBER 1, 1972

*(presumable they are all shortenings?)*

	<u>South Tube</u>	<u>Total Tunnel</u>	
South tunnel roof east of west service building Station 54+84	<i>C</i> 0.042 inch	0.107 inch	<i>probably on ceiling slab?</i>
Centre line of roof struts (line A)	<i>A</i> 0.048 inch	0.122 inch	<i>under side of roof</i>
False roof (extensometer)	<i>D</i> 0.045 inch	-	
Roof slab (line B)	<i>B</i> 0.028 inch	0.103 inch	

Note: For location of measurements see Plate 1.



Table 2

PREDICTION OF STRESSES IN SOUTH STRUT  
NOVEMBER 2, 1971

---

Steel stress - April 26, 1972 -32,700 psi

Note: Revised value due to  
recalibration of instrument  
(Based on measured strains)

Steel stress change - November 2,  
1971 to April 26, 1972 (Based on  
measured strain and thermal changes)

---

Steel stress - November 2, 1971 = -38,400 psi

Average modular ratio

15 (measured)

Concrete stress  
November 2, 1971

2,560 psi

Reduction in concrete compressive  
stress due to shrinkage  
(calculated on basis of east  
service building observations)

---

+ 667 psi

Calculated concrete stress

= 1,893 psi

Measured concrete stress

1,900 to 2,700 psi  
(2,500 psi average)

Effective modular ratio to  
equate calculated and measured  
concrete stresses

12.2

Note: Positive indicates  
compression or increase  
in compressive stress.



Table 3

PEAK REINFORCING STEEL STRESSES  
IN SEPTEMBER 1972

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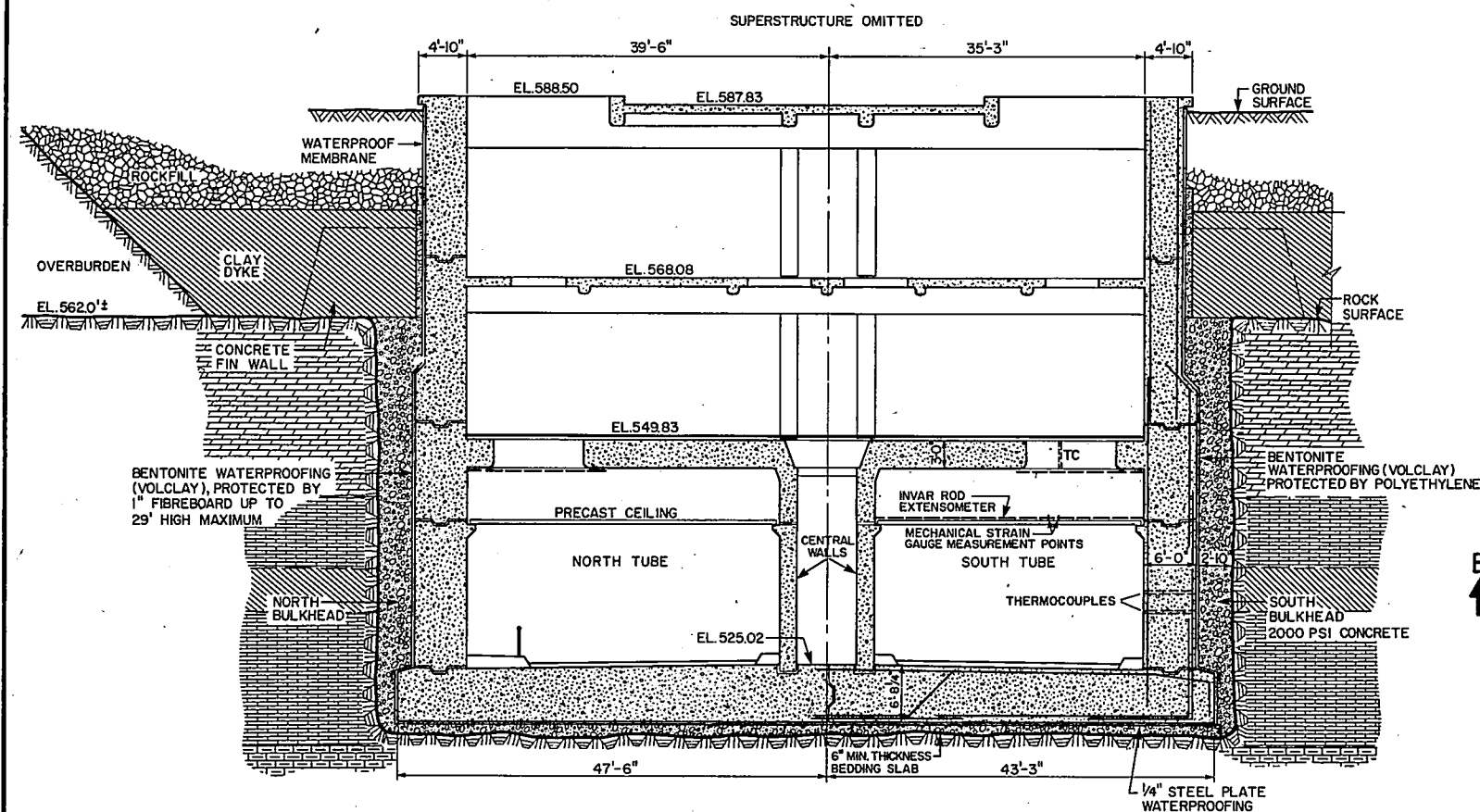
Steel Stress <u>April 26, 1972</u>	South Strut <u>32,700 psi</u>	North Strut <u>20,100 psi</u>	Roof Slab <u>12,000 psi</u>
Thermal steel stress change - April 26 to September 1, 1972 (Based on measured thermal changes)	6,400	6,400	5,800
Steel stress change due to rock squeeze April 26 to September 1, 1972 (Based on measured strain changes)	5,800	5,800	4,400
Peak steel stress September 1, 1972	44,900	32,300	22,200
Equivalent concrete stress ( $m = 15$ )	3,000	2,200	1,500
Reduction due to shrinkage (Estimated)	670	670	670
Peak concrete stress	2,330	1,530	830
Using effective modular ratio required to equate calculated and measured concrete stresses ( $m = 12.2$ , see Table 2)	2,970	1,950	1,150

Note: Positive indicates  
compression or increase  
in compressive stress.



