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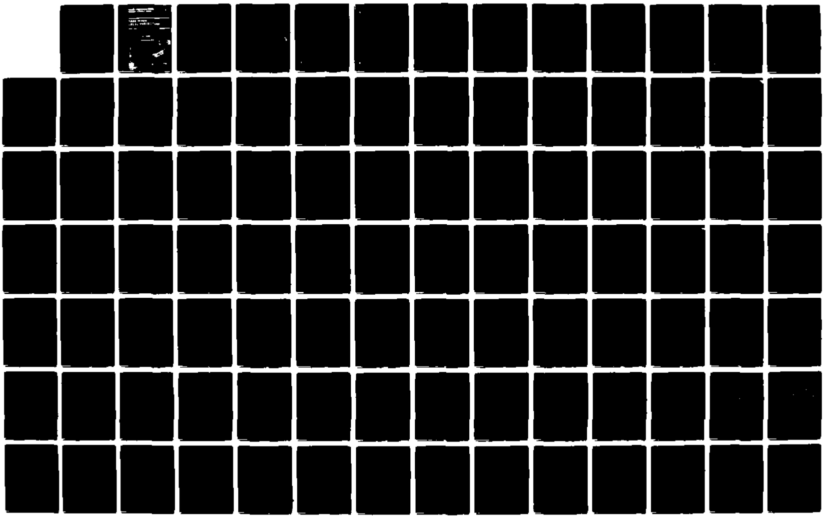

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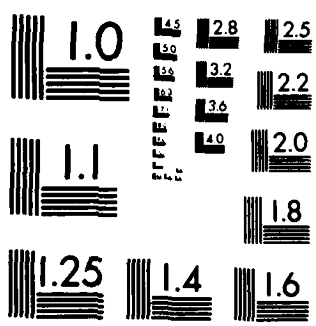

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VOLUME II
APPENDICES

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PARK RIVER AUXILIARY TUNNEL
AS-BUILT FOUNDATION REPORT

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report presents a record of the instrumentation of the auxiliary conduit design, and construction. The data recorded includes piezometer records, observation wells, MPBX precast strain gages, vibration monitoring, photographic record and a post-construction summary and evaluation of the instrumentation program.

PARK RIVER LOCAL PROTECTION PROJECT

AUXILIARY CONDUIT TUNNEL

CONSTRUCTION FOUNDATION REPORT

VOLUME II

APPENDICES

A. Geotechnical In-Situ Stress Measurements and Photographic Geologic Documentation, Park River Auxiliary Conduit, Hartford, CT

B. Subsidence Survey Report

APPENDIX A

GEOTECHNICAL IN-SITU STRESS MEASUREMENTS AND PHOTOGRAPHIC GEOLOGIC DOCUMENTATION, PARK RIVER AUXILIARY CONDUIT, HARTFORD, CONNECTICUT

FINAL REPORT

GEOTECHNICAL INSTRUMENTATION,
IN-SITU STRESS MEASUREMENTS AND
PHOTOGRAPHIC GEOLOGIC DOCUMENTATION
PARK RIVER AUXILIARY CONDUIT
HARTFORD, CONNECTICUT

Submitted to

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1. INTRODUCTION

1.1 Purpose

The purpose of this project, which included tunnel instrumentation, stress relief overcoring and continuous photographic recording of the geology at the springline of the tunnel, was to obtain data for design verification for the Park River Auxiliary Conduit and future design applications in other tunnels and to monitor the performance of the tunnel during construction so that the safety of all personnel could be evaluated by the Corps of Engineers (COE) as work progressed.

1.2 Scope

The scope of work performed by Geotechnical Engineers Inc. included the following:

a. Preparation of a detailed proposal, including instrument locations, hardware, monitoring schedule, reporting format, geologic camera recording system and a rock stress measuring program.

b. Numerous meetings and correspondence with Roger J. Au & Son, Inc. and the Corps of Engineers to finalize the instrumentation and geologic documentation program.

c. Installation of six Casagrande-type piezometers and two PVC observation wells in the surficial materials above the tunnel.

d. Monitoring of the 6 piezometers, the 2 observation wells and 13 observation wells previously installed by the COE for 35 months on a monthly basis. (Note: some of these wells were monitored more frequently during a 5½-month period, but these data have not been made part of this report.)

e. Installation and maintenance of 12 rock bolt load cells in the drill and blast section of the tunnel and monitoring of these load cells for approximately one month during the advance of this section of the tunnel.

f. Installation and maintenance of 10 six-position multiple position borehole extensometers (MPBX's) from the ground surface above the centerline of the tunnel. The installation of the MPBX's included a check of the verticality of each borehole. The MPBX's were monitored for 2 to 20 months during construction. (The duration of monitoring was governed by their position along the tunnel route.)

g. Installation and maintenance of a total of 36 embedded and 28 surface-mounted vibrating wire strain gages on 9 test rings and monitoring of these gages for 1 to 9 months during construction (depending on their position along the tunnel route).

h. Performance of 12 unconfined concrete modulus tests on concrete cylinders obtained during various liner pours.

i. Installation and maintenance of a total of nine vibrating wire piezometers through the tunnel lining at the nine test ring locations and monitoring of these piezometers for two to eight months during construction.

j. Installation and maintenance of a total of 32 tape extensometer monitoring points at various locations along the tunnel and the monitoring of the convergence and divergence of these points for 2 to 9 months during construction.

k. Technical coordination of the drilling of two orthogonal boreholes in the tunnel wall at the drill and blast section, during which in-situ rock stress measurement was performed by means of the overcoring technique.

l. Design, installation, operation and interpretation of an automated camera system mounted in the tunnel boring machine (TBM) which was used to continuously record the bedrock encountered at the tunnel springline through a 2 ft x 3 ft opening in the shield. This work included the mapping and strip photographic recording of the geology encountered in a portion of the drill and blast tunnel which served as a calibration during the interpretation of the production strip photography.

m. Preparation of an Interim Report at the completion of the drill and blast section of the tunnel to provide an update of the instrumentation program and to solicit comments relative to the anticipated final report format.

n. Preparation and transmittal of 36 weekly reports during the tunnel advance, including the instrument data collected and a summary of the respective instrument performance since the previous report.

o. Alerting the proper personnel both in these reports and verbally whenever abnormal data trends were noted.

p. Preparation of this final report.

1.3 Authorization

This work was performed as required in specification sections 13A and 13B and as modified by GEI submittals on The Geologic Mapping and Instrumentation Program dated December 23, 1977; April 12, 1978; May 4, 1978; and August 21, 1978. This work was performed under Roger J. Au & Son, Inc. Contract DACW33-77-C-0099, dated December 12, 1977 and signed by Mr. Roger C. Au.

2. EXECUTIVE SUMMARY

The following is a brief summary of the observations and conclusions which are presented in this final report.

2.1 Instrumentation Program

Rock Movements

- The rock movements measured above the tunnel crown at the 10 test sections were relatively small, generally ranging between 0.03 in. to 0.26 in. within 2.0 to 3.7 ft of the crown. The largest movement recorded was 0.62 in. The larger rock movements were observed to occur in fault zones.

- Most of the measured rock movement occurred as the tunnel heading advanced through a zone 50 ft past the measuring instrument, with practically all of the measured movement taking place after the heading had progressed 100 ft past the respective instruments.

- The rock movement decreased upward from the crown of the tunnel.

Structural Liner Response

- Relatively small strains were generally measured in the nine instrumented liner rings during the tunnel advance. Eighty-four percent of the strain gages indicated strains less than $200\mu\varepsilon$.

- The strain and resulting load distribution in the instrumented liner rings were not uniform, with the largest strains (some exceeding $500\mu\varepsilon$) generally measured in the crown segment.

- There was a general trend in all of the instrumented rings of increasing compressive strain or decreasing tensile strain with time. This indicates that a time-dependent load redistribution was occurring in the rings.

Groundwater Response

- There were two distinct hydrologic regions through which the tunnel was constructed: the eastern end of the project appeared to be significantly influenced by the Connecticut River while the region to the west was not.

- Piezometric drawdowns of 100 to 130 ft were measured in the bedrock at the tunnel elevation during construction. Piezometric drawdowns of 20 to 25 ft were measured in the overlying glacial till during the advance of the tunnel.

- Drawdown of the piezometric levels in the bedrock and glacial till generally began 300 to 500 ft ahead of the tunnel heading.

Instrument Performance

- Whenever possible, instrumentation should be installed well in advance of the construction to be monitored.

- Vibrating wire measurement systems performed very well in the adverse tunnel environment, while electrical resistance systems were less satisfactory.

- Tunnel chord and diameter measurements were very difficult, if not impossible, to obtain in a timely manner on this project due to interference with the tunnel excavation equipment.

2.2 Overcoring Stress Measurements

- The rock stresses measured during the overcoring program from within the drill and blast tunnel were lower than those obtained during a prior program conducted from the ground surface. This may be due in part to: measurements made less than one diameter from the tunnel, the effects of different jointing systems, or the effects of blasting.

2.3 Photographic Geologic Documentation

- Automated photographic documentation is better suited in identifying structural characteristics of bedrock rather than identifying lithological units.

- Apparent dip angles of joints, joint sets, fractures and faults can be estimated.

- Any protective device (door) should be designed to, or be constructed of materials which would permit photographs to be taken even when in place.

- Stationing systems should be designed into shutter actuating mechanics.

- The use of photographic documentation to record geologic features encountered during advancement of machine-bored tunnels is feasible but requires additional research and modification of existing systems.

3. PROJECT DESCRIPTION

3.1 General

The Park River Auxiliary Conduit Project consists of the construction of a 22-ft I.D. inverted siphon-shaped rock tunnel and adjacent shafts below the City of Hartford, CT (see Fig. 1). This tunnel is designed to take the overflow flood waters from the Park River and divert them to the Connecticut River. The tunnel is approximately 9,100 ft long and passes 160 to 200 ft below the city.

3.2 Construction Operation

The tunnel was advanced upgrade from the outlet shaft adjacent to the Connecticut River (see Fig. 2). Upon completion of the outlet shaft, approximately the first 235 ft of the tunnel was advanced using drill and blast excavation techniques to form a U-shaped chamber approximately 26 ft x 26 ft. A 120-ft-long tail tunnel 9-ft-wide by 13-ft-high was also excavated by drill and blast methods. The roof of the tunnel in the drill and blast sections of the project were supported with 10-ft-long fully encapsulated resin rock bolts installed on approximately 4 to 5 ft centers.

After completion of the drill and blast sections, the tunnel boring machine (TBM) was assembled in the excavated chamber and the tunnel advance using the TBM was begun. The TBM was a fully shielded, rotary hard-rock machine manufactured by the Robbins Company, Seattle, Washington, which cut a 24 ft, 3 in. diameter bore. The temporary support and final lining are provided by four-segment precast concrete liner rings which were erected in the tail shield of the TBM approximately 35 to 40 ft behind the cutter face. Each of the four segments is 9-in.-thick and about 6-ft-wide and a completed ring provides a finished inside diameter of 22 ft. The annular space between the excavated rock surface and outside surface of the liner was filled with 1/2-in.-diameter peastone generally two to three rings behind the last ring installed. Cement grouting of the peastone was generally accomplished from an independent grout gantry about 200 to 500 ft behind the last installed ring.

All excavated tunnel muck was transported from the tunnel heading to the outlet shaft by train where it was removed by crane. All infiltrated water was collected at a sump at the outlet shaft and was pumped from the tunnel.

3.3 Geology

The City of Hartford, including the project site, is located in the Connecticut Valley which is a broad lowland basin underlain by Triassic age rock generally consisting of shale, sandstone and conglomerates and including sills and dikes of basalt. The beds generally dip to the east at 15^{o} to 20^{o}. Numerous fault blocks are found in the region mainly in and around the igneous rock bodies. The bedrock in the region has been covered by glacial till and later glacial lake deposits of stratified sands and varved clays and silts.

The tunnel was constructed on an east-west alignment, essentially normal to the strike of the dipping sedimentary beds. The dominant rock is a reddish brown shale and/or siltstone which is thin bedded, calcareous, moderately hard, and usually fresh or moderately weathered. Gray-black shale was also encountered which was similar in physical properties to the red shale. The bedding strikes approximately north-south and generally dips 10^{o} to 20^{o} to the east. The primary jointing occurs along these beds. Frequently, beds of hard, red to whitish sandstone occur within the shales. The thickness of the sandstone layers ranges from less than 1 in. to greater than 4 ft. Basalt dikes intrude these sedimentary rocks at various locations. The basalt is hard and contains frequent hairline fractures.

Minor faulting was encountered along most of the tunnel route. However, a major fault zone was encountered between Sta 94+50 and 96+50 consisting of anomalously high bedding angles, severe brecciation and fracturing and an approximately 65-ft-wide zone of a plastic mixture of sand, silt and clay with some rock fragments and no apparent rock structure.

Overlying the bedrock for the entire tunnel route is a dense glacial till varying in thickness from 5 to 35 ft. Glacial lake deposits of varved clay and silt are found over the glacial till deposits especially at the western and eastern ends of the project where the thicknesses at each shaft are 40 to 45 ft. These deposits are lightly to moderately overconsolidated and compressible. Overlying the glacial lake deposits and/or glacial till are areas of miscellaneous fill and granular floodplain deposits generally consisting of fine sand. The maximum thickness of the floodplain deposits is approximately 40 ft near the outlet shaft.

Groundwater levels measured prior to construction of the tunnel indicate that the piezometric level in the bedrock is normally 155 to 190 ft above the invert of the tunnel.

Refer to Fig. 2 for a simplified profile of the geology along the tunnel route which is based on the exploratory borings performed by the Corps of Engineers during the design of the project.

4. INSTRUMENTATION PROGRAM

4.1 General

The tunnel instrumentation program was designed to provide the tunnel designers and the contractor with performance information relative to the ground-structure interaction of the tunnel in order to evaluate design assumptions and safety of the construction and to provide meaningful data for future designers. With these purposes in mind, a monitoring program was mutually developed by GEI, COE and Roger J. Au & Son to measure rock movement and water pressure information at selected locations along the tunnel route together with structural liner performance data of strain and diametrical deformations at the same locations. In all, ten test sections were chosen, one in the drill and blast section and nine in the TBM driven, precast-concrete-lined section.

Test Section #1 in the drill and blast portion of the tunnel consisted of a six-anchor MPBX installed over the crown of the tunnel from the ground surface and twelve rock bolt load cells installed on selected roof bolts in the immediate vicinity of the MPBX. The remaining nine test sections in the TBM portion of the project each consisted of a six-anchor MPBX installed over the crown of the tunnel from the ground surface. The four-segment precast-concrete-liner ring directly below each MPBX was instrumented with four embedded vibrating wire strain gages (one in each segment) and three surface-mounted vibrating wire strain gages (on the crown and two side segments). In addition, tape extensometer reference points were also located on each strain gaged ring to measure possible length changes in various chord distances for the ring. A vibrating wire piezometer was also installed into the adjacent bedrock through the precast concrete liner to monitor the hydrostatic pressure on each test ring. Refer to Table 1 for a summary of the instrumentation test sections for this project and Fig. 2 for the location of these test sections.

4.2 Selection of Hardware

It has generally been observed in other rock tunnel projects that a major portion of the rock movement over the crown of the tunnel occurs during the time the tunnel heading is advancing to about two tunnel diameters past the measurement point. Since the TBM used for this project was about 40 ft long and fully shielded, it was not possible to gain access to the tunnel crown until the heading had progressed at least two tunnel diameters past the intended measuring point. In addition, the installation of an MPBX from within the tunnel would have delayed the TBM progress during the instrument installation. Therefore, it was decided that the MPBX's would be installed from the ground surface in advance of the tunnel heading.

In consideration of the adverse environment existing in a tunnel for delicate electronic equipment, vibrating wire strain gages were selected due to their comparatively robust nature, relative ease of installation and repair, and good performance in dirty, wet environments. Vibrating wire piezometers were also selected at each test ring to provide a system compatible with the strain gages and allow for ease of monitoring as only one type of readout device was required.

4.3 Selection of Test Section Locations

The location of the instrumented test sections was based on one or more of the following considerations: to provide a comparison between different tunneling methods in similar geology, to provide a comparison of the tunnel performance in different geologic structures and lithology, to monitor the rock movements and structural performance of the tunnel in areas of key buildings located over the tunnel at the ground surface, and to provide access at the ground surface for the MPBX's.

The following table briefly provides the rationale for locating the ten test sections at each specific location:

Test Section	Station Location	Rationale
1	8+20	Drill/Blast - to compare behavior of similar rock when excavated by drill/blast vs TBM
2	11+32	TBM - to compare behavior of similar rock when excavated by drill/blast vs TBM
3	15+25	Tunnel partly in sandstone, partly in shale
4	24+00	Church of the Good Shepard nearby
5	27+00	Church of the Good Shepard nearby
6	43+50	St. Peter's Church nearby
7	58+55	Dip Reversal - Fault Zone
8	61+19	Interbedded shale/sandstone. Dip 20°
9	91+00	Transition Zone - Possible Fault
10	95+54	Interbedded shale/sandstone. Dip 55°. Fault Zone

4.4 Installation and Data Collection Procedures

4.4.1 Multiple Position Borehole Extensometers (MPBX)

The multiple point borehole extensometers used for this project were Model E10G untensioned rod extensometers manufactured by IRAD/GAGE, Lebanon, NH. Each of the six 1/8-in.-diameter stainless steel rods is jacketed in a lubricant-filled, 1/4-in.-diameter, Schedule 40 PVC hollow tube. An 8-in.-long, #6 reinforcing bar anchor is attached at the lowermost end of each rod. The installation included installing NW flush-joint casing through the overburden to bedrock, coring an NX hole to the desired depth in bedrock, installing the instrument hardware down the borehole, tremie-grouting the borehole to the ground surface and attaching the instrument reference head to the NW casing which was left in place. Odd-numbered Figs. 3 through 21 show the locations of the six grouted anchors for each instrument. Prior to installing the instrument hardware down the borehole, a verticality survey of each boring was made using inclinometer techniques. The vertical alignment of the borehole for each MPBX is shown on even-numbered Figs. 4 through 22.

Photograph 1 shows the assembled MPBX and grout tube prior to bundling of the rods and installation. Photograph 2 is of the reference head in the protective manhole prior to the installation of the brass reference plate. Note the six anchor rod reference tips.

Measurements of the depth to the top of each stainless steel rod relative to the brass reference plate in the instrument head were made using a digital depth micrometer, with a resolution of 0.001 inch. Nine of the MPBX's were monitored on a daily basis when the tunnel heading was in a position of 100 ft before to 150 ft past the instrument location. Readings were then decreased to twice weekly or weekly for approximately the next four weeks, after which they were decreased to about monthly until the end of the tunnel advance. The brass reference plate was optically surveyed periodically using standard surveying equipment, with a resolution of 0.01 ft and estimates made to the nearest 0.001 ft.

The installation of MPBX-8, i.e., grouting of the anchors, was completed on March 25, 1980 after the tunnel heading had progressed approximately 75 ft past the instrument location at Sta 61+19. This installation was delayed because the original location had to be moved closer to Park Street due to inaccessibility of the drill rig. Therefore, it was not possible to obtain initial readings of the anchor positions prior to the tunnel heading approaching to within 100 ft of the instrument, nor was it possible to monitor the rock movement above the tunnel as the heading passed beneath the MPBX.

4.4.2 Rock Bolt Load Cells

The rock bolt load cells used during this program were Model PC-60, 60-ton capacity, full bridge, hollow center cells, manufactured by Terrametrics, Golden, CO. The cells were supplied with 30 ft of armored cable and were read using the Terrametrics P-350A readout which is a modified version of the Vishay P-350 null indicator strain gage readout box. The cells were conditioned to the tunnel environment for at least five hours prior to obtaining a no-load reading and installation on the rock bolts. The instrumented rock bolts were 10-ft-long steel reinforcing rods, which were installed with a resin grouted length of approximately 2 ft, allowing for about 8 ft of free anchor length. Steel bearing plates were placed on both sides of the load cells and wedges were used between the plate and the rock to make the load applied to the cells as uniform as possible. The lock nut for each instrumented bolt was snugged up to the lock-off load to complete the installation. See Photographs 3 and 4 for examples of typical load cell installations. Figs. B-1 through B-6 in Appendix B show the approximate load cell locations relative to the tunnel cross section.

The load cells were installed on the rock bolts approximately 20 to 30 ft behind the heading. This installation schedule was mutually agreed on by GEI, Roger J. Au & Son and the U. S. Army Corps of Engineers to limit the damage that might be experienced by the load cells during blasting.

The load cells were monitored on an approximate daily basis when the tunnel heading was within 100 ft of each cell. The readings were then decreased to approximately three times a week until the cells were removed. The temperature in the tunnel at the location of the load cell installation was also recorded for every reading of the cells.

4.4.3 Embedded Strain Gages - Casting Yard

The embedded strain gages used were the Model EM-5 vibrating wire type manufactured by IRAD/GAGE Inc., Lebanon, NH. There are three basic components of the gage: (1) vibrating wire, (2) cover tube and flanges, and (3) plucking coil and readout cable. The wire is clamped to the flanges at each end of the cover tube. The flanges are free to move along the axis of the wire and provide a means of fastening the gage to the rebar. The plucking coil and readout cable are fixed to the cover tube by the manufacturer and provide remote readout capability.

The embedded strain gages were installed in the precast liner segments at the San-Vel Co. casting yard in Littleton, MA. The gages were suspended on the rebar cage using 24-gage steel wire (see Photo 5). The steel wire was arranged such that the gage would not be allowed to move during casting. The gage cover tube was lightly greased prior to casting of the segment to prevent concrete from adhering to it. The flanges were not greased so a bond could develop between concrete and vibrating wire. The readout cable was attached to the rebar using cable clamps and excess cable was temporarily placed in a steel handy box which was attached to the rebar and cast into the segment (see Photo 5). Readings were obtained on all gages prior to sealing the handy box.

Embedded gages were generally located as close to the center of segment as possible, midway between the inner and outer circumferential rebar (see Table 2). The gages were situated to measure strains approaching the free field concrete condition between the rebar. Any inconsistencies in the circumferential locus of the gages was largely due to small variations in the spacing of the longitudinal rebars.

Casting of the instrumented segments was done on two separate days. Following the placement of the rebar cage in the steel forms, the steel wire suspending the gage had to be tightened due to yielding of the wire during handling of the semi-stable rebar cage. Once the gages were again firmly fixed to the rebar, kiel marks were made on the inside of the forms indicating the location of gage and handy box. When the segments were removed from the forms after casting, the kiel marks were visible on the concrete surface and the excess concrete was chipped away from the handy box covers. After opening the handy boxes following the first cast, it was recognized that a small portion of concrete had intruded within the box, making wire removal difficult. Prior to the second cast, the handy boxes were more thoroughly sealed which prevented concrete intrusion.

Instrumented segments were stored at the San-Vel Storage Yard and periodic strain measurements were made during curing. Locations of the embedded gages were also painted on the surface of the segment providing a more durable reference than the kiel mark.

Concrete test cylinders with a length and diameter of 8 and 4 inches, respectively, were made from the concrete batches used for the instrumented ring pours. These cylinders were cured under environmental conditions similar to those experienced by the liner segments. These cylinders were later tested by the H. G. Protze Co., Newton Highlands, MA to determine representative concrete moduli for the various segments. This testing is discussed in more detail in Section 4.5.4 of this report.

The final wiring of the embedded gages to the terminal boxes and the monitoring schedule which was followed is discussed in Section 4.4.4.

4.4.4 Surface-Mounted Strain Gages

The surface-mounted strain gages used were the Model SM-5A vibrating wire type manufactured by IRAD/GAGE, Inc., Lebanon, NH. There are three basic components to the gage: the vibrating wire, plucking coil, and two weldable end blocks for mounting. The strain gage consists of a wire clamped at both ends protected by 5/16-in. stainless steel cover tube with one end free to move along the axis of the wire. The plucking coil which excites the wire and measures its natural period can be wired to a terminal box for remote readout.