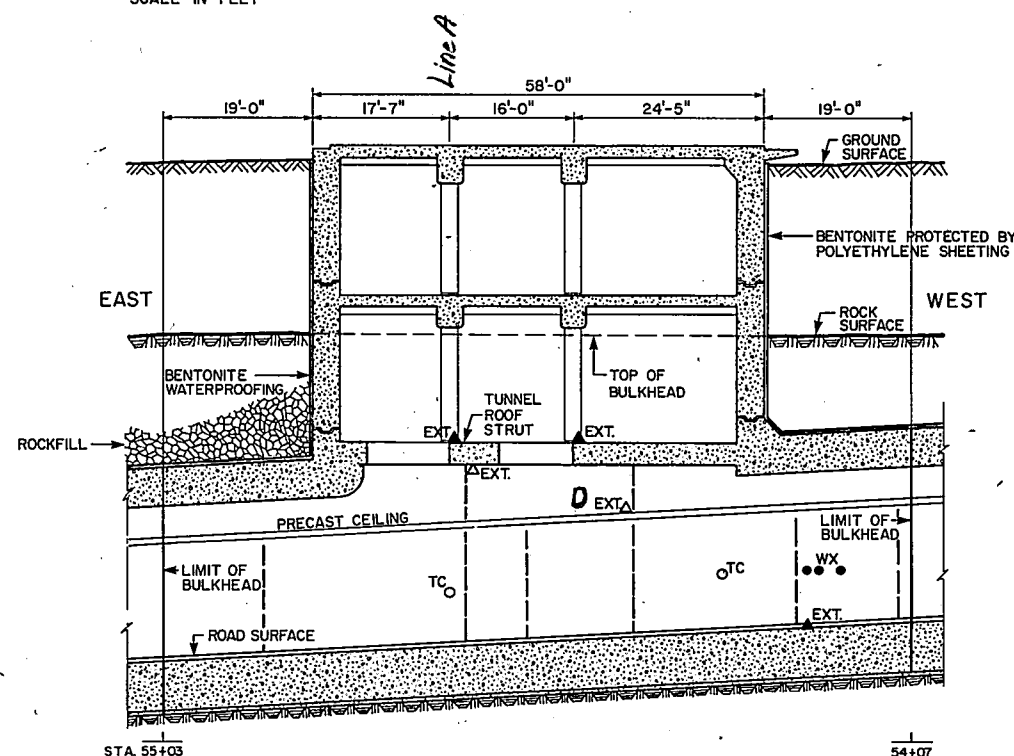
PLATES





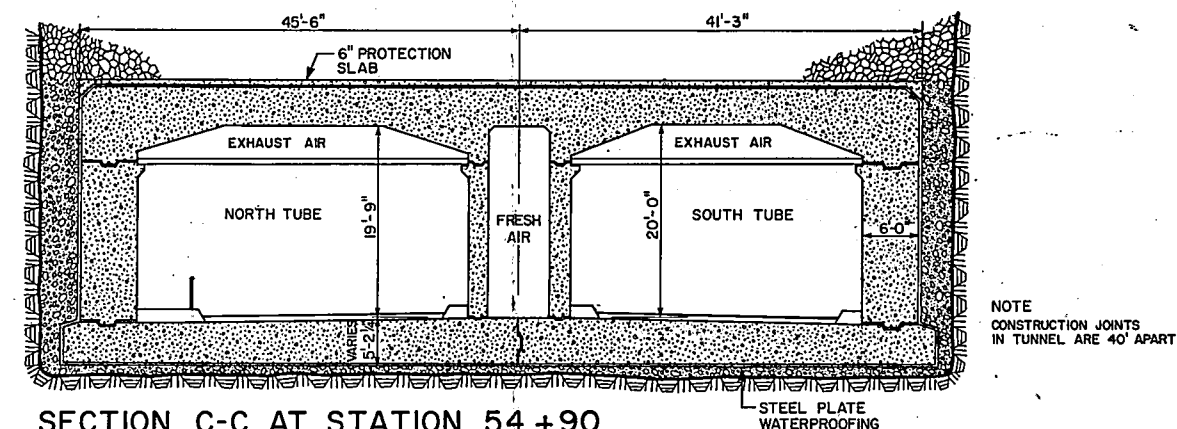
SECTION A-A AT STATION 54+55

SCALE IN FEET



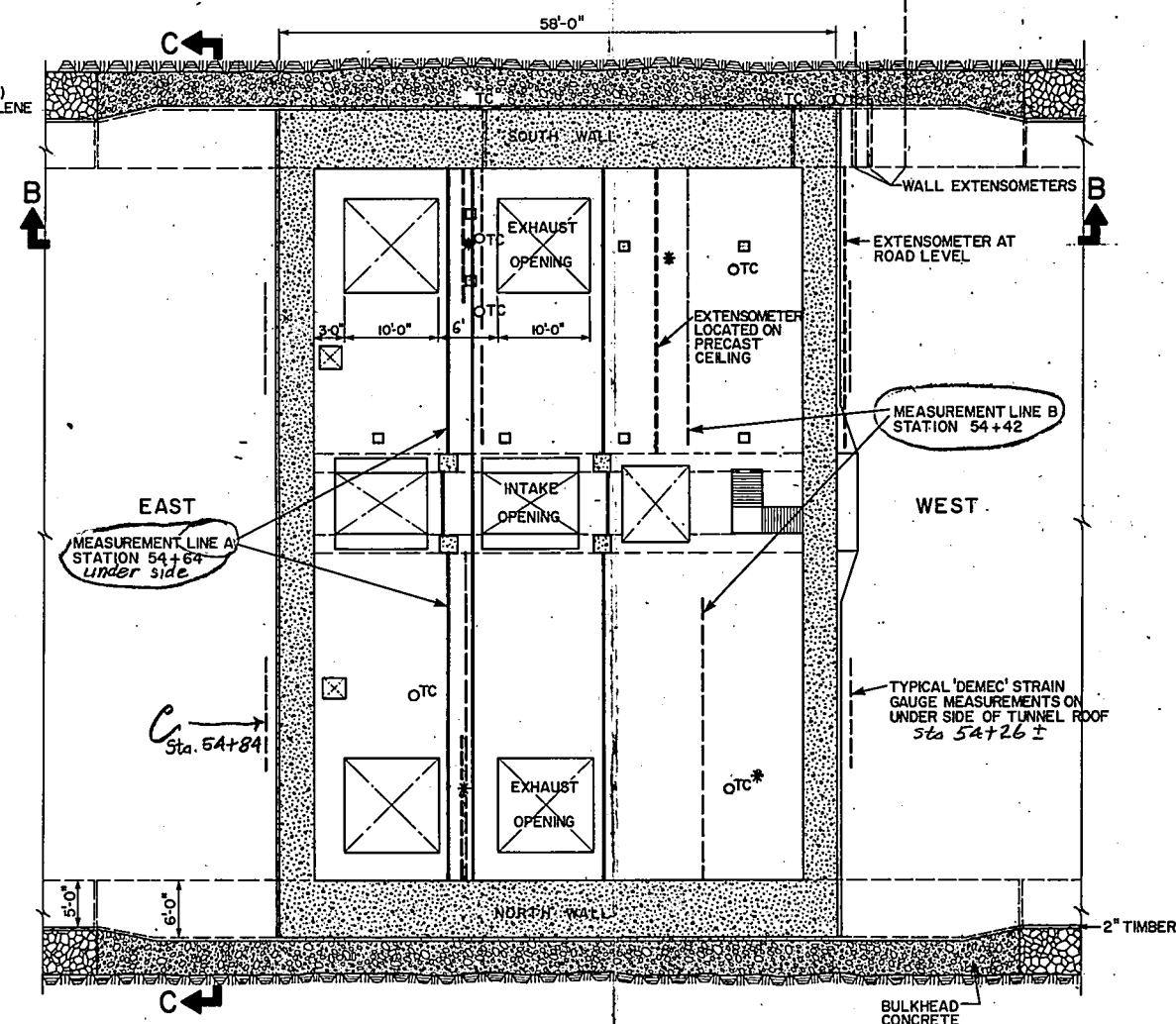
SOUTH TUNNEL WALL - SECTION B-B

SCALE IN FEET



SECTION C-C AT STATION 54+90

SCALE IN FEET



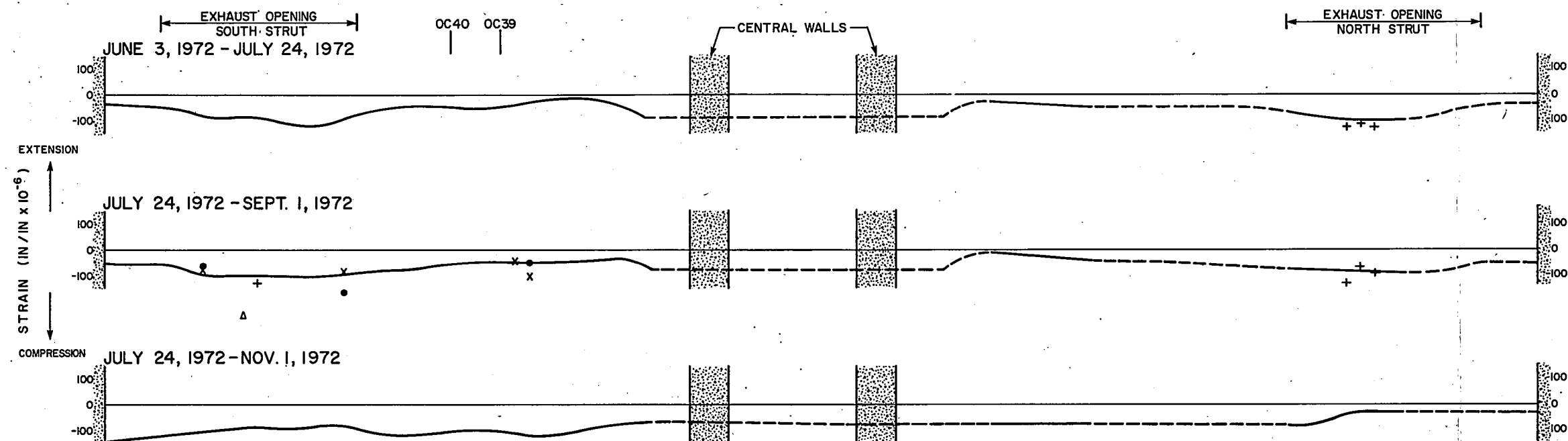
PLAN AT EL. 549.83

SCALE IN FEET

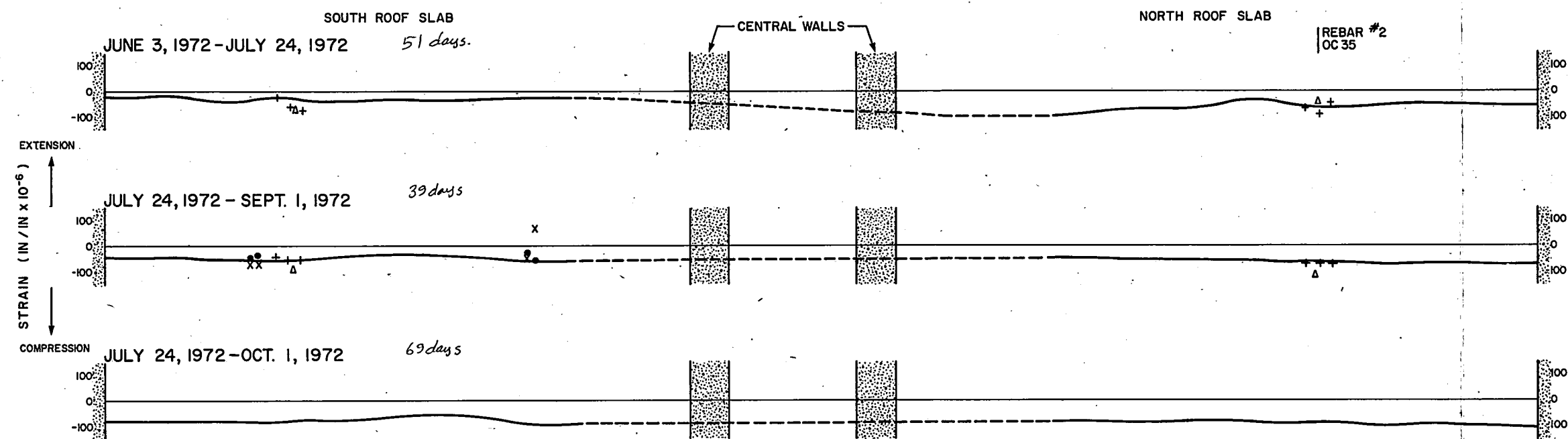
# LEGEND

- WX• TUNNEL WALL EXTENSOMETER POSITIONS
- TC○ THERMOCOUPLE INSTALLATIONS
- LINE OF 8" 'DEMEC' STRAIN GAUGE MEASUREMENT TOP AND BOTTOM OF ROOF SLAB AND SURFACE OF SOUTH TUNNEL WALL
- EXT. --- TOP OF ROOF SLAB EXTENSOMETERS
- EXT. --- BOTTOM OF ROOF SLAB EXTENSOMETERS
- LASER EXTENSOMETERS TOP AND BOTTOM OF ROOF SLAB (INSTALLED BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS)
- \* INSTRUMENTED REINFORCING STEEL TOP OF ROOF SLAB ONLY





LINE A - CENTER LINE OF ROOF STRUTS STATION 54 + 64



LINE B - ROOF SLAB STATION 54 + 42

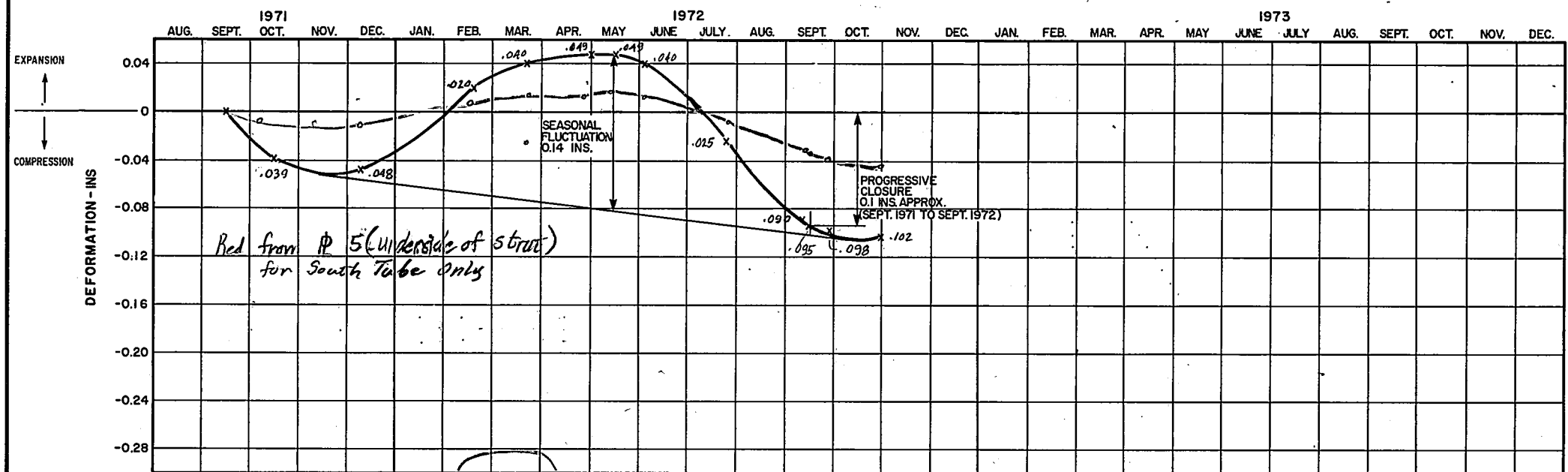
LEGEND

- + CONCRETE STRAIN (TOP)
- + DEMEC MECHANICAL STRAIN GAUGE
- x TOP } MINISTRY OF TRANSPORTATION AND
- BOTTOM } COMMUNICATION STRAIN MEASUREMENTS
- Δ ELECTRICAL RESISTANCE STRAIN GAUGES (TOP STEEL)
- CONCRETE STRAIN DISTRIBUTION DETERMINED BY STRAIN MEASUREMENTS AT 8" CENTRES
- - - INTERPOLATED STRAIN DISTRIBUTION