The end blocks were welded onto 6-in.-long #6 rebars. To ensure proper spacing and orientation of the end blocks during installation, an aluminum alignment bar was inserted into the end blocks and secured by the set screws forming the mount assembly. Two 1-in.-diameter holes were drilled 6-in.-deep into the precast concrete lining at each gage location. The holes were wetted and partially filled with a mortar cement mix with Sika #2 added as a hardening agent. The mount assembly was then inserted into the holes so that the centerline of the alignment bar was approximately 1/2 in. from the inside surface of the precast lining. Enough mortar was placed in each hole such that at full insertion a small portion of mortar would be extruded out of the hole to ensure full contact around the rebar.

The cement mortar was allowed to cure for approximately 24 hours to ensure hardening of the mortar prior to removing the aluminum alignment bar. The strain gage was then inserted into the mounts and secured by the set screw at one end. A plucking coil was attached to the gage and a spring loaded tensioning device was used to set the initial reading at midrange (approximately 6800 display units). When the display was at midrange, the set screw at the opposite end of the gage was tightened and the display recorded. See Photo 6 for an example of a typical surface strain gage installation during adjustment.

Surface-mounted strain gages were generally located at the tunnel springline and tunnel crown adjacent to the embedded gages (see Table 2 and Fig. T-1). Surface gages were located at the same locus circumferentially but approximately one foot ahead of the embedded gages to avoid damaging the embedded gage during installation. Any inconsistencies in the circumferential location of the gages was due to a variation in the longitudinal rebar spacing and, on occasion, the encountering of a rebar while drilling holes for the mount assembly.

Following the advance of the TBM trailing gear past the instrumented ring, the wire leads from the three surface gages and four embedded gages for each ring were routed to a common terminal box. The leads were attached to the surface of the liner using handy clips. See Photo 10 for an example of the typical wire routing and terminal box installation.

The initial reading for the embedded gages were obtained immediately after the ring was erected, prior to installation of peagravel. The initial readings for the surface gages were obtained approximately two days after the erection of the ring following gage placement and mounting cement cure. The gages for each ring were then generally monitored on a daily basis for about two to three weeks after which the monitoring frequency was reduced to about once a week.

4.4.5 Tape Extensometer Chord Measurements

Tape extensometer measurements of selected tunnel chords were made using a SINCO Model 51855 Tape Extensometer system. This system includes a proving ring and adjustment screw for applying a reproducible tension, a standard engineer's measuring tape, and the stainless steel anchor points. Basically, the instrument consists of two anchor sockets, tape locking pin, tape reel, tape reading dial indicator, tension adjustment nut, and tension dial indicator and proving ring (see Fig. 23). The tension adjustment nut has a range of two inches. The standard engineer's tape has been modified by punching small holes at exactly two-inch intervals to accommodate the locking pin latching system and by fixing an anchor socket at the end of the tape. The anchor points consist of a 1-in.-diameter stainless steel sphere welded onto a 3/8-in.-diameter, 2½-in.-long stud. The anchor points were attached to the precast concrete tunnel liner by means of an expandable star anchor (see Fig. 23).

The procedure for obtaining the instrument measurements starts with the tape fully wound on the reel and the locking pin disengaged. The free end of the tape is attached to the anchor point forming a ball and socket type joint (see Photo 7). The instrument is then carried to the next anchor and attached in the same fashion. The tape is reeled in and the locking pin engaged until there is little or no sag in the tape. The tension adjustment nut is then turned until the proving ring dial gage reads zero (see Photo 8). The dial gage was precalibrated to read zero when a forty-pound tensile load was applied to the tape. The attachment sockets are then checked to be sure that they are seated properly on the anchor. Frequently, the socket was not seated properly and further adjustment of the tension adjustment nut was necessary. The instrument is then moved back and forth to the position of minimum tension to ensure that the minimum chord length is measured.

The actual reading is taken from the tape reading dial indicator and the tape at the point where it enters the latching end of the instrument. Temperature was recorded at the time of the measurement. See Photos 8 and 9 for examples of the data collection procedures used.

Periodically the extensometer was cleaned and the tape lightly oiled and wiped clean. Two standard reference points were installed in the GEI lab and measurements were taken periodically to determine background error and to check the tape calibration. A 40-lb weight was also used to periodically calibrate the proving ring dial.

Anchors were typically located in the same vertical plane perpendicular to the tunnel axis as the surface strain gages (see Table 2 and Fig. T-1). This was primarily done so that should any appreciable movements occur, correlations could possibly be made with respect to strain gage data and chord measurements in the same plane. The anchors were located adjacent to the upper and lower grout holes in side segments #2 and #3. Anchors were not located in the crown segments because of poor accessibility during construction. Anchors were also not located in the invert segment due to interference with the grout gantry and muck train.

The three chord distances shown on Fig. 24 were measured on approximately a weekly schedule after the trailing gear had passed since it was not possible to obtain these dimensions or other meaningful chord lengths when the trailing equipment was present.

4.4.6 Vibrating Wire Tunnel Piezometers

The piezometers installed through the tunnel lining at the nine test ring locations are the Model PW-100 vibrating wire type manufactured by IRAD/GAGE, Inc., Lebanon, NH with a range of 0 to 100 psi and a resolution of 0.1 psi. Prior to installation, each instrument was calibrated by the manufacturer and pressure tested by GEI in the lab. Some instruments (Test Sections 3, 6, 7, 8) were surrounded by Ottawa sand and are in 2-3/8-in.-O.D., 2-in.-I.D. slotted PVC tubing, approximately one foot long (see Photo 9). Some instruments (Test Sections 2, 4, 5, 9) were only surrounded by Ottawa sand for at least two feet ahead and behind the instrument.

The tunnel piezometers were installed in 2½-in.-dia. holes drilled by percussion air drills approximately six feet into rock adjacent to the tunnel lining at each test section. Three 2-in.-dia., 6-ft-long holes were drilled at or near each test section to intercept water-bearing joints in the rock. Those holes exhibiting the greatest amount of water flow were chosen and, if necessary, the hole was enlarged to accommodate the oversized PVC tubing. Prior to insertion of the instrument, the holes were blown out using compressed air to rid the hole of fines which could clog the porous stone tip. After insertion of instrument and Ottawa sand, lead wool was inserted in the hole. The remainder of the hole was filled with bentonite and plugged with styrofoam. The readout cables from these piezometers were then routed along with the strain gage leads to the terminal box at each test ring.

The tunnel piezometers were usually monitored on a weekly basis from the day of installation until the completion of the tunnel advance.

4.4.7 Surface Piezometers and Observation Wells

The Corps of Engineers installed 13 piezometers in the bedrock in 1976 (prior to award of the tunnel contract) which were designated as the FD-series. Details of the FD-series piezometer installations are not presented herein; however, the location and elevation of the piezometer tips are shown on Fig. F-1. The piezometers and observation wells installed under this contract (PZ-1 through 6, OW-1 and OW-2) were installed in 1977. The details of the PZ and OW series installations are presented on Figs. 25 through 32 and the locations and tip elevations are also shown on Fig. F-1.

All of the piezometers and observation wells shown on Fig. F-1 were read once per month starting in January 1978, except when individual instruments were inaccessible due to ice, snow or other obstructions. A select number of these piezometers were monitored on essentially a daily basis for a period from mid August 1979 to January 1980, when the tunnel heading was in the general vicinity of the respective instruments. This additional data has not been included on Fig. F-1 but is available in the appropriate weekly reports. All measurements were made using an electronic sounding device to determine the top of the water surface relative to the top of the protective casing at the ground surface.

4.4.8 Concrete Modulus Testing

The concrete cylinders obtained at the San-Vel casting yard during the pours for the instrumented segments were tested to determine the elastic modulus of the concrete by H. G. Protze, Inc., Newton Highlands, MA. The secant modulus of elasticity to 35% of the anticipated ultimate load was determined using two 2½ in. or 4½ in. electrical resistance strain gages mounted along the axis of the cylinder and diametrically opposed. After completion of the modulus testing, all cylinders were loaded to compressive failure.

4.5 Data Collected and Instrument Performance

4.5.1 General Summary

It should be noted that the measurements obtained for the various instruments used during this project are subject to a range of error depending on the instrument type and monitoring procedure used. Previous experience has indicated that the accuracy and repeatability of the measurements made using the various instruments is controlled by the instrument or instrument omponent in the measuring system with the lowest resolution or subject to the most operator error during the measurement procedure. Thus, the data that are presented in the Appendices and which are summarized in this section of the report are considered accurate to within the following ranges:

Type of Instrument	Probable Range of Error
MPBX	± 0.003 in.
Load Cell	± 115 lb
Strain Gage	± 5$\mu\varepsilon$ (1$\mu\varepsilon$ = 10^{-6} in/in)
Tape Extensometer	± 0.01 in.
Tunnel Piezometers	± 0.1 ft H_2O (± .05 psi)
Ground Surface Piezometers and Observation Wells	± 0.5 in.

The maximum rock movements measured by the MPBX's are summarized in Table 3 and the movement versus time are graphically presented in Appendix A. All of the MPBX instruments performed satisfactorily.

The data collected for the rock bolt load cells during the monitoring period from November 14, 1979 to December 15, 1978 are graphically presented in Appendix B. In general, the load experienced by the instrumented rock bolts increased very little, if at all, above the lock-off load during the monitoring period. The rock bolt load cells were quite prone to damage due to flyrock during the tunnel advance: 10 of the 12 cells required repair and/or reinstallation during this period.

The strain gage data obtained for the nine instrumented rings are graphically presented versus time in Appendix C and the maximum measured strains for each ring are presented on Table 4. For the most part the maximum strains measured were less than 200$\mu\varepsilon$. The ranges of measured maximum strain and percentages of gages within that range are summarized in the following table.

Maximum Measured Strain Per Gage ($\mu\varepsilon$)

	Percentage of Gages					
	0-100	100-200	200-300	300-400	400+	Damaged
Embedded	39.1	7.8	1.6	1.6	1.6	4.6
Surface	23.4	14.1	0	0	4.6	1.6
Total	62.5	21.9	1.6	1.6	6.2	6.2

Based upon an average of the elastic modulus measured in the concrete cylinders tested in this program, a factor of 4 psi/microstrain can be applied to the strain measurements to obtain a reasonable approximation of stress.

Of the 36 embedded gages installed, three were damaged or otherwise failed to function properly. Of the 28 surface gages installed, one was damaged and one operated but indicated questionable data.

The tape extensometer data for the monitored chords at each test section are graphically presented in Appendix D. Generally, with a few exceptions, the measured changes in the three chord lengths monitored at each test ring (18 total) were all within $\pm$ 0.10 in. based on the initial readings. These measurements were laborious to undertake but all equipment performed satisfactorily for the life of the job.

The data for the eight vibrating wire piezometers installed in the first eight instrumented rings are graphically presented in Appendix E. All eight of the instruments performed satisfactorily. However, four of the piezometers apparently were installed in boreholes which did not communicate with the surrounding groundwater since there was little agreement with these four tunnel piezometers and nearby piezometers installed from the ground surface.

The data for the 12 rock piezometers, 6 glacial till piezometers and 2 observation wells installed in the floodplain deposits are presented on Fig. F-1 in Appendix F. All of the piezometers installed in bedrock measured very large drops (100 to 130 ft) in the bedrock piezometric level as the tunnel heading approached and passed their locations. The piezometric level in the glacial till stratum was also found to decrease, but to a much lesser amount (20 to 25 ft) as the tunnel heading passed in the vicinity of an instrument. The observation wells did not appear to be significantly influenced by the tunnel construction. All of the instruments performed satisfactorily except one which was damaged by other construction activity in the area.

The data obtained for the concrete modulus testing is presented in Appendix G and generally indicated an elastic secant modulus ranging from 3.43×10^6 to 4.5×10^6 psi with a compressive strength ranging from 7470 to 8550 psi.

4.5.2 Summary By Test Section

The following discussions present a more detailed summary of the data collected at each of the ten instrumented test sections. As on Table 4, the strain gages will be referred to by segment number (1-invert, 2-right, 3-left, 4-crown) and mount type (E-embedded, S-surface), e.g., 2S = segment 2, surface gage.

Test Section #1

Test Section #1, which consisted of 12 rock bolt load cells and MPBX-1, was located between Stas 8+16 and 8+28 in the drill and blast section of the tunnel.