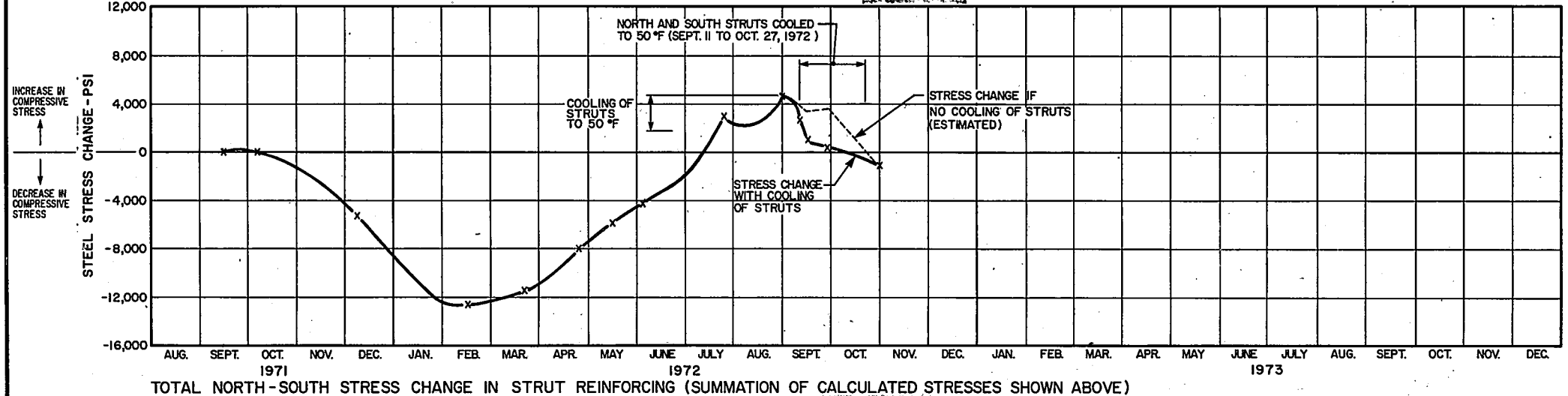
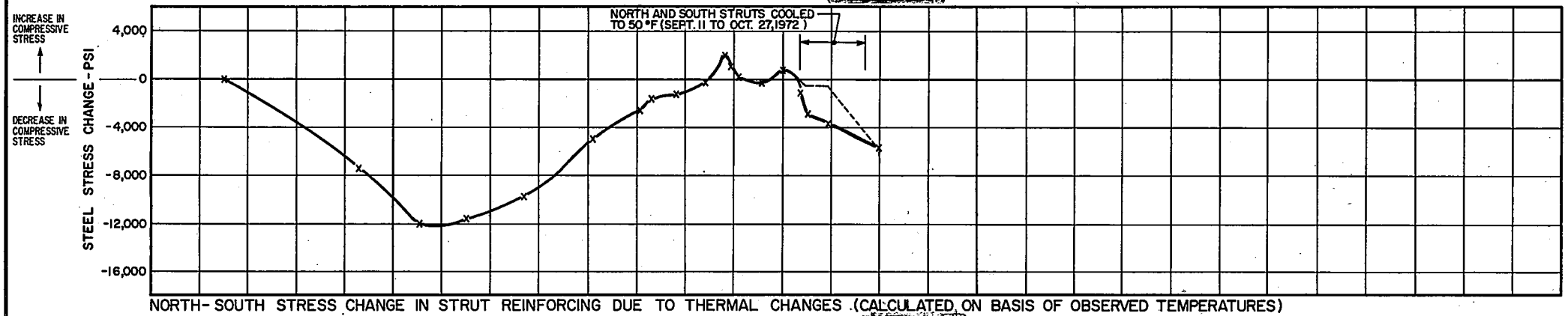
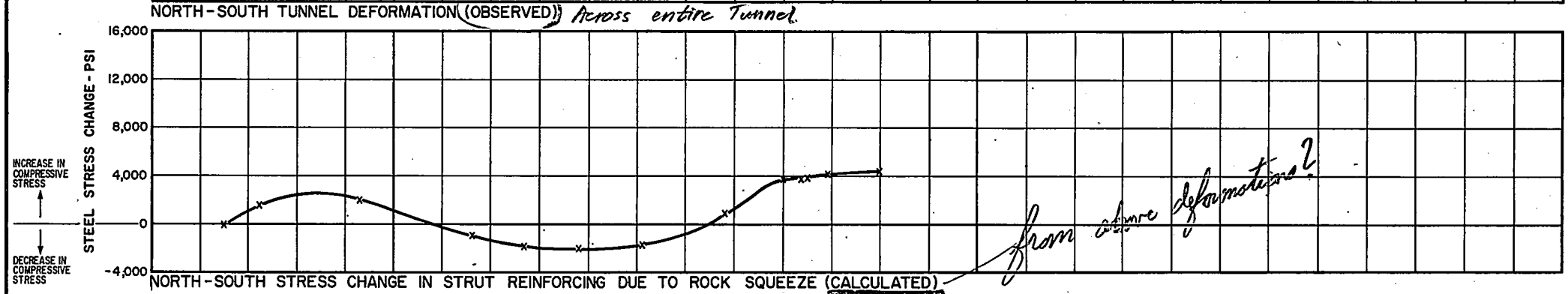
NOTE

FOR LOCATION OF MEASUREMENT LINES A AND B, SEE PLATE 1

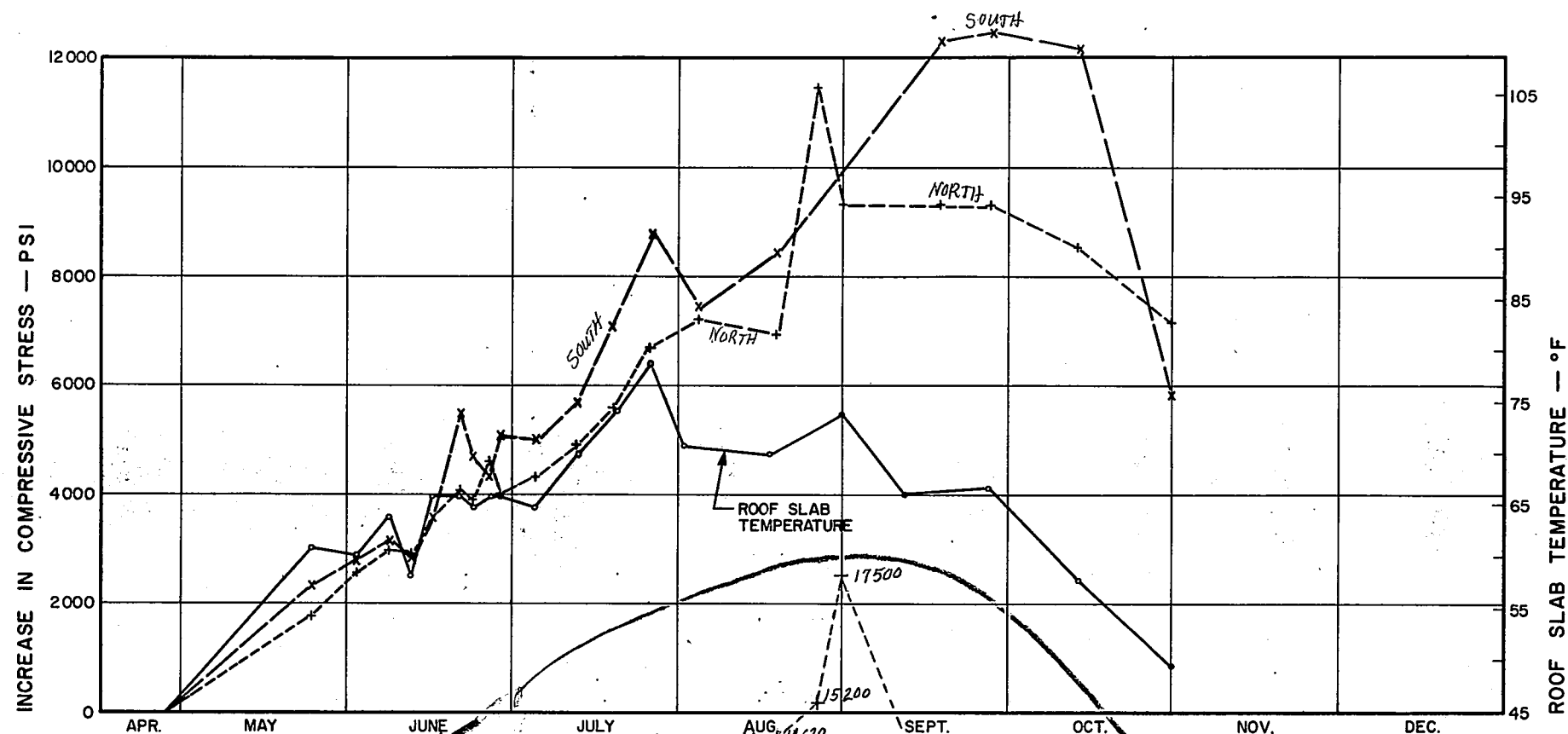




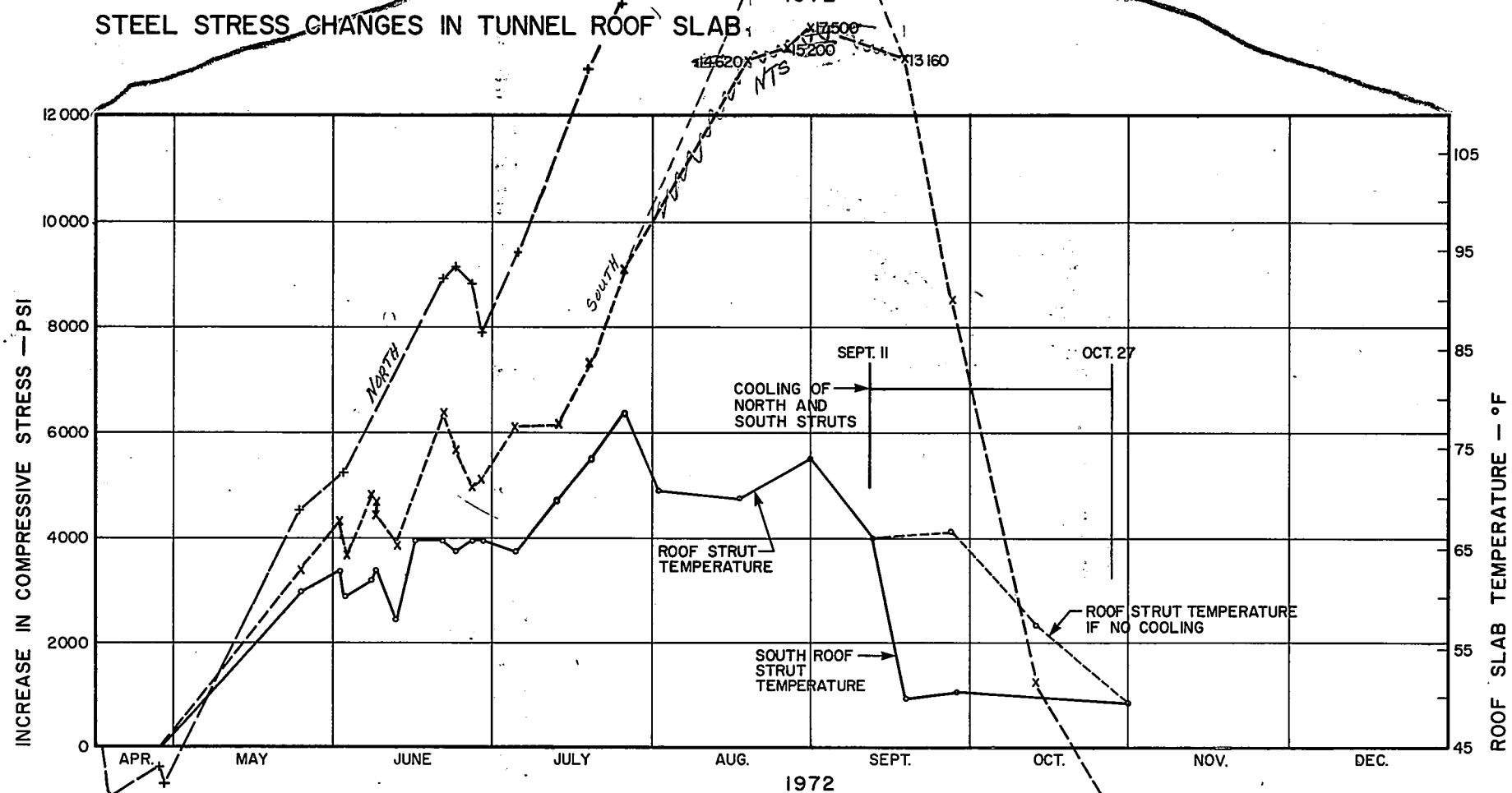
NOTE  
DEFORMATION AND STRESS CHANGE OBSERVATIONS  
STARTING FROM SEPTEMBER 15, 1971







STEEL STRESS CHANGES IN TUNNEL ROOF SLAB



STEEL STRESS CHANGES IN TUNNEL ROOF STRUTS

#### LEGEND

- ROOF SLAB TEMPERATURE
- + TOP OF TUNNEL ROOF SLAB ABOVE NORTH TUBE #2
- x TOP OF TUNNEL ROOF SLAB ABOVE SOUTH TUBE #3

#### LEGEND

- ROOF SLAB TEMPERATURE
- x TOP OF SOUTH TUNNEL ROOF STRUT
- + TOP OF NORTH TUNNEL ROOF STRUT (DISCONTINUED AFTER JULY AND REPLACED BY EXTENSOMETER FOR FULL LENGTH OF STRUT)

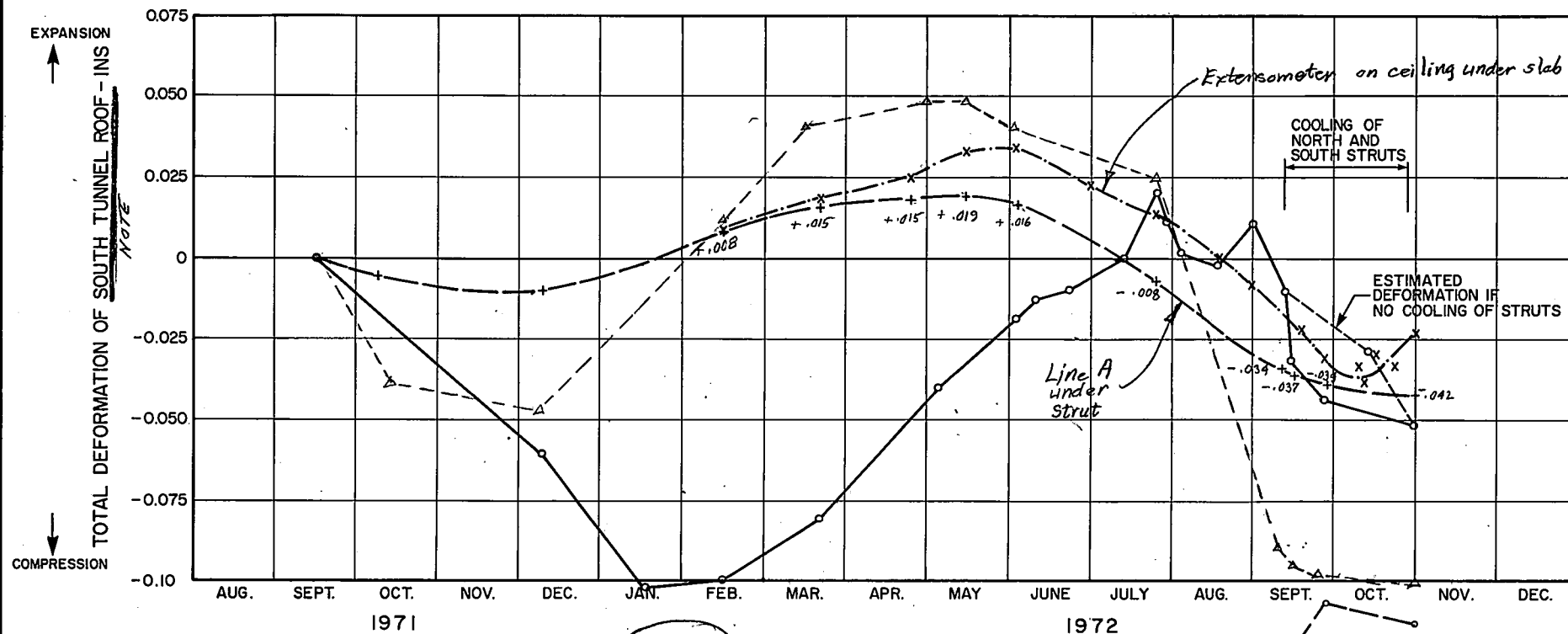
#### NOTES

FOR LOCATION OF REBARS SEE PLATE 1 AND 10

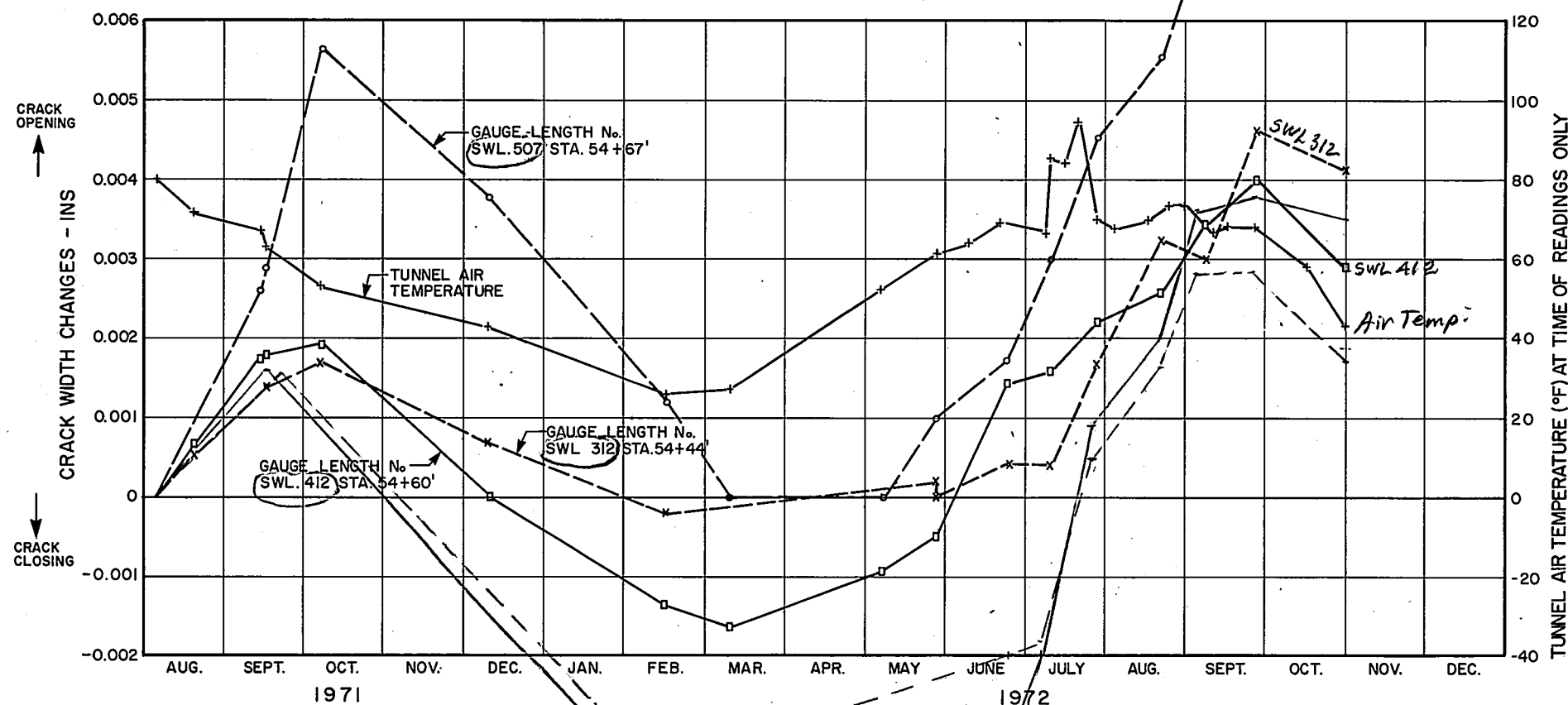
STEEL STRESS CHANGES CALCULATED USING MEASURED STRAIN AND TEMPERATURE CHANGES

FOR STRESSES IN APR '72, see Plate 10





TUNNEL ROOF DEFORMATION (LINE A) - CENTER LINE OF ROOF STRUTS STATION 54+64



CRACK WIDTH AND TUNNEL TEMPERATURE CHANGES WITH TIME

Red - East Bldg.  
North Wall.

Red plot from #3 (N-S Tunnel Deformation, Observed)  
measured across entire tunnel.

#### LEGEND

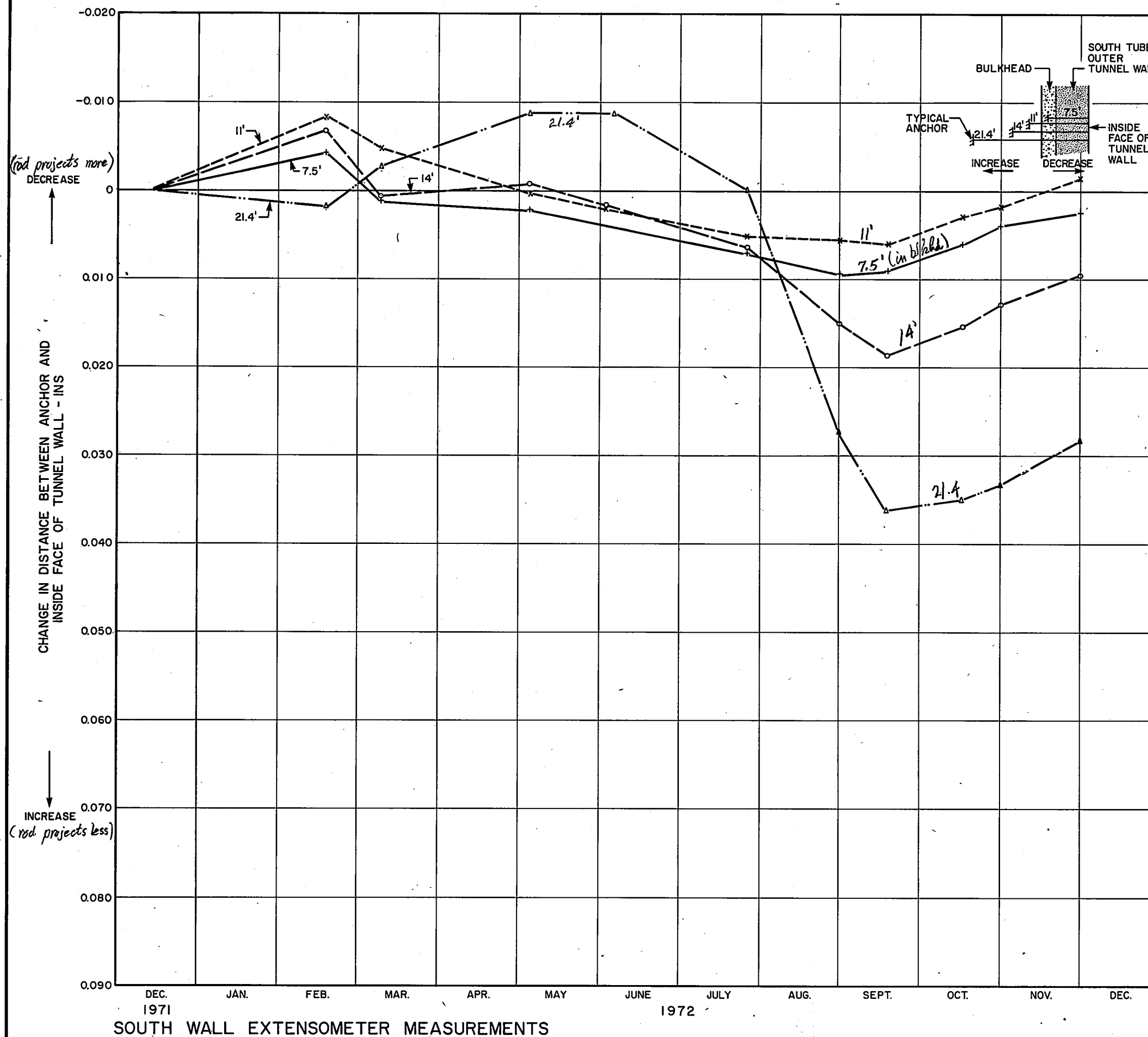
- + DETERMINED FROM STRUCTURAL CONCRETE MEASUREMENTS ON UNDER SIDE OF TUNNEL ROOF SLAB AND STRUTS (LINE A)
- x DETERMINED FROM EXTENSOMETER (FALSE ROOF)
- o SOUTH TUNNEL ROOF DEFORMATION CALCULATED USING MEASURED ROOF SLAB AND STRUT TEMPERATURES