The data collected for the six anchors in MPBX-1 relative to the brass reference plate and assuming the reference plate fixed are shown in Fig. A-1a, Appendix A. This data would seem to indicate that the anchors and, thus the rock, are moving upward as the tunneling operation proceeds. In addition, the upward movement seems to be occurring well before the face of the tunnel has reached Sta 8+20 and all six anchors appear to be moving upward with an equal uniform magnitude. Such upward movement is unlikely. Based on these observations it is felt that the brass reference plate is actually moving downward relative to the stainless steel rods, giving the impression of upward rock movement. This conclusion is borne out by the optical survey measurements which indicate a downward movement of the reference plate. However, due to the difference in resolution of at least an order of magnitude between the micrometer and the optical survey equipment and the lack of a proper bench mark at the beginning of the survey measurements, no rigorous comparison of the data can be made. Since the reference plate and head of MPBX-1 is securely attached to the NW casing seated in the bedrock, the reason for this apparent settlement of the reference head is not readily apparent but may be due to an elastic shortening of the steel casing to which the reference head is attached caused by downdrag forces from the surrounding varved clay stratum.

A reasonable analysis of the data can be made if the uppermost anchor (#6) is considered fixed and the remaining five anchor movements are plotted relative to it. These data are presented in Fig. A-1b. Since anchor #6 is located approximately 47 ft above the tunnel crown, the assumption of zero or negligible movement of anchor #6 seems appropriate.

As can be seen in Fig. A-1b, the anchor movements relative to anchor #6 are quite credible in their direction and magnitude. The maximum rock movement which was detected by MPBX-1 is on the order of 0.03 inches downward at anchor #1, which is located approximately 2.8 ft above the tunnel opening. The magnitude of the rock movement decreased moving upward from the tunnel opening, with a downward movement of about 0.005 in. detected at anchor #5, which is located approximately 24 ft above the crown. The majority of the movement occurred in all the anchors between the time the tunnel face reached Sta 8+20 until it had progressed to about Sta 8+60. It also seems as though some upward movement of the rock may have occurred as the face approached Sta 8+20, but this fluctuation in the data may be due to dirt on the top of some of the stainless steel rods at the time of these measurements. Based on these data, it would appear that all downward movement of the rock above the tunnel crown stabilized after the tunnel face was located about 50 ft past the MPBX position.

The data obtained for the 12 rock bolt load cells, during the monitoring period from November 14, 1978 to December 15, 1978, are graphically presented in Figs. B-1 through B-6, Appendix B. The position of the tunnel face relative to each load cell station is shown on the plots. Significant events which occurred during the monitoring period are also noted. The locations given for the load cells are approximate since it was considered more important to place the cells against flat, sound rock than at a particular grid coordinate.

Eight of the load cells were damaged during the monitoring program by flyrock from blasting. Six of these damaged cells (Nos. 1, 2, 7, 9, 11 and 12) were removed, repaired, and reinstalled. The repairs consisted of mending or replacing damaged cable and/or connectors. Two of the cells (Nos. 3 and 6) required reinstallation after the rock against which they were bearing was blown away during the heading advance.

The daily fluctuations in the readings recorded for all the cells were due either to moisture entering the signal cable through minute cuts in the vinyl casing which resulted from blasting or to moisture at the connection to the readout box. Precautions were taken to protect the instruments from water. These included the taping of all connectors after each reading, wrapping the connectors in plastic bags, and mending all cuts in the cable as soon as they were detected.

Fluctuations of approximately ± 115 lbs can be expected with these cells between readings, since the repeatability of the cells is about ± five digits over the calibration range and the average calibration factor was about 22.5 lbs/digit. The temperature in the tunnel at the location of the load cell installations was recorded for every reading of the cells, but no correlation of temperature variation with cell reading fluctuation was apparent.

It was observed that 7 of the 12 load cells indicate a peak point occurring on December 8, 1978. This relatively sharp apparent increase in load was noted during the data collection and the cells were read again the same day. Special care was taken to dry all connections as well as the interior of the readout box. All readings repeated to within five digits of the values obtained earlier in the day. The major tunnel construction activity on December 8 was the grouting of the outlet shaft. However, it is not readily apparent why this activity would produce an increase in load which would again sharply drop off.

Test Section #2

Test Section #2 consisted of MPBX-2 and instrumented liner Ring #15 which included the standard seven strain gage array, piezometer, and chord measurements. The center of this test section was located at Sta 11+32.

The data collected for the five lower anchors in MPBX-2 were plotted relative to anchor #6 for the reasons discussed for Test Section #1 and are presented on Fig. A-2. The maximum anchor movement was measured in the #1 anchor and is of the order of 0.26 in. downward. The anchor movement and corresponding rock movement attenuated moving upward from the tunnel opening, with a maximum downward movement of 0.022 recorded for the #5 anchors, located about 20 ft above the tunnel crown. Practically all of the measured rock movement occurred as the tunnel heading progressed - about 100 ft past the instrument location (Sta 11+32).

The strain gages on Ring 15 were installed on September 7, 1979 and were monitored for approximately 45 weeks. This data is presented on Fig. C-1. The maximum strain measured was +538$\mu\varepsilon$ (compressive) measured 37 weeks after installation in the 4S gage. The 4S gage increased to +214$\mu\varepsilon$ less than one week after gage installation and the compressive strain gradually increased during the next 36 weeks to its maximum and remained stable ($\pm$10$\mu\varepsilon$) for the following six weeks when monitoring ceased. Gage 2S increased to +109$\mu\varepsilon$ four weeks after installation and continued a gradual trend toward increasing compressive strain during the remaining 41 weeks of monitoring, attaining +156$\mu\varepsilon$, the maximum strain measured for this gage at the end of this period. Gage 3S reached -107$\mu\varepsilon$ (tension) less than two weeks after installation. The gage indicated relatively erratic strains varying as much as 140$\mu\varepsilon$ during the remaining 43 weeks of monitoring. The embedded gages in this ring all indicated changes in strain less than 100$\mu\varepsilon$ for the entire 45 week period. All of the gages for this ring, except the last reading for 3S, generally indicated increasing compressive strain over the course of the monitoring period.

The tape extensometer data are presented on Fig. D-1, Appendix D. All of these data have been corrected for the expansion or contraction of the steel measuring tape due to changes in the tunnel atmosphere temperature between readings. Chord length 8-3 indicated an increase in length of about 0.05 in. while chords 7-2 and 7-3 both decreased in length a little over 0.1 in. This could indicate that segment 3 near point 9 may have rotated slightly away from the tunnel opening while the other three points may have moved inward.

The data obtained for the vibrating wire piezometer at Test Section #1 are presented on Fig. E-1, Appendix E. The elevation of the piezometric surface measured in the bedrock by this instrument agreed to within 2 ft of the elevation measured in nearby surface piezometer FD-27 (see Appendix F), located about 7 ft off of the tunnel springline.

Test Section #3

Test Section #3 consisted of MPBX-3 and instrumented Ring #81 which included the seven strain gage array, piezometers, and chord measurements. The center of this test section was located at Sta 15+25.

The deflection data collected for the five lower anchors in MPBX-2 were plotted relative to anchor #6 for reasons similar to those discussed for Test Sections #1 and #2 and are presented on Fig. A-3. The maximum measured anchor movement occurred in the two anchors closest to the tunnel opening (#1 - 2.4 ft, #2 - 3.9 ft) and were about 0.16 in. downward. The rock movement as measured by the three higher anchors attenuated moving away from the opening, with a maximum downward movement of 0.02 in. recorded for anchor #5, located about 26.7 ft above the crown. Almost all of the measured movement was recorded as the tunnel heading progressed about 100 ft past the instrument location (Sta 15+25).

The installation sequence for Ring 81 was different than that followed for the other instrumented rings included in this project. The peastone and grout were placed around this ring immediately after erection, as opposed to the other eight instrumented rings for which the peastone was installed about two to three rings back from the most recently completed ring and the grout was not installed for two to five hundred feet after the ring was erected. The strain gages on Ring 81 were installed on October 19, 1979 and were monitored for approximately 39 weeks. The data is presented on Fig. C-2. The maximum strain measured was $+237\mu\varepsilon$ in the 2E gage during the last week of monitoring. Prior to this, the 2E gage increased in compressive strain to $+72\mu\varepsilon$ one week after installation and for the next 25 weeks gage 2E remained stable ($\pm 15\mu\varepsilon$). During the remaining 13 weeks of monitoring gage 2E indicated a gradual increase in compressive strain to its maximum at the time of the last measurement. Gages 1E and 3E indicated trends somewhat similar to 2E during monitoring. Both had maximum strains in the 100-200$\mu\varepsilon$ range. One week after installation gages 1E and 3E indicated +60 and +82$\mu\varepsilon$, respectively. For the remaining 38 weeks there was a very slight trend toward increasing compressive strains in both gages. Gages 2S and 3S indicated similar trends during monitoring, i.e., increasing compressive strain, and both were in the 0 to 100$\mu\varepsilon$ range at the last measurement. The gage 4E readout cable was severed during removal from the embedded storage box during final wiring up to the terminal box. Readings obtained before the cable was damaged indicated +79$\mu\varepsilon$ one week after installation. On April 23, 1980 it was discovered that readout cables leading to the terminal box from all of the strain gages were severed, presumably by the rolling grout gantry which had just passed. All cables were respliced and repaired by May 8, 1980.

The tape extensometer data, including temperature corrections, are presented on Fig. D-2. All three chord lengths decreased about 0.10 in. within the first month of monitoring and remained in that position, with some fluctuation, for the remainder of the monitoring period.

The data obtained for the vibrating wire tunnel piezometer are presented on Fig. E-2. For the first four months after installation this instrument appeared to be responding to an increase in hydrostatic pressure. However, the readings then began to decrease contrary to the levels measured in nearby surface piezometer FD-29. We feel that the poor agreement with FD-29 was due to the fact that a joint was not intercepted with the borehole in which the piezometer was installed which could provide adequate communication of the hydrostatic pressure in the adjacent bedrock. During an attempted removal of the instrument to check it for proper operation, the piezometer was irrepairably damaged.

Three new boreholes were drilled through the grout holes in the segment adjacent to Ring 81 to attempt to reach a water-bearing joint. The hole drilled at Ring 83 provided a large inflow of water and a new piezometer was installed at this location. However, it was not possible to adequately seal this piezometer in the bedrock and some leakage continued to occur through the grout hole. The readings presented for June and July 1980 reflect this leakage.

Test Section #4

Test Section #4 consisted of MPBX-4 and instrumented Ring 225. The center of this test section was located at Sta 24+00.

The deflection data collected for the six anchors in MPBX-4 relative to the brass reference plate are presented on Fig. A-4. The maximum anchor movement was measured in anchor #1 and was about 0.07 in. downward. The movement attenuated away from the tunnel opening with no practical movement detected at the anchor #5 or #6 locations. Practically all of the measured movement occurred as the tunnel heading progressed about 50 ft past the instrument location (Sta 15+25). Fig. A-4 shows some apparent additional downward movement occurring in April 1980. However, it is assumed that this is not true rock movement due to the similar magnitudes measured for all six anchors and is probably due to a slight heave of the instrument head or reference plate.

Measurements of the strain gages at Ring 225 were carried out for approximately 33 weeks after installation of the strain gage on December 4, 1979 and the data are presented on Fig. C-3. The maximum measured strain was +136$\mu\varepsilon$ obtained for the last measurement of gage 3S. The 3S gage exhibited tensile strains of up to -53$\mu\varepsilon$ two weeks after installation. Following grouting, two weeks after gage installation, the 3S gage indicated a compressive strain of +19$\mu\varepsilon$. The strains remained stable ($\pm$15$\mu\varepsilon$) during the next 20 weeks. In the remaining 11 weeks, gage 3S indicated strains with a trend toward increasing compression to its maximum at the last measurement. All of the embedded gages and gage 2S indicated strains in the 0 to 100$\mu\varepsilon$ range during the entire monitoring period. However, 2S did not exhibit the general increase in compressive strain noted for the other gages on this ring. The end mounts for gage 4S were damaged during installation and initial tensioning of the gage. Attempts were made to repair the mounts but none were successful while the trailing gear was in position to provide access and make repairs.

The tape extensometer data for Test Section #4 are presented on Fig. D-3. Overall, chord lengths 8-3 and 7-2 indicated basically no change in length with chord length 7-3 decreasing by about 0.075 in.

The data obtained for the piezometer at this test section is presented on Fig. E-3. It appears obvious from this data and from the data obtained in nearby surface piezometer FD-30 that this instrument was not monitoring the hydrostatic pressure in the rock adjacent to the tunnel. In April 1980 the piezometer was removed from the borehole drilled at Ring 225 and was found to be operating properly. Two new boreholes were then drilled at adjacent Rings 226 and 227 in an attempt to intercept a hydrostatically communicative joint system. The piezometer was reinstalled at Ring 227 and it appears from the data obtained in June and July that such a joint system was not located.

Test Section #5

Test Section #5 consisted of MPBX-5 and instrumented Ring 275. The center of this test section was located at Sta 27+00.

The deflection data collected for the six anchors in MPBX-5 relative to the brass reference plate are presented on Fig. A-5. The lower five anchors all showed very small upward movement, the largest being -0.03 in. for anchors 1 and 2. Most of the measured movement occurred prior to the heading progressing 150 ft past the instrument. There was some attenuation of movement noted in the anchors progressing upward from the tunnel and no reason to suspect that the instrument was not functioning properly. Therefore, it is assumed that the measured upward displacement is true movement.

The strain gages on Ring 275 were installed on December 17, 1979 and were monitored for approximately 30 weeks. The data are presented on Fig. C-4. The maximum strain measured was $-351\mu\varepsilon$, at gage 1E, one week after ring erection. For the remaining 29 weeks of monitoring the strains measured in gage 1E indicated a slight trend toward lesser tensile strains with the last measurement indicating $-305\mu\varepsilon$. Gage 4S increased in compressive strain to $+78\mu\varepsilon$ four weeks following gage installation. Subsequent strain measurements of the 4S gage indicated a slight trend toward increasing compressive strains. The maximum strain measured by gage 4S was $+129\mu\varepsilon$ approximately 28 weeks after gage installation. Measurements of embedded and surface gages on segments 2 and 3 exhibited strains in the $0\text{-}100\mu\varepsilon$ range with no definite trends apparent except for a small increase in compressive strain at 2S and 3S. Measurements obtained from gage 4E indicated strains in the 0-100$\mu\varepsilon$ range with a general increase in compressive strain strain during the first 23 weeks of monitoring. The final measurement of gage 4E indicated a compressive strain in excess of 200$\mu\varepsilon$ and, because of the relative inconsistency with prior measurements, is believed to be an erroneous measurement.

The tape extensometer data for Ring 275 are presented on Fig. D-4. Chord 8-3 and 7-2 measurements indicated an increase and decrease of about 0.1 in. and 0.075 in., respectively, within eight days after monitoring began and remained at the new lengths, except for a fluctuation at the end of April, until readings were terminated in July. Chord 7-3 measurements tended to fluctuate during the seven-month monitoring period with a general increase in length of about 0.05 in. noted.

The data obtained for the piezometer at this test section are presented on Fig. E-4. As with the data obtained for the piezometer at Ring 225, there was little agreement noted between this instrument and the closest surface piezometer at FD-30. As a result, this piezometer was pulled and tested at the end of May and was found to be functioning properly. Two new boreholes were drilled at adjacent Rings 274 and 276, but no groundwater inflow was detected from any of the three boreholes at this test section. The instrument was reinstalled at Ring 276.

Test Section #6

Test Section #6 consisted of MPBX-6 and instrumented Ring 548. The center of this test section was at Sta 43+50.

The deflection data obtained for the six anchors in MPBX-6 relative to the brass reference plate are shown on Fig. A-6. The maximum movement of 0.07 in. toward the tunnel was measured in anchor #1. The movement measured by anchors #3 through #6 indicated attenuating rock movement progressing up from the tunnel. Practically all of this movement was measured as the heading progressed about 125 ft past the instrument. No movement was measured for anchor #2 which is not reasonable based on the relative movement of the other anchors. However, the reason for the malfunction of this anchor point is not apparent.

The strain gages on Ring 548 were installed February 11, 1980 and were monitored for approximately 23 weeks. The data are presented on Fig. C-5. The maximum strain measured was +514$\mu\varepsilon$, obtained 23 weeks after gage installation in the 4S gage. The strain measurements of the 4S gage exhibited a definite trend toward increasing compressive strain during the last 21 weeks of monitoring. Measurements of gages 4E and 3E indicated a similar trend toward increasing compressive strains as in 4S but the maximum strains measured were +165$\mu\varepsilon$ for the final measurement of gage 4E and +105$\mu\varepsilon$ for gage 3E 21 weeks after initial measurements were made. Measurements of gages 1E, 2S and 3S indicated strains in the 0-100$\mu\varepsilon$ range during the entire monitoring period with some increase in compression. Some gage 2E measurements were obtained while at the San-Vel storage yard. However, following shipment of the instrumented segments to Hartford, the gage failed to respond to all attempts at monitoring. The reason for this failure is not apparent.

The tape extensometer data for Ring 548 are presented on Fig. D-5. All three of the measured chords generally tended to shorten over the six-month monitoring period, i.e., chord 8-3 about 0.05 in., chord 7-2 about 0.05 to 0.075 in., and chord 7-3 about 0.1 in. All three of the chord lengths exhibited fluctuations during this period.

The data obtained for the tunnel piezometer at this section are presented on Fig. E-5. In the last three months of monitoring the elevation of the piezometric surface as measured by this instrument was about 12 ft lower than that measured by nearby surface piezometer FD-12 (see Appendix F) which is located about 18 ft from the tunnel springline. This difference may be due to a steep depression of the drawdown curve, closer to the tunnel excavation as groundwater drains into the tunnel. It can also be postulated that some leakage may be occurring from the hole in which the tunnel piezometer is installed since a drop of about 15 ft of head occurred during the last three months of monitoring which was not reflected in the FD-12 data. Without this drop, the hydrostatic pressure measured by the tunnel piezometer would be only 2 to 3 ft lower than that measured in FD-12.

Test Section #7

Test Section #7 consisted of MPBX-7 and instrumented Ring 797. The center of this test section was at Sta 58+55.

The deflection data obtained for the six anchors in MPBX-7 relative to the brass reference plate are shown on Fig. A-7. A maximum movement of 0.18 in. toward the tunnel opening was measured in anchor #1. The remaining five anchors indicated attenuating rock movement away from the tunnel opening with the movement measured by anchors #3 and #4, located about 8 ft and 12 ft above the crown, both indicating about 0.07 in. of downward movement. Most of the movement measured for all of the anchors occurred as the heading progressed about 125 ft past the instrument.

The strain gages on Ring 797 were installed March 18, 1980 and monitored for approximately 18 weeks. The data are presented on Fig. C-6. The maximum strain measured was +162με for the last measurement made at gage 3S. During the first two weeks after gage installation, strain measurements at 3S indicated tensile strains of approximately -10με ($\pm$5με). Approximately three weeks following gage installation strain measurements indicated compresive strains of +30με($\pm$10με). During the fourth week of monitoring, strain measurements increased overnight to +130με. Subsequent measurements obtained during the following three weeks exhibited a decreasing compressive strain trend reaching +78με. For the remainder of the monitoring period the decreasing compressive trend was reversed. Measurements obtained from embedded and surface gages on segment 4 exhibited trends toward increasing compressive strain attaining measured maximum strains of +95με and 152με, respectively, in the 16th and 15th week of monitoring, respectively. Measurements of gages 1E, 2E, and 2S have indicated strains in the 0 to 100με range with a very slight trend toward increasing compressive strain.

The tape extensometer data for Ring 797 are presented on Fig. D-6. All three of the measured chord lengths tended to shorten by about 0.03 in. during the four-month monitoring period. The last readings obtained in July 1980 indicated that the chord lengths had practically returned to their initial values.

The data obtained for the tunnel piezometer are presented on Fig. E-6. These data agree closely with the levels measured in surface piezometer FD-22A (see Appendix F) which is located about 31 ft from the tunnel springline. The approximate 7-ft difference in head can reasonably be explained by a deeper groundwater drawdown closer to the tunnel as discussed for Test Section #6.

Test Section #8

Test Section #8 consisted of MPBX-8 and instrumented Ring 840. The center of this test section was at Sta 61+19.

The deflection data obtained for the six anchors in MPBX-8 relative to the brass reference plate are shown on Fig. A-8. Since the installation of this instrument was not completed until the tunnel heading had progressed about 75 ft past the instrument and the grout may not have had a sufficient time to set until the heading had progressed over 150 ft, total rock movements could not be measured. The only conclusions which can be made is that essentially no rock movement was detected above the crown of the tunnel at this location after the heading had progressed 100 ft beyond it.

The strain gages were installed on Ring 840 on March 25, 1980 and monitoring continued for approximately 17 weeks. The data are presented on Fig. C-7. The maximum measured strain was +164$\mu\varepsilon$ which occurred in the 2S gage in the 17th week. Gage measurements obtained from 3S and 4S indicated maximum strains of +131$\mu\varepsilon$ and +99$\mu\varepsilon$ in the 16th week of monitoring. Measurements obtained during the entire monitoring period indicated maximum strains in the $\pm$ 0 to 100$\mu\varepsilon$ range for all of the embedded gages. There was a general trend of increasing compressive strain noted in all of the strain gages for this ring during the 3½-month monitoring period.

The tape extensometer data for Ring 840 are presented on Fig. D-7. During the 3½-month monitoring period an increase of about 0.04 in. was recorded in chord lengths 8-3 and 7-2, while chord length 7-3 decreased by about the same amount.

The data obtained for the tunnel piezometer installed at Test Section #8 are presented on Fig. E-7. These data agree closely with the hydrostatic levels measured in nearby surface piezometer FD-18 which is located about 21 ft from the tunnel springline (see Appendix F). The approximate 8-ft difference in head can probably be explained by deeper groundwater drawdown closer to the tunnel as discussed for Test Sections #6 and #7.

Test Section #9

Test Section #9 consisted of MPBX-9 and instrumented Ring 1324. The center of this test section was at Sta 91+00.

The deflection data obtained for the six anchors in MPBX-9 relative to the brass reference plate are shown on Fig. A-9. The August 15, 1980 data were supplied by the COE. The maximum anchor movement of 0.21 in. was recorded in anchor #1 with the apparent rock movement attenuating rapdily to anchor #3 which was located about 8 ft above the crown and which indicated practically no downward movement. The relative displacement of the three higher anchors suggests some slight upward rock movement, but it is not certain whether this was true movement or related to settlement of the reference head. Practically all of the measured rock movement occurred as the tunnel progressed about 100 ft past the instrument location.

The strain gages were installed on Ring 1324 on June 4, 1980 and monitoring continued for approximately nine weeks. The data are presented on Fig. C-8. The August 7, 1980 data were supplied by the COE. The maximum strain measured was $-142\mu\varepsilon$ at gage 1E the third week following gage installation. Subsequent measurements of gage 1E indicated a trend toward lesser tensile strains and, eventually, in the sixth week of monitoring indicated a compressive strain of $+56\mu\varepsilon$. Gage 4S measurements generally indicated a continued trend toward increasing compressive strain with the final measurement attaining $+79\mu\varepsilon$. Gages 2E, 2S, 3E, 3S, 4E, all indicated tensile strains in the 0 to $100\mu\varepsilon$ with the exception of gage 2S, measured during the sixth week, which indicated a compressive strain of $+9\mu\varepsilon$. There was a general trend during the last month of monitoring of slight increasing compressive strain.

The tape extensometer data for Ring 1324 are presented on Fig. D-8. During the one-month monitoring period, chord 8-3 indicated a decrease in length of about 0.1 in., chord 7-2 increased in length about 0.08 in. and chord 7-3 showed no definitive trends.

The data obtained for the tunnel piezometer are presented on Fig. E-8. The August 7, 1980 data were obtained from the COE. There were no nearby surface piezometers installed into bedrock which were monitored during this program. However, it appears from the data that this instrument is functioning properly and is responding to the hydrostatic pressure adjacent to the tunnel at this location.

Test Section #10

Test Section #10 consisted of MPBX-10 and instrumented Ring 1399. The center of this test section was at Sta 95+54.