#### NOTES

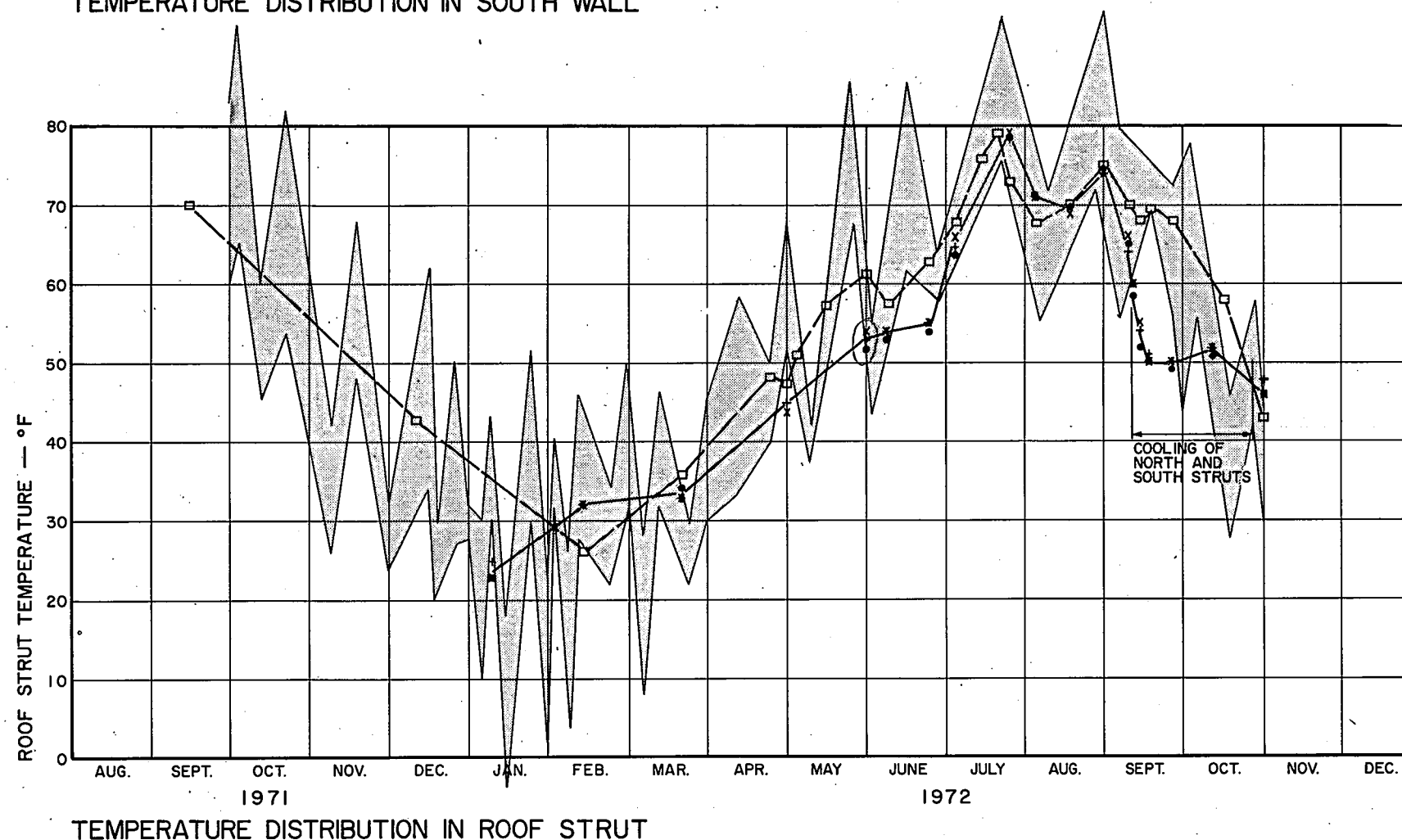
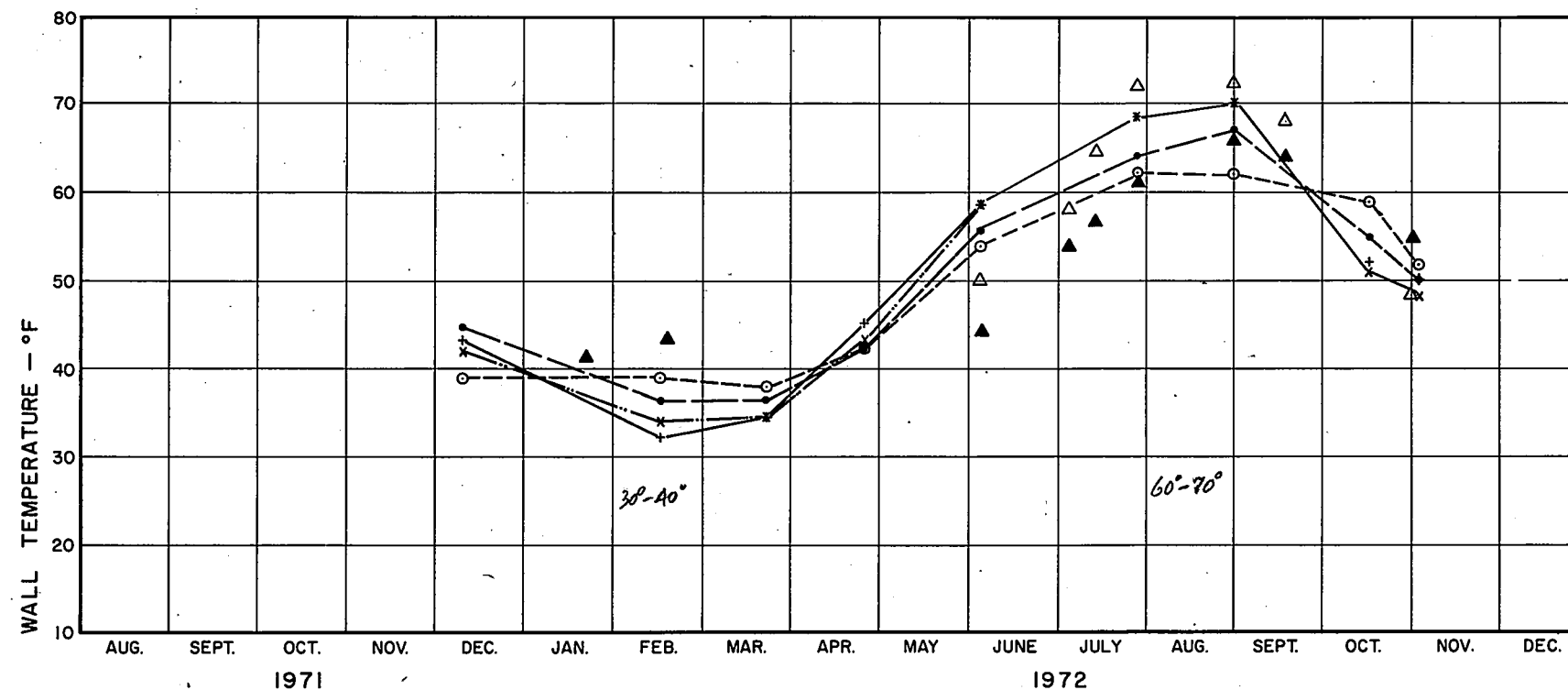
1. FOR LOCATION OF MEASUREMENTS REFER TO PLATE 1
2. TUNNEL ROOF DEFORMATION IS CALCULATED FROM STRAINS MEASURED BY EXTENSOMETERS SHOWN IN PLATE 1
3. CRACK WIDTH MEASUREMENTS PRESENTED TYPICAL OF SIMILAR OBSERVATIONS

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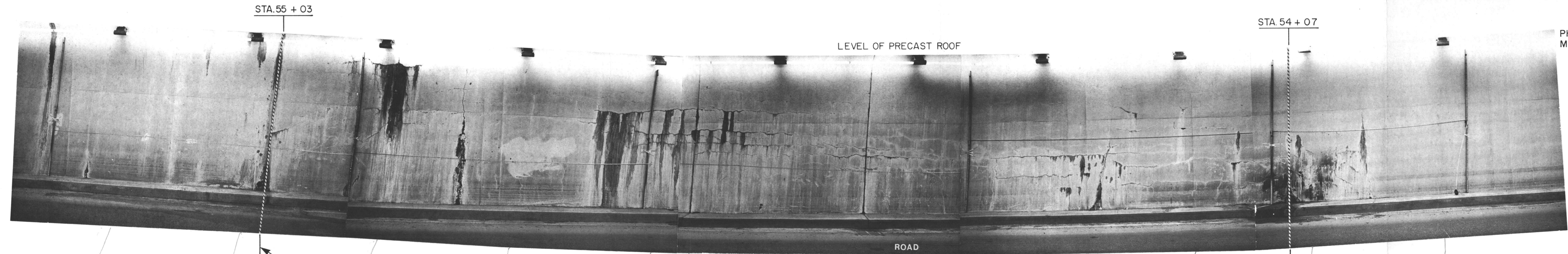
## NOTES

1. PATTERN OF CRACKING  
MAPPED NOVEMBER 1972
2. CRACKS ABOVE PRECAST CEILING  
FIRST OBSERVED FEB. 1972  
FIRST MAPPED APRIL 1972

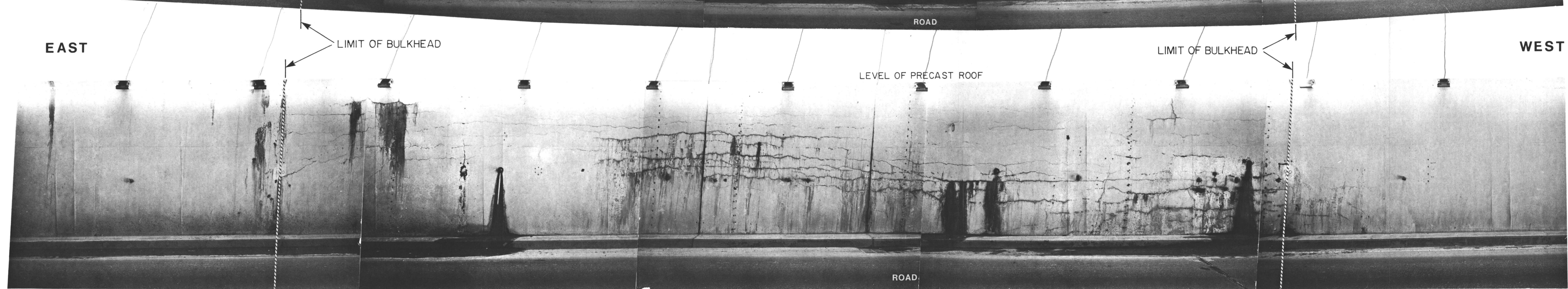
### LEGEND

- MEASUREMENT GAUGE LENGTH
- STANDARD "DEMCO" 6" GAUGE LENGTH
- 1-5
- 5-10
- 10-15
- 15-20
- 20-25
- APPROXIMATE CRACK WIDTHS  
IN THOUSANDS OF AN INCH
- SWS SOUTH WEST WALL INDIVIDUAL GAUGE LENGTH NO. 5
- SWS OR SOUTH WEST WALL ROW 3 GAUGE LENGTH NO. 2





PHOTOGRAPHED  
MAY 1971



PHOTOGRAPHED  
NOV. 1972

ACRES DEPARTMENT OF TRANSPORTATION  
AND COMMUNICATIONS  
THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

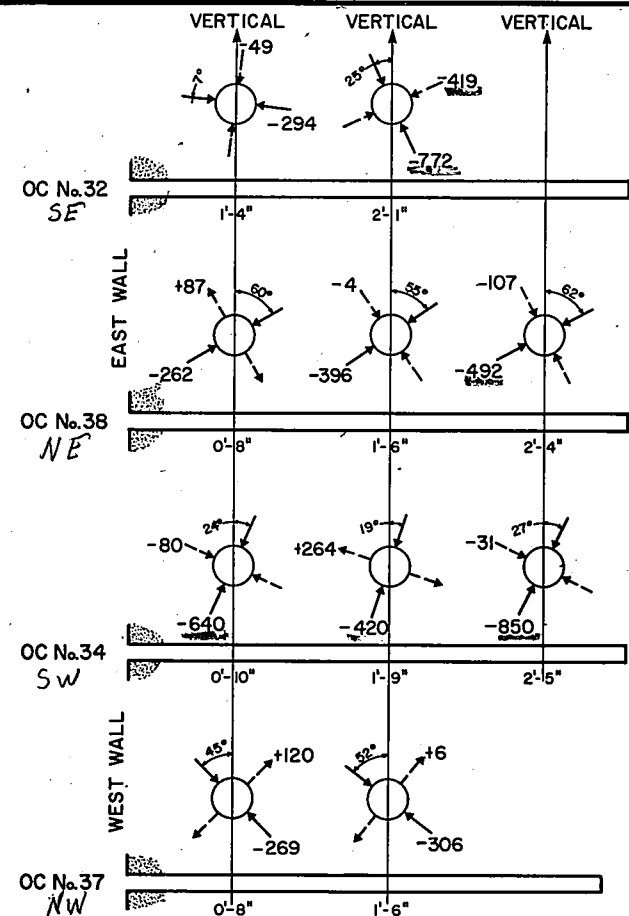
WEST SERVICE BUILDING  
VIEWS OF  
SOUTH TUNNEL WALL

ACRES CONSULTING SERVICES LIMITED

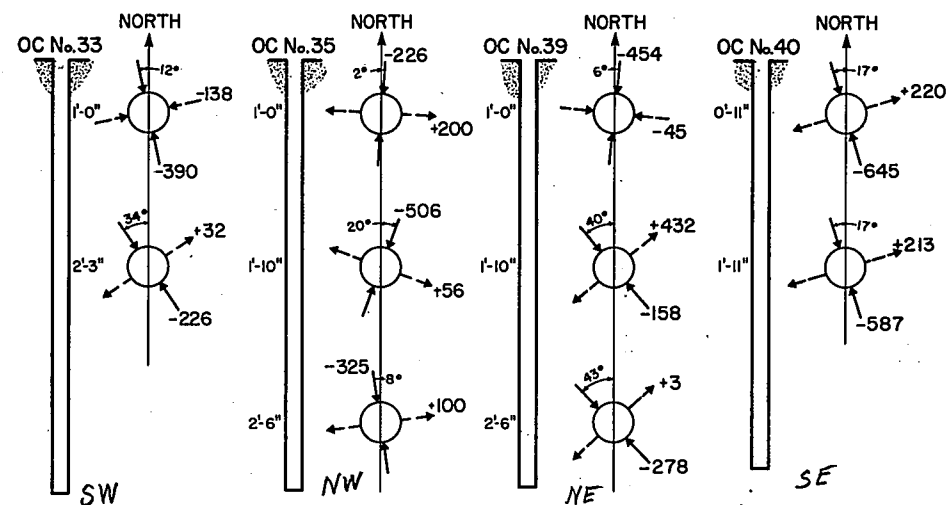
DECEMBER 1972

PLATE  
9





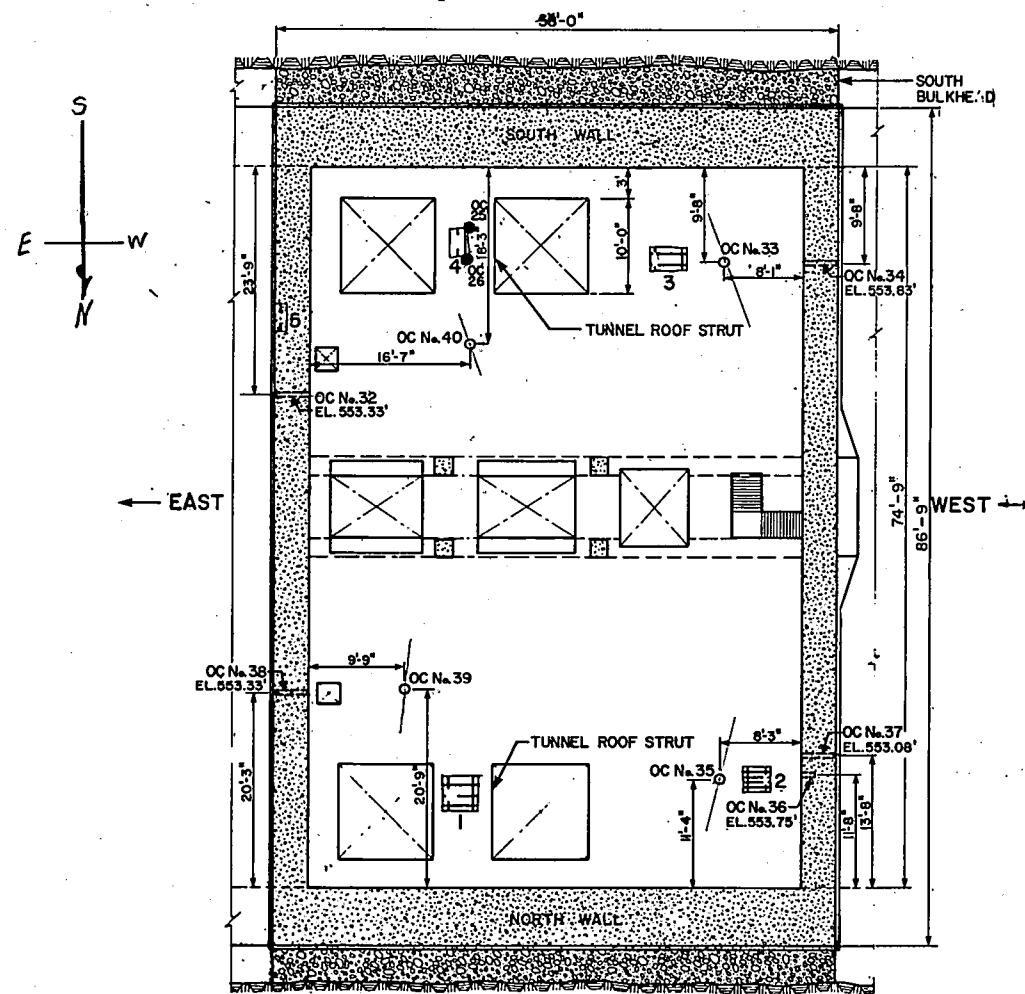
HORIZONTAL HOLES IN WALLS



VERTICAL HOLES IN FLOOR SLAB

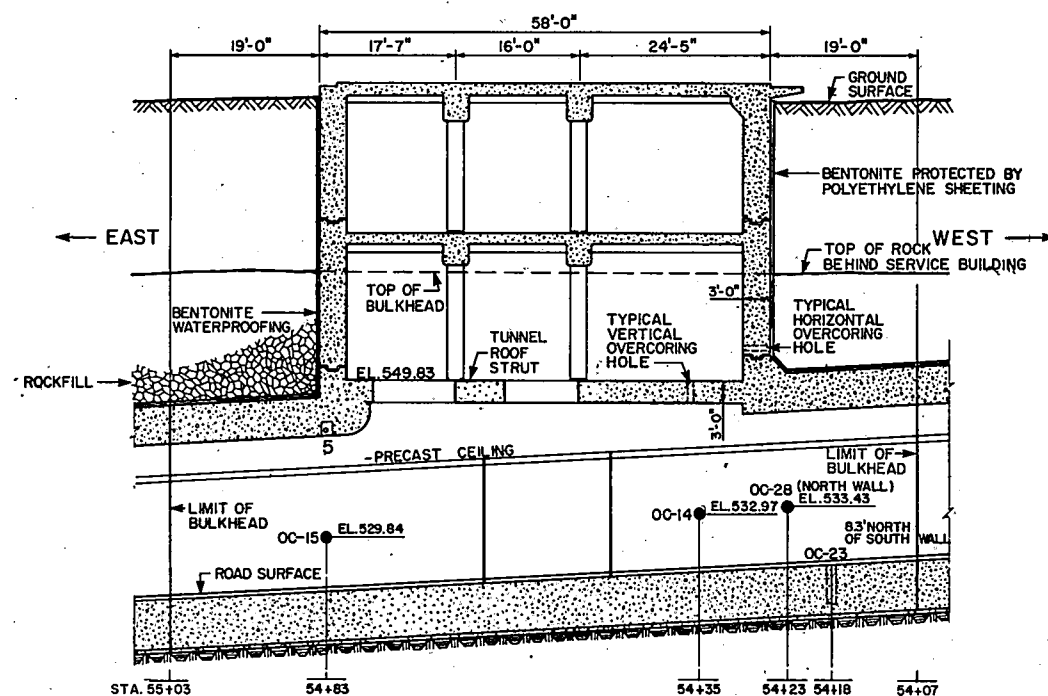
#### NOTES

1. ALL RESULTS PLOTTED LOOKING INTO HOLE FROM SURFACE OF CONCRETE
2.  $\sigma_1$  → MAJOR PRINCIPAL STRESS  
 $\sigma_2$  → MINOR PRINCIPAL STRESS
3. POSITIVE DENOTES TENSION
4. ALL STRESSES IN PSI
5. ALL READINGS TAKEN BETWEEN APRIL 6/72 AND APRIL 13/72



PLAN AT EL. 549.83

SCALE IN FEET



SOUTH TUNNEL WALL - ELEVATION

SCALE IN FEET

#### REINFORCING BAR STRESSES - PSI Apr 1972

POSITION NUMBER	NORTH-SOUTH STRESS $\epsilon$ (TEMPERATURE)	EAST-WEST STRESS $\epsilon$ (TEMPERATURE)
N. Str. 1	-20 100 (45°F)	NO BARS EXPOSED
NW 2	-10 500 (46°F)	+1350 (46°F)
SW 3	-12 060 (46°F)	-2970 (46°F)
S. Str. 4	-32 700 (48°F)	NO BARS EXPOSED
5	-14 340 (50°F)	NO BARS EXPOSED