The deflection data obtained for the six anchors in MPBX-10 relative to the brass reference plate are shown on Fig. A-10. The August 12 and 15, 1980 data were supplied by the COE. The maximum anchor movement recorded as of August 15, 1980 was 0.62 in. downward at anchor #1. Although the majority of the anchor movements were measured as the heading advanced about 100 ft past the instrument (to the end of the drive at about Sta 98+30), there is some time dependent movement observable in the data. Two months after the passage of the heading below MPBX-10 there still appears to be small downward movement occurring in all of the anchors. The anchors also indicated attenuating movement progressing upward from the tunnel crown. It also appears that some upward movement on the order of 0.04 in. occurred as the heading approached the MPBX-10 location.

The strain gages on Ring 1399 were installed on June 18, 1980 and were monitored for approximately nine weeks. The data are presented on Fig. C-9. The August 15, 1980 data were supplied by the COE. The maximum strain measured as of August 15 was +1646$\mu\varepsilon$ at the 4S gage in the ninth week of monitoring. The 4S gage measurements obtained during the first four weeks of monitoring exhibited relatively large and rapidly increasing compressive strains. Ring 1399 was contact grouted in place during the latter part of the third week and beginning of the fourth week. An alternate surface gage (4S-A) was installed near gage 4S at the beginning of the fourth week (see Table 2) to act as a check and backup gage for the original 4S gage. In anticipation of measuring high compressive strains, gage

4S-A was set with a high initial tension. Since installation, the alternate gage has followed a trend of increasing compression similar to the original 4S gage, although the magnitude of increase has been less. (4S has increased by about 240$\mu\varepsilon$ while 4S-A has increased by about +88$\mu\varepsilon$ during the last five weeks.) This difference in magnitude could be a result of a number of factors, including slightly different gage alignments or different locations on the segment. Therefore, it is felt at the time of this writing that the original 4S gage is performing properly and that segment 4 has experienced fairly large changes in strain since installation. The embedded gage in segment 4 failed to operate after installation of the liner ring in the tunnel. Measurements obtained from gage 1E indicated -356$\mu\varepsilon$ one day after initial dead load measurements were taken. Subsequent measurements continued to indicate tensile strains, with no definitive trend apparent, in the -360$\mu\varepsilon$ to 430$\mu\varepsilon$ range for the remainder of the monitoring period. Measurements of embedded and surface gages on segments 2 and 3 indicated predominantly tensile strains in the 0 to 100$\mu\varepsilon$ range.

Tape extensometer monitoring points were not mounted on Ring 1399 and the tunnel piezometer was not installed at this test section due to lack of access and interference caused by the trailing gear.

4.5.3 Surface Piezometer and Observation Wells

Fluctuations from the Connecticut River have been found to influence the groundwater elevation at the eastern end of the tunnel and, as a result, the project has been separated into two data zones. Zone I includes monthly groundwater data from the area effected by the Connecticut River, which encompasses Sta 6+25 to Sta 32+00, and are presented on Fig. F-1, Appendix A. Zone II includes monthly groundwater data from the area between Zone I and the Park River, which is from Sta 32+00 to Sta 98+25, and are also presented on Fig. F-1. The locations and elevations of the piezometers and observation wells and the proposed tunnel alignment are shown on the figures in addition to notations indicating significant events which occurred during the monitoring period.

The groundwater data collected in Zone I exhibited some observable trends in the data attributable to various construction phases, the strata in which the particular piezometer was located, and the fluctuations of the Connecticut River:

1. Prior to the start of the machine tunneling, the piezometric levels were observed to be influenced by the operation of the shaft dewatering wells at all locations except PZ-3, PZ-4, OW-1 and OW-2. This phenomena was exhibited by the rebound of the piezometric pressures which occurred in June 1978 at FD-27, 29 and 30 and PZ-1 and 2, one month after the dewatering wells were grouted. During this period the piezometric and groundwater elevations at PZ-3 and 4 and OW-1 and 2, respectively, dropped in elevation as did the Connecticut River. The dewatering of the shaft depressed the groundwater measurements, the largest amount being near the shaft had a decreasing effect with distance from the shaft.

2. In November 1978, the piezometric pressure in FD-29 dropped below PZ-1 which is located about 130 ft closer to the shaft than FD-29. The piezometers in FD-29 are located in bedrock which is bedded and contains some joints. PZ-1 is located in the red sandy silt to silty sand (i.e., glacial till) deposit overlying bedrock. A possible explanation for this response may be that the bedrock contains joints which give it a relatively high permeability that provided hydraulic communication between the shaft and tunnel excavation and the piezometers at FD-29. As a result, flow through the jointed rock may have caused more rapid dissipation of pore pressures than in less permeable material such as the overlying glacial till.

3. From January 1978 to July 1979 the piezometric levels within the rock and groundwater levels in the overburden soils seemed to fluctuate with the Connecticut River. This fluctuation, although indicating definite signs of drawdown of 20 to 25 ft due to the advance of the TBM tunnel, was also apparent in the overburden piezometers (PZ series) for the remainder of the project. However, any response fluctuations of the rock piezometers (FD) after the start of the machine tunneling were masked by the large drawdowns of 100 to

over 115 ft monitored in these instruments. There was no apparent tunnel-related drawdown measured in the overburden observation wells (OW) and the groundwater level in these instruments continued to fluctuate due to seasonal effects for the entire monitoring period.

4. The large drops measured in the rock piezometers (FD) seemed to begin when the tunnel heading was 300 to 500 ft before the respective instrument. After passage of the heading and installation and grouting of the liner, the groundwater level in these piezometers began to recover by about 10 to 45 ft during the remainder of the monitoring period.

The data from the piezometers located in Zone II indicate that the piezometric levels in this zone have not been noticeably affected by the Connecticut River. This is evidenced by the data collected prior to the large drawdowns of 110 to 130 ft after January 1980 due to the tunnel advance which showed no real correlation with the Connecticut River fluctuations. There also did not appear to be any noticeable recovery of the large drawdowns in the groundwater levels in Zone II as observed for Zone I during the last 10 months of monitoring. Although there is limited data for the piezometers in the glacial till in this zone, drawdowns of about 14 ft at PZ-5 and 8 ft at PZ-6 were recorded at the end of the monitoring period.

Another observation may be made relative to the drawdowns measured in the bedrock by the FD-series piezometers in both zones. It appears that the drawdown of the piezometric surface in the bedrock is not uniform progressing away from the tunnel to the ground surface. The large drawdown near the tunnel opening is indicated by the data from FD-27A and B, FD-29A and B, and FD-12A and B in which the piezometer tips are located at the tunnel elevation or at a maximum of about 5 ft above the crown as in the case of FD-29B. The bottom of piezometers FD-19A and FD-24 are also located near the tunnel crown and the magnitude of the drawdowns measured in these two instruments agrees closely with the above noted piezometers. However, FD-19B, with its tip about 15 ft above the crown, and FD-22B, with its tip about 25 ft above the crown, show higher piezometric levels than their relative "A" instruments by about 40 ft and 100 ft, respectively. Therefore, it appears that the magnitude of the drop in piezometric level decreased proceeding away from the tunnel similar to drawdown observed near a pumped well.

All of the instruments performed satisfactorily for most of the monitoring period except for FD-19A and B which were irrepairably damaged in April 1980 during sidewalk repairs in the area and FD-30B in which a small stone became lodged in the riser tube during monitoring in April 1980.

4.5.4 Concrete Modulus Testing

The data reports obtained from H. G. Protze, Inc. for the concrete modulus tests performed on the cylinders obtained during the casting of the concrete liner segments are presented in Appendix G.

The cylinders that were obtained coincide to different pour days, different pours during the same day, and different cure ages roughly corresponding to different ring installations. Therefore, the data have been summarized on Table 5 so that the modulus values obtained may be correlated with their respective ring segments.

4.6 Interpretations and Conclusions

Rock Movements

Based on the data obtained for the movement of the MPBX anchors during tunnel construction, the rock movement that was measured over the crown of the tunnel excavation was relatively small in the areas of competent shale and sandstone bedrock. The movements recorded two to four feet above the crown at the first nine test section locations, except TS-8, ranged from 0.03 to 0.26 in. (Note MPBX-8 is not included since its installation was completed after the passage of the tunnel heading.) The movements recorded at MPBX-1 in the drill and blast section were the smallest observed during the project. This may be partially due to the fact that the top portion of this section of tunnel was first excavated (approximately 16 ft high) and the roof bolts were installed before the lower bench was removed. Since the initial opening was smaller than in the TBM driven tunnel, the stress redistribution around the tunnel and resulting rock movement may have been smaller. When the bench was removed, the additional stress which needed to be transferred was probably accomplished more uniformly and distributed further into the rock mass due to the rock bolts.

The larger measured rock movements (MPBX-7 and 9) were measured in areas that were identified as possible fault zones. (The large movements recorded in MPBX-10 will be discussed below.) The large movements measured at MPBX-2, where no apparent fault zone was determined, may be partially due to start-up procedures and the early part of the learning curve for the tunneling crew which is inherent in every job of this type.

The rate of the measured rock movement at the first nine MPBX locations appeared to be somewhat dependent on the rate of heading advance in the zone approximately 50 to 100 ft past the respective instrument locations. In all of these instruments practically all of the measured rock movement occurred as the heading passed through the 100 ft zone, with most of the movement measured as the heading advanced through the first 50 ft of the zone. For example, at MPBX-2 the tunnel heading advance was slower than at MPBX-7 and the measured rock movement also seemed to develop more slowly (see Figs. A-2 and A-7). While the rate of movement seems somewhat dependent on the rate of heading advance, there was no apparent correlation of the magnitude of rock movement at the various instrument locations and the rate of tunnel advance.

The reason for the apparent upward rock movement recorded at MPBX-5 cannot be readily explained. The movement appears to be attenuating toward the ground surface, and therefore, it is unlikely that the reference head is settling (see discussion Section 4.4). The movement also seemed to continue until the heading had progressed about 175 ft past the instrument which is probably well past any influence due to the compressive forces exerted by the TBM and before any grouting of the ring had taken place in the area.

The rock movements which were measured over the tunnel crown were the most substantial at Test Section #10 as measured by MPBX-10. While a large percentage (approximately 80%) of the measured movement occurred as the heading progressed 100 ft past the instrument, there was some additional movement measured up to 1½ months after the heading exited this zone. A small upward movement can also be observed as the tunnel heading approached MPBX-10. These phenomena were not present or as pronounced at the other MPBX locations and are most likely a result of the highly fractured and weathered nature of the shale and sandstone at Test Section #10, which is located in a major fault zone. The clay infilling in the numerous joints and apparent decomposition of a large portion of the shale rock mass would account for the larger movements and "creep" of the rock mass above the tunnel opening.

All ten of the MPBX's indicated that the measured rock movements attenuated proceeding upward from the tunnel opening.

The rock bolt load cell data are also an indication of the relatively small rock movement and redistribution of most of the rock stresses fairly close to the advancing face. The lack of load pickup by the instrumented rock bolts may be a result of installing the cells approximately one tunnel diameter behind the face, after most of the rock movement had taken place. This hypothesis is supported by the data from MPBX-1 which indicated that most of the downward rock movement had occurred by the time the face had reached Sta 8+50. In addition, the data from MPBX-1 also indicated that the rock movement at the roof of the tunnel was very small (on the order of 0.03 in.) and, thus, most of the rock load was probably redistributed by arching in the rock mass with very little load transferred to the rock bolts.

Structural Liner Response

The strain gages located on the precast concrete liner segments in most of the test sections measured relatively small strains in the liner at these locations. Therefore, it seems that during the monitoring period which this report covers, very little rock load has been transferred to these liners. This would tend to agree with the observations concerning rock movement which appeared to have essentially ceased before the liner installation was completed (i.e., peastone and grouting) at each location to transfer the load. The major exceptions to this occurred at Test Sections 2, 6 and 10 where maximum compressive strains at the crown segment of over 500$\mu\varepsilon$ were recorded. A possible explanation of these trends is as follows:

Test Section 2: Only about 60% of the total rock movement had occurred at this location before Ring 15 was at least partially installed (i.e., some peastone, but no grout). Therefore, some load could have been transferred to the liner as the rock continued to move.

Test Section 6: MPBX-6 at this location indicated slight upward rock movement as the heading progressed. It could be possible that since no downward movements were recorded at this location the stresses induced in the rock close to the tunnel opening are higher than at most of the other locations, and therefore, more load is being transferred to this liner ring.