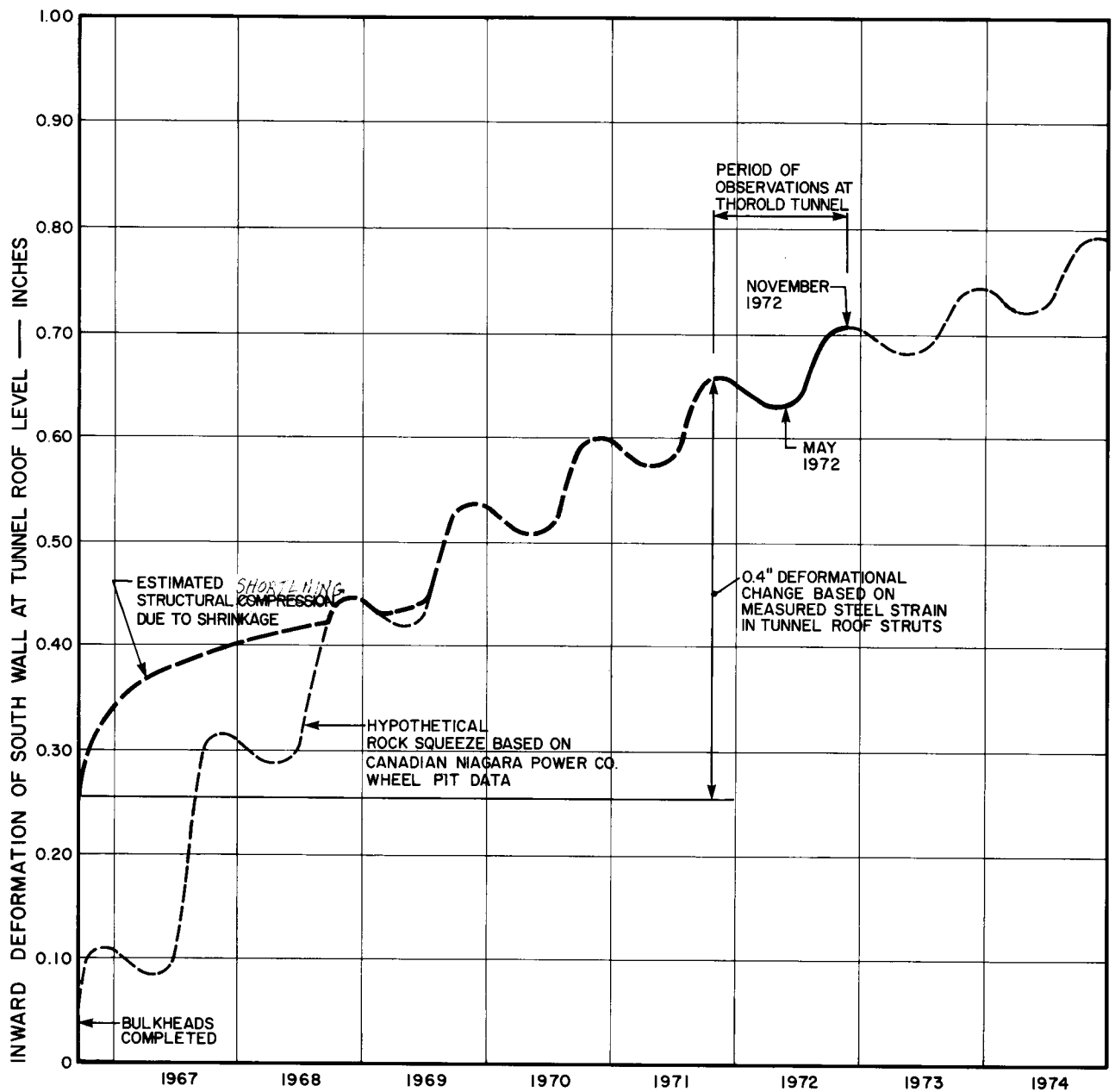
#### NOTE

A POSITIVE SIGN DENOTES TENSION IN THE STEEL  
EAST - WEST STRESSES APPROXIMATE ONLY

#### LEGEND

- OC No. 25 LOCATION OF OVERCORING STRESS MEASUREMENT OCTOBER 1971
- OC No. 39 LOCATION OF OVERCORING STRESS MEASUREMENT APRIL 1972
- 3 LOCATION OF REINFORCING BAR STRESS MEASUREMENT AND DESCRIPTION OF CUT BARS  
Measurement 2





#### NOTES

HYPOTHETICAL CURVE OF DEFORMATIONS DEVELOPED FROM MEASUREMENTS OF WHEEL PIT MOVEMENTS MADE BY ONTARIO HYDRO AND ON 1971-72 FIELD OBSERVATIONS AT THOROLD



MINISTRY OF TRANSPORTATION  
AND COMMUNICATIONS

THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING  
IMPOSED DEFORMATION  
DUE TO ROCK SQUEEZE  
HYPOTHETICAL CURVE

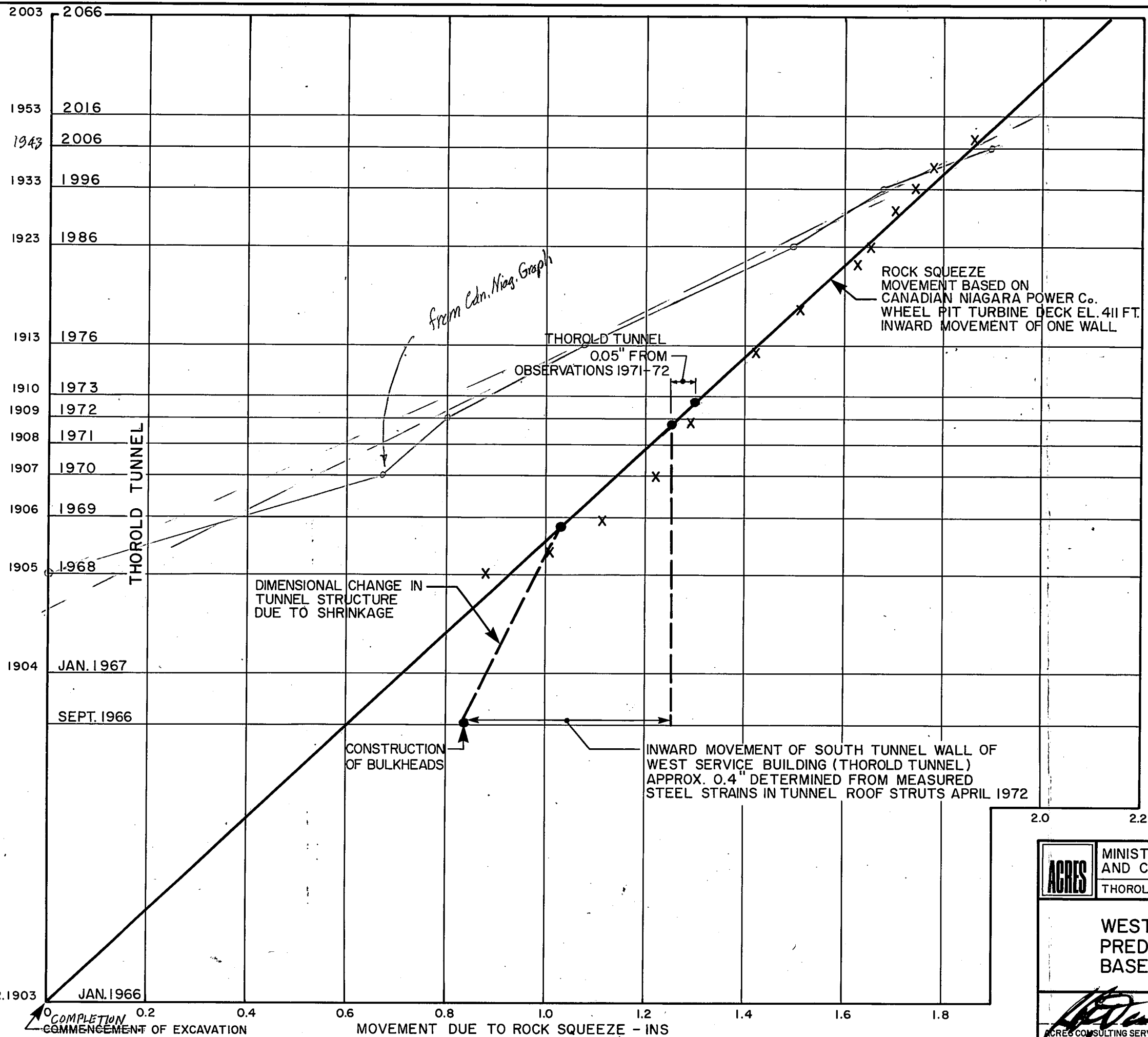
*W. T. Tanner*  
ACRES CONSULTING SERVICES LIMITED

DECEMBER 1972

PLATE  
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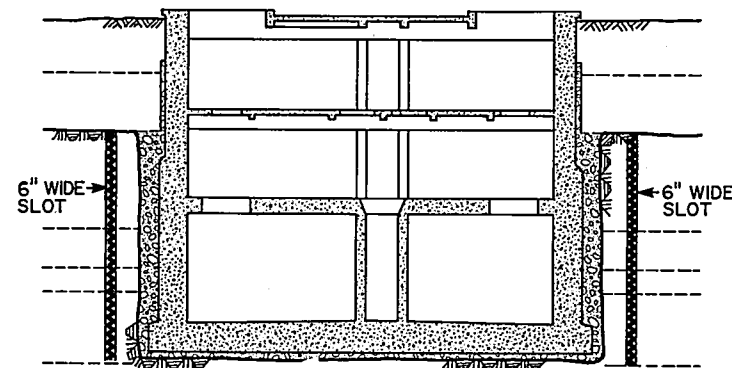
DATE OF OBSERVATIONS - YEARS  
CANADIAN NIAGARA POWER Co. WHEEL PIT



NOTES  
ANNUAL FLUCTUATIONS ARE NOT SHOWN ON THIS CURVE  
THOROLD TUNNEL OBSERVATIONS ARE SUPERIMPOSED ON CANADIAN NIAGARA POWER Co. WHEEL PIT DATA FOR COMPARISON ONLY



## (A) NO RESTRAINT OF ROCK



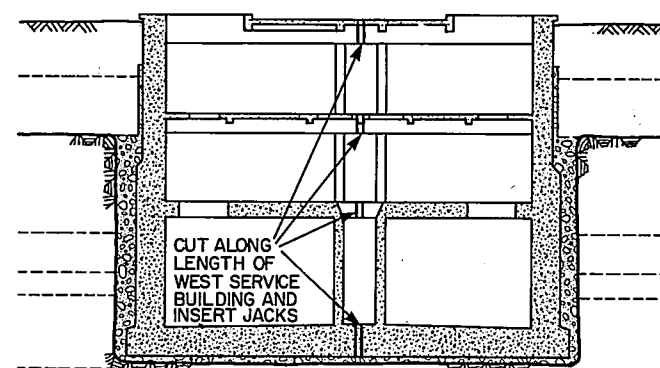
### 1. REMOVE ROCK DEFORMATION FROM STRUCTURE

CUT SLOTS IN THE ROCK BOTH SIDES OF THE STRUCTURE

WILL REMOVE MAJOR PART OF PRESENT DEFORMATIONS IMPOSED ON STRUCTURE BY ROCK SQUEEZE AND WILL ELIMINATE FUTURE IMPOSED DEFORMATIONS

THIS MEETS THE TWO OBJECTIVES OF REDUCING PRESENT HIGH STRESSED CONDITION AND ELIMINATING FUTURE HIGH STRESSES

THIS WORK CAN BE CARRIED OUT FROM GROUND SURFACE AND WILL NOT INTERFERE WITH TRAFFIC IN THE TUNNEL

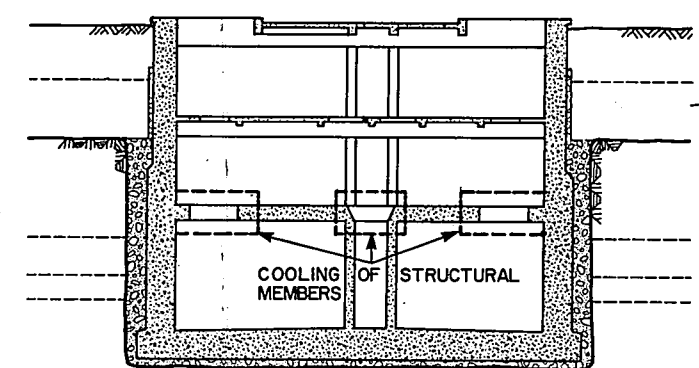


### 2. MODIFY STRUCTURE TO REDUCE INTERNAL STRAIN TO ACCEPTABLE LEVEL

CUT STRUCTURAL MEMBERS ACTING AS STRUTS BETWEEN ROCK FACES

WILL REMOVE PRESENT DEFORMATIONS ON STRUCTURE BY ROCK SQUEEZE AND COULD ELIMINATE FUTURE IMPOSED DEFORMATIONS

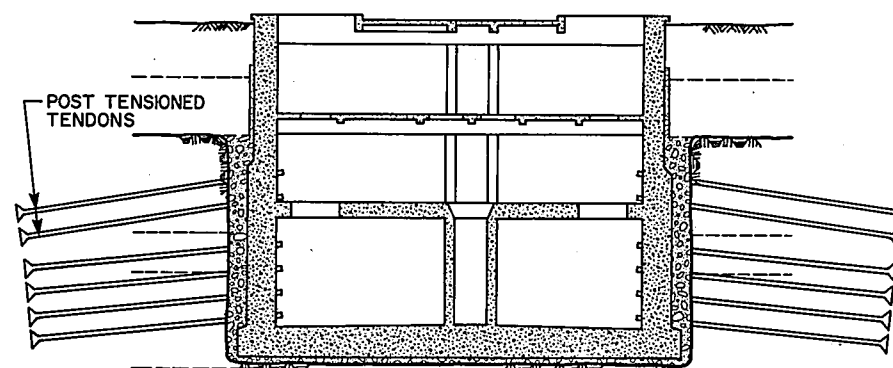
IT IS IMPRACTICABLE TO CUT THE MORE RIGID MEMBERS WITHOUT ENDANGERING THE SAFETY OF THE STRUCTURE OR ENGAGING IN MAJOR ALTERATIONS



### 3. COOLING OF STRUCTURAL MEMBERS

APPLY REFRIGERATION TO MEMBERS ACTING AS STRUTS TO REDUCE THEIR LENGTH AND PERMIT ROCK SQUEEZE TO CONTINUE

CANNOT BE REGARDED AS A PERMANENT SOLUTION AS TEMPERATURES REQUIRED ARE EXCESSIVELY LOW AND NOT PRACTICABLE TO APPLY TO ALL MEMBERS



### 1. POST-TENSIONED TENDONS IN ROCK

LARGE DIAMETER TENDONS AT CLOSE SPACING INSTALLED FROM INSIDE TUNNEL

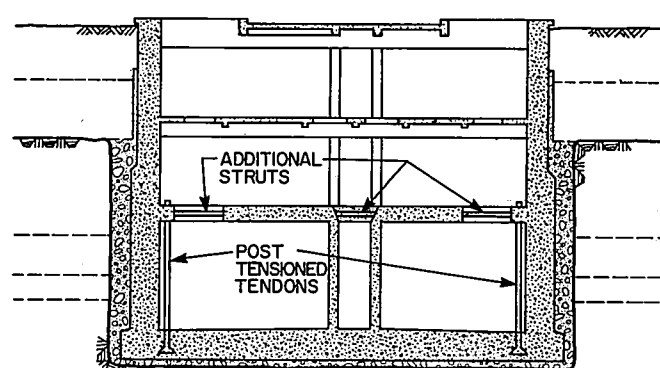
ASSUMES ORIGIN OF SQUEEZE IN ROCK IS UNDERSTOOD

INVOLVES ATTEMPT TO PRECOMPRESS THE ROCK

TO EITHER a. MAINTAIN STRUCTURE IN PRESENT CONDITION OF STRAIN, THIS MAY BE PRACTICAL BUT WILL NOT REDUCE PRESENT UNACCEPTABLY HIGH STRESS LEVELS IN THE STRUCTURE

OR b. RELIEVE STRAINS IN STRUCTURE, THIS IS NOT PRACTICABLE

ALSO INVOLVES CLOSING ONE TUBE OF TUNNEL FOR PROTRACTED PERIOD



### 2. STIFFENING THE STRUCTURE

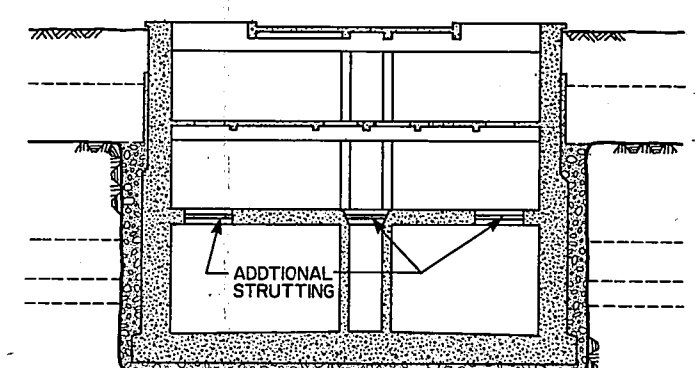
POST-TENSIONING THE WALLS AND PROVIDING ADDITIONAL STRUTTING AT TUNNEL ROOF LEVEL

ASSUMES THAT FUTURE DEFORMATIONS IMPOSED BY ROCK SQUEEZE CAN BE PREDICTED WITH SOME ACCURACY

IT IS IMPOSSIBLE TO RELIEVE PRESENT DEFORMATIONS OF STRUCTURE BY THIS MEANS

WILL NOT PREVENT FURTHER DEFORMATION DUE TO ROCK SQUEEZE

REQUIRES CLOSING ONE LANE AT LEAST TO IMPLEMENT AND WILL REDUCE TUNNEL CLEARANCES



### 3. STRUTTING ACROSS STRUCTURE

PROVIDING ADDITIONAL STRUTTING IN VENTILATION OPENINGS AT TUNNEL ROOF LEVEL

ASSUMES THAT FUTURE DEFORMATIONS IMPOSED BY ROCK SQUEEZE CAN BE PREDICTED WITH SOME ACCURACY

IT IS IMPOSSIBLE TO RELIEVE PRESENT STRESSES IN STRUCTURE BY THIS MEANS

WILL NOT PREVENT FURTHER DEFORMATION DUE TO ROCK SQUEEZE

WALLS WOULD CONTINUE TO DEFORM LEADING EVENTUALLY TO FAILURE

CANNOT BE REGARDED AS PERMANENT SOLUTION AS IT DOES NOT RELIEVE IMPOSED DEFORMATION FROM TUNNEL WALLS

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MINISTRY OF TRANSPORTATION  
AND COMMUNICATIONS

THOROLD TUNNEL STRUCTURAL INVESTIGATIONS

WEST SERVICE BUILDING  
POSSIBLE REMEDIAL  
MEASURES

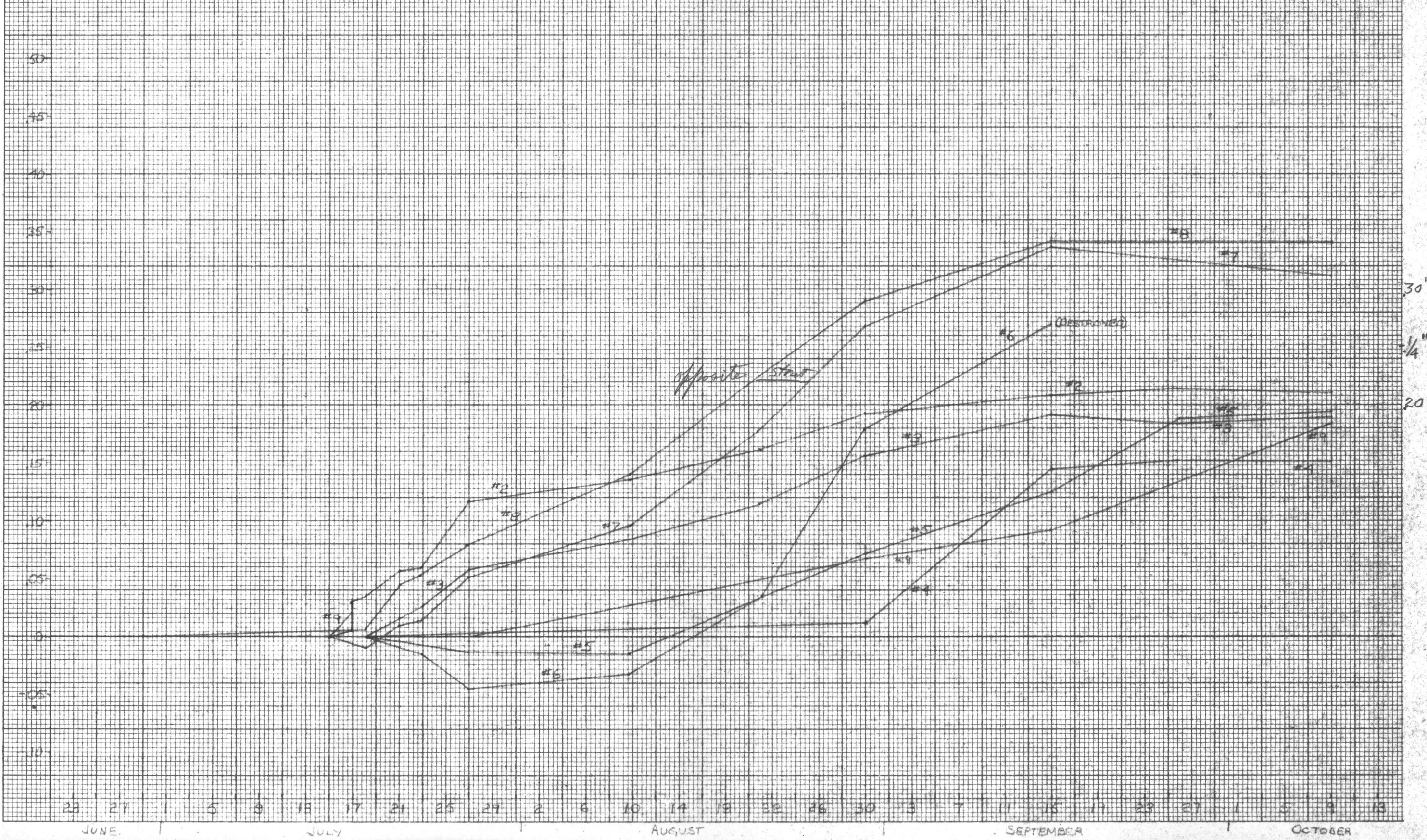
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DECEMBER 1972

PLATE  
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# Top of SLOT DEMEC STUDS



MICROGRAPH  
CCE2

MADE IN CANADA  
50 X 50 to the inch  
G-11L



# WALL EXTENSOMETERS 3<sup>RD</sup> FLOOR WEST SERVICE BLDG.

LEGEND

BH 18@25'	•	}	
18@50'	⊙		
19@10'	x	}	West
19@16'	+		
20@10'	⊕	}	
20@16'	Δ		

slab  
BH 18@10'  
BH 18@25'  
BH 18@50'

Strut  
WEST 3/2 DB  
BH 20@10'

near side of bay

BH 20@10'  
BH 19@10'



MADE IN CANADA  
50 X 50 10 per inch  
G-11L