Test Section 10: The very large compressive strain measured on the crown segment at this location can be correlated with the rock movements monitored by MPBX-10. Only about 50% of the measured downward rock movement had occurred at this location by the time instrumented Ring 1399 was installed and peastone placed behind it. Evidently there has been substantial load transfer to the crown segment as the rock moves downward. Stress redistribution within the rock mass would also be less in this area due to the general poor quality of the rock.

A trend which was generally noted in most of the strain gages was a decrease in tensile strain, which was initially acquired after ring erection, or an increase in compressive strain with time. By the end of the monitoring period, most of the gages were indicating compressive strain or were moving in that direction as the loads were redistributed around the ring. Ring 81 (Test Section 3), which had a different installation procedure than the other eight instrumented rings in that it was grouted immediately after erection, responded somewhat differently than the other rings during the early period following ring erection. It appeared as though the installation of the grout at the earlier stage caused the ring to develop a more compressive load distribution sooner than the other rings which were instrumented. The earlier grouting did not seem to have an effect on the relative magnitude of the strains which were measured.

Another indication of load redistribution in the instrumented rings can be seen in the tape extensometer measurements. Slight bending and/or shifting of the side segments is evident from the chord measurements, probably due to changes in the load distribution in the ring.

Groundwater Response

Based on the data collected during this project, it appears that the groundwater regime which exists along the project route is actually two separate hydrologic regions, one heavily influenced by the Connecticut River (Zone I) and one which is not (Zone II). The separateness of these regions or zones is seen in the fluctuations of the groundwater levels in Zone I coincident with the fluctuations of the Connecticut River level. This phenomena was not observed in the Zone II data. In addition, the Connecticut River appears to be acting as a supply reservoir for the Zone I area in that a much more rapid recovery of the piezometric levels in the rock after drawdown due to tunnel construction was observed in Zone I than in Zone II.

It was observed from the data collected during the tunnel advance that the groundwater drawdown due to tunnel construction typically occurred 300 to 500 ft ahead of the tunnel heading. The magnitude of the drawdown over the tunnel was also observed to decrease proceeding to the ground surface, similar to the drawdown adjacent to a pumped well.

Instrument Performance

In general, the instrumentation hardware utilized and monitoring procedures followed for this project performed very well. The following discussions and recommendations are made to assist in the planning of future tunnel instrumentation projects.

- Based on the data collected for this project and supported by published data from other projects, measurement of total rock movement around an underground opening excavated using techniques similar to those employed on this project can only be made with instruments installed prior to excavation. Extensometers installed from within the tunnel would have been totally ineffective in measuring rock deformation due to the tunnel operation since practically all of the movement would have occurred before the instrument could have been installed. In addition, extensometer installation from within the tunnel causes delay for the contractor and instrument damage is much more likely.

- Instruments employing electrical resistance monitoring systems, such as the rock bolt load cells used for this project, are not well suited for the adverse environment which is encountered during underground construction. The wet, oily, grimy environment makes monitoring, maintenance and repair of this type of system very difficult since clean, dry splices and connections are mandatory.

- An alternate electrical monitoring system employing vibrating wire technology performed very well in the tunnel environment. Since the parameter measured is frequency, and not resistance, measurements, modifications and repairs can be made in the field rapidly and easily because high quality splices and connections are not necessary. In addition, the vibrating wire strain gages employed on this project were easy to install, fairly robust, and the surface-mounted gages could be replaced with little difficulty if other methods of repair were not satisfactory.

- It was found to be difficult to locate the embedded strain gages at an exact preselected location within the liner segment due to slight variations in the configuration of the rebar cage and their locations in the forms. A prefabricated jig which would position the gage relative to the forms could possibly be used in the future if more accurate gage locationing is found to be necessary.

- It is recommended that piezometers installed from within the tunnel be placed in boreholes at least 3 in. in diameter to facilitate easier instrument placement and better seal installation. In addition, sustained water inflow from the borehole should be observed prior to placement of the instrument to ensure that water-bearing joints in the rock have been intercepted.

- Machine excavation methods, such as those used on this project in which extensive trailing equipment follows directly behind the heading and extends back for a substantial distance, do not allow for extensive or timely measurements of interior diameters and chord lengths. Interference from the trailing equipment on this project did not permit these measurements to be made until the tunnel heading had progressed about 300 ft past the instrumented area.

  In addition, although the initial investment of this type of monitoring equipment may usually be less than more remote systems, the additional expense incurred during monitoring due to the more time-consuming and laborious measurement more than offsets the lower initial costs for the instrument hardware over the course of a project.

- Finally, it is advisable to have some redundency in components of an instrumentation program which cannot be easily repaired or replaced during the life of the project. This would have been desirable on this project particularly with the embedded strain gages.

5. IN-SITU STRESS MEASUREMENTS BY OVERCORING

5.1 Introduction

The purpose of this section of the report is to present results of measurements of in-situ stresses by the use of the borehole deformation gage and overcoring technique in a bed of sandstone which is intersected by the Park River Auxiliary Conduit. The tests were performed 183 to 195 ft below the ground surface.

Two holes were drilled in a 3-ft sandstone bed which strikes north and dips 16 degrees to the east. The holes were drilled at 45° to the axis of the tunnel (see Fig. H-1). OC-1 at Sta 9+30 was drilled S45°W and inclined 11 degrees below the horizontal. The holes were drilled to a length of 30 ft. Each measurement consisted of deformation readings on three axes 120° apart from each other in a plane normal to the axis of the borings. Measurements were planned for four depths in each hole, but all were not possible due to rock fracture. Two measurements in Boring OC-1 gave suitable information. No further tests were possible in OC-1 since a joint dipping parallel to the axis of the borehole continued for the last 6 ft of the test hole. One test in Boring OC-2 gave suitable information out of four tests attempted. The other tests failed due to rock fracture, instrument malfunction, and a shut off of the drilling water supply.

Moduli of elasticity of the rock were measured on two annular cores from Boring OC-1 and one annular core from Boring OC-2. These moduli were used with the measured deformations and published formulae to compute the magnitude and direction of the largest and smallest principal stresses in the plane perpendicular to the axis of the boring.

The tests were carried out by Geotechnical Engineers Inc. and the drilling was performed by Guild Drilling Co., East Providence, RI.

5.2 Method of Measurement

5.2.1 General

The overcoring technique consists of three phases:

a. Measurement of borehole deformation during overcoring.

b. Determination of the modulus of elasticity of the rock, for rebound to zero stress, preferably at the point of measurement.

c. Calculation of stresses using the theory of linear elasticity and the measured deformations and moduli.

5.2.2 The Overcoring Technique

A 6-1/4-in. thin-wall core barrel was used to reach the desired depth. A 6-in. long EX core barrel, 1.5-in. O.D. was then placed on a centering device and a pilot hole drilled. A 30-in. EX barrel was then used to take 30 additional inches of core. The recovered EX core was examined to determine whether the rock was sufficiently free from defects to attempt stress measurement. In Boring OC-1 at the locations of Tests 1 and 2, the quality of the core was such that conditions were considered adequate for stress measurements. In Boring OC-2, all of the EX core was so fractured and the recovery so low that no measurement would have been attempted if the quality of the EX core alone was considered. In all of the ensuing attempts of measurement, the rock fractured. In Test 3, rock fracture damaged the borehole gage and in Test 4 malfunction of the replacement gage prevented measurement. Table 6 is a summary of the tests attempted for this study.

The borehole gage was inserted into the EX hole using setting rods. These rods were used to orient the measuring points and for measuring depths accurately. In the tests, the gage was positioned so that the measuring points were at least 9 in. beyond the end of the 6-in. overcore barrel. The drilling rig was capable of an 18-in. stroke without resetting. Where a measurement point was more than 9 in. beyond the kerf of the 6-in. overcore barrel, an initial short cut was made and the machine reset to overcore the measurement point without stopping.

Overcoring with the 6-in. thin-wall core barrel was then carried out at a rate of about 1 inch per minute. Readings of deformation on three axes 120^{o} apart in the plane perpendicular to the axis of the boring were taken continuously until the core barrel was about 9-in. below the measuring points, until the readings stopped changing rapidly, or until core breakage or other coring abnormality necessitated stoppage to prevent damage to the borehole gage.

The borehole gage used was the standard borehole gage as developed by USBM and constructed by Terrametrics, Inc. with the elongate case which provides springs at the front as well as the rear of the gage. The gage calibration was supplied by Terrametrics and confirmed by Geotechnical Engineers Inc. Readout was measured with a BLH Model 1200 Strain Indicator using a Model 1225 Switching and Balancing Unit (see Photo 11). Another Model 1200 strain Indicator with battery supply was on hand for backup use. It was used in an attempt to eliminate fluctuation of readout as observed in OC-2, Test 2. The fluctuation was not eliminated by using the independent power supply.

Figs. H-2 through H-4 are the data obtained during the overcoring operation.

5.2.3 Determination of Rock Modulus

Determination of the modulus of elasticity of the rock was made by placing the overcored section of rock within a biaxial cell (as furnished by Terrametrics, Inc.) and applying radial stresses while measuring the rock strains by means of the borehole gage placed within the core at the same point as when the in-situ overcoring took place (see Photo 12). See Figs. H-5 through H-13 for the biaxial test results. In the case of OC-2, Test 2, fracture of the rock core necessitated obtaining the modulus at a point on the core 10 inches deeper in the hole. There was no observable difference in the sandstone. The modulus was calculated using the unloading portion of the curve.

5.2.4 Calculations

The change in borehole diameter read by means of the strain indicator (plotted on Figs. H-2 through H-4 as indicator units, R) is multiplied by the calibration factor (K, in micro inches, see Table 7) to determine the diameter change, U, in the three directions of measurement. These directions are 60 degrees apart in the plane of measurement which is perpendicular to the borehole axis. The values for U_1, U_2, and U_3, and the modulus of elasticity derived from the biaxial tests on the overcored and recovered segment of rock are the basis for the determination of the maximum and minimum stresses in the plane perpendicular to the borehole.

Equations based on the analysis of Obert and Duvall Rock Mechanics and the Design of Structures in Rock, Wiley, New York, 1967, are used to determine the magnitude of the stresses and the orientation of the principal stress axis. The equations used in this report are as follows:

$$P_c = \frac{E}{6d} (U_1 + U_2 + U_3) + \frac{\sqrt{2}}{2} \{(U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_3 - U_1)^2\}^{1/2}$$

$$Q_c = \frac{E}{6d} (U_1 + U_1 + U_3) - \frac{\sqrt{2}}{2} \{(U_1 - U_2)^2 + (U_2 - U_3)^2 + (U_3 - U_1)^2\}^{1/2}$$

where P_c is the maximum normal stress (psi);

E is the modulus of elasticity (psi);

d is the "EX" hole diameter (inches); and

U_1, U_2, U_3 are measurements of diametral deformation along three axes 60 degrees apart. Deformation (inches) is positive for increasing diameter during overcoring;

Q_c is the minimum normal stress (psi).

The orientation of the principal stress axis is calculated from:

$$\theta_p = \frac{1}{2} \text{ arc tan } \frac{\sqrt{3}(U_2 - U_3)}{(2U_1 - U_2 - U_3)}$$

where θ_p is the angle from the U_1 axis (positive in a counter-clockwise direction) to the major principal stress.

The modulus of elasticity for each of the three axes is calculated individually from:

$$E = \frac{(4ab^2)(\Delta P)}{(b^2 - a^2)(\Delta U)}$$

where E is the modulus of elasticity (psi);

a is the radius of the "EX" hole (inches);

b is the radius of the core (inches);

ΔP is the change in applied pressure (psi); and

ΔU is the change in diameter during the pressure increment. ($\Delta U - \Delta R \times K$)

5.3 Conclusion

Table 8 shows the major (P_c) and minor (Q_c) stresses in the plane of measurement for each of the tests (see Fig. H-1). It also gives the bearing of the principal stress, as measured counterclockwise from vertical in the plane of measurement.

The values are lower than those determined by overcoring performed from the surface in a prior testing program in which the average value was 452 psi. The lower values may result in part from measurements at less than one diameter from the tunnel. The different values measured may also reflect the effect of the joint system and adjacent blasting in the tunnel. (A nearly vertical joint with horizontal striations was found between the sites of the two tests in OC-1 and water began to flow from the borehole at about 1 gal per minute before Test 2 was made.)

6. PHOTOGRAPHIC GEOLOGIC DOCUMENTATION

6.1 General

As described in Section 3.2, the excavation of the Park River Auxiliary Conduit employed the use of a fully shielded tunnel boring machine in whose shielded tail precast concrete tunnel liner rings were erected before the tail shield was advanced. As a result, the excavated bedrock surface could only be observed along the invert of the tunnel prior to the installation of the invert segment for each precast liner ring. At best, this would permit a geologist or engineer, who would need to be present at the tail of the TBM at all times during tunnel advance, to briefly view the muck coated rock surface at the invert and perhaps sketch or photograph any observable features before an invert segment were placed. Therefore, at the start of the project is was decided by the project team to design and operate an automatic system to photographically record a small portion of the geology which was encountered along the tunnel route, near the tunnel springline.

The following sections of this report describe the system that was designed and used and the operation and maintenance procedures which were performed, discuss the results which were obtained with reference to the photographs presented in Appendix I, and present conclusions and recommendations concerning the use of similar systems in the future.

6.2 Design and Installation

The photographic system which was used was composed of two separate but dependent systems: the protective and external power system and the actual photographic equipment which recorded an image of the exposed rock surface.

The protective housing and power system was designed and built by the Robbins Co. in conjunction with GEI. The protective housing basically consisted of a two-section, steel box about 24-in.-high x 35-in.-wide x 40-in.-long fabricated of 1/4-in.-thick steel plate. The housing was welded to the inside skin of the TBM near the front right side of the TBM at approximately the tunnel springline, see Fig. 33. An approximate 23-in. x 34-in. hole was cut in the shield prior to the housing installation to provide visual access to the rock surface.

The front portion of the housing (see Fig. 34 for a cross-sectional sketch of the camera housing) included a hydraulically operated steel door in front of the opening in the shield to keep the inside of the housing free of TBM cuttings when photographs are not being taken. Water spray jets were mounted on top of the housing in front of the door to wash the rock surface just prior to obtaining a photograph. An access door was also present to permit periodic clean out of any muck which may have entered when the protective door was open.

The rear portion of the housing was sealed from the front portion and the surrounding environment so that it was essentially dust and moisture proof. This is where the photographic equipment was located. The front face of the rear housing was plate glass through which the photographs were taken. A locking trap door permitted access to the photographic equipment. The camera was mounted in the rear portion of the housing using rubber isolation mounts to limit the transmission of TBM vibrations to the rather delicate camera equipment.

The photographic equipment which was used for this system included:

1 NIKON F2 camera body

1 NIKKOR 28 mm, f 3.5 lens

1 NIKON M2 motor drive

1 NIKON MF-1 250 Frame, film magazine back

1 NIKON MA-4 AC/DC Converter

2 SUNPAK 411 variable power flash units with AC adaptors

The film utilized for the entire project was Kodak Vericolor II 5025, ASA 100, color print negative film which was obtained in bulk and then loaded into the 250 frame film cassets in a dark room.

The advance of the TBM was monitored by a telescoping proximity switch with a six-foot stroke. As the TBM advanced every two feet, the camera system would be triggered by the proximity switch and a photograph would be taken. The field of view of the camera was about 2½-ft-wide x 2-ft-high so that there would be some overlap along the axis of the tunnel and a continuous photographic strip would be obtained. The sequence of operation of the photographic system proceeded as follows:

1. After the TBM had advanced 2 ft, the water jet spray would be activated for 10 seconds to clean the rock face and front of the housing door.

2. After 10 seconds, the spray would shut off and the steel door would open.

3. Seven seconds later the camera and flash system would be activated and a photograph obtained.

4. After the photograph was obtained, the steel door would close and the film would advance.

5. After the TBM advanced another 2 ft, the procedure would be repeated.

6.3 Operation and Maintenance

In general, the camera system performed well, considering the relatively hazardous environment for this type of equipment and the fact that the camera was checked on the order of only two to three times a week. The operation of the system involved three basic activities: (1) reference exposure stationing, (2) film changing, and (3) maintenance.

Reference exposures were established by means of taking several exposures per roll with the lens cap in place at a known station. The station was determined by summing three dimensions obtained prior to the reference exposure. The three dimensions, recorded to the nearest tenth of a foot were (1) segment number (from which the station of the leading edge was calculated), (2) segment to TBM (leading edge of segment to rear of TBM), and (3) stroke dimension (accounting for variable extended length of the TBM), see Fig. 33. Stationing of the leading edge of the segment was corrected every 100 segments from data provided by Roger J. Au & Son, Inc. Reference exposures were taken on an alternating sequence of one and two exposures to reduce the ambiguity of the station corresponding to the reference exposure.

The method employed to assign stations to bedrock exposures between the reference exposures was basically a process of elimination. Reference exposures were identified by roll number, date and known station on each contact sheet. Duplicates and overlaps were then distinguished and so identified. Stations were then assigned (to the nearest foot) to the bedrock exposures using the following incremental increases: two feet for each valid exposure, one foot for each overlapped exposure, and zero feet for each duplicate exposure.

Film changing generally occurred as often as necessary and averaged twice per month. Initially the camera was dismounted from the housing, brought to the ground surface, reloaded, and then remounted back in the housing. This procedure was abandoned because it proved to be time consuming and posed a greater threat of damage to the camera during handling. Subsequently, the film was loaded with the camera mounted in the housing during TBM operations. This scheme, while more difficult to execute, exposed the camera to less dust and water than the previous scheme. After the film was reloaded, a series of exposures were taken to advance the exposed film onto the winding cassette and ensure proper advancement of the film. Occasionally the film would fail to advance properly at the beginning of the roll and some photographic coverage would be lost until the problem was corrected. While the magazine back was opened for film changing, the interior of the camera was inspected for any dust or moisture accumulation.

The maintenance performed on the system during operation primarily involved cleaning the glass window in front of the lens and changing the dessicant in the housing. The glass window separating the front and rear portions of the housing was subject to moisture from the water spray and dust generated during TBM operations. Each time a reference exposure was taken the window would be inspected for any dirt accumulation and cleaned if necessary. The window was cleaned with paper towels from the access opening in the front portion of the housing. Hydraulic pressure to the cylinder operating the outside door was secured by a needle valve to prevent injury from the door opening while cleaning the window. The needle valve also served to regulate the rate at which the door opened and closed during normal operation. Occasionally, TBM cuttings had to be removed from the front portion of the housing.

Three one-pound sacks of silica gel were used as a dessicant in the housing. The sacks were placed loosely in the rear portion of the housing, located generally at opposite ends to absorb any moisture present. There were two sets of three used, one set in the housing and one set drying in the oven for re-use. The dessicant was changed approximately once a month, more often if moisture began to accumulate within the housing. Most of the moisture present within the housing was due to perspiration of the operator during maintenance in the relatively humid environment.

In late December 1979 tunnel advancement ceased for the holidays. This corresponded to approximately Sta 30+00. All of the photographic equipment was removed and professionally cleaned and checked. Following the cleaning, the camera equipment was reinstalled and photographing resumed as tunnel advance continued. The photographic system continued operating until about Sta 96+55 when clay from the large fault zone in this area squeezed past the protective steel door and shattered the glass window at the rear portion of the housing.

During the TBM advance from the discharge to inlet shaft, approximately 23 rolls of 250 exposure film were used for the geologic documentation.

6.4 Results and Interpretation

Overall, the automatic photographic system resulted in approximately 75% coverage of the exposed bedrock surface of the tunnel springline in the nearly 8800 ft excavated by the TBM. Approximately 5% of these exposures were taken with the glass window between the camera and tunnel wall fogged or otherwise not clear enough to distinguish many features of the bedrock. Generally, the interpretations of the exposures can be subdivided into three categories: (1) those exhibiting geologic features, (2) those exhibiting excavation features (inherent in the method of construction) and (3) those exhibiting functional features (characteristic of the system). Dip angles were measured with respect to the frame of the film assumed horizontal. The photographs referred to in the following discussions are presented in Appendix I along with brief interpretive captions for each photograph.

Geologic Features

In the first category, features such as fractures, jointing, bedding and certain lithology can be distinguished. Photo I-1 taken at Sta 96+01 is an example of fractures which appear near the fault zone encountered at Sta 96+65. This exposure appears to be gray shale, fractured, with fines present on the brecciated surfaces. There is no readily apparent dip to the bedding evident in this exposure. Another exposure which appears to have been taken near a fracture zone is Photo I-2. This exposure was obtained at Sta 91+32 and appears to be calcite-healed breccia in contact with gray sandstone at an apparent dip of 19° to the east. Two parallel calcite-healed joint sets appear, one set approximately perpendicular to bedding with an apparent dip of 71° to the west, the other approximately 25° to bedding with an apparent dip of 43° to the east. These parallel sets are interpreted as short tension joints resulting from fault movements. An additional example of jointing appears in Photo I-3. This exposure was taken at Sta 38+72 and appears to be an interbedded sandstone in contact with red shale at an apparent dip of 14° to the east. At the upper left and right center of Photo I-3, cross bedding jointing features are evident at apparent dips of 82° and 86° to the west, which is approximately perpendicular to bedding.

Bedding features are among the most common encountered in the exposures and are more readily distinguished where contrasting colors exist in the bedding. For example, Photo I-4, taken at Sta 65+66, appears to be an interbedded sandstone/gray shale in contact with gray shale. Bedding has an apparent dip 15° to 18° to the east and exhibits interbeds of contrasting color. However, in Photo I-3 the bedding of the red shale would be difficult to determine if it were not in contact with the sandstone. In Photo I-4 the third and fifth water nozzles are spraying water but the bedding structure is still evident.

Distinguishing lithology in the exposures can be very difficult, particularly if no samples were obtained at or near the exposure location to verify interpretations. For example, Photo I-5, taken at Sta 62+01, appears to be maroon shale with calcite stringers parallel to the bedding. This observation agrees with rock core samples obtained from MPBX-8 at Sta 61+19. On Photo I-5, the three calcite stringers which are evident have apparent dips of 19°, 11° and 7° from top to bottom, respectively. On the other hand, Photo I-6, taken at Sta 53+43, could be interpreted as red shale or red sandstone. No sampling was performed at or near Sta 53+43; hence, it is more difficult to verify the lithology. Structural features, however, can be measured. The apparent dip of the calcite stringer is 19° to the east. The dark spots which appear generally to lower left are an accumulation of dirt on the glass between the camera and tunnel wall.

The sedimentary structure of the exposed bedrock in Photo I-7, taken at Sta 21+66, is a useful characteristic by which to visually identify the geologic origin of the lithology. In contrast, Photo I-8 (Sta 86+76) has few structural characteristics evident by which the geologic origin might be interpreted. Rock core sampling performed at boring FD-14T provided samples identified as basalt. Based upon the visual appearance alone, Photo I-8, without the sample information, could be falsely identified as gray shale.

Excavation Features

The second subdivision in the interpretation of the exposures can be generally described as those exposures exhibiting features inherent in the method of excavation. As expected and observed in unlined portions of the TBM advanced tunnel, near-vertical striations appeared, some more evident than others, in a good portion of the exposures. These near-vertical striations are due to the gage cutter (outermost on cutterhead) shearing into the rock and leaving behind the imprint of its path (see Photo I-9). Photo I-9, taken at Sta 23+36, appears to be of red shale with bedding with an apparent dip of 14° to the east. The distance between the near-vertical striations indicates the forward advancement in one revolution of the entire cutterhead.

The next most notable characteristic intrinsic to the system was the result of not fully extending the TBM to its six-foot stroke. The proximity switch was based upon the assumption that three exposures would be taken and a full six-foot stroke would be completed before regripping. Any stroke short of six feet would fail to trigger the system at the last two-foot interval. After regripping, the proximity switch would trigger the system at less than two feet from the exposure taken prior to regripping. The result of this premature regrip was that frequent overlaps and duplicates would appear on the contact prints (see Photo I-14A). During the stationing process these duplicates and overlaps were taken into consideration in order to assign the correct station to each exposure.

To further exemplify features commonly appearing in the exposures which were due to TBM tunneling see Photo I-10. The lighter area in the center of the photo appears to be muck pressed against the tunnel wall as a result of the forward shield dragging muck between the exposed bedrock and the shield. To the left of the smeared muck there are several cutterhead striations visible. The rock appears to be red shale with no structural features evident. There is some accumulation of dirt on the window between the camera and tunnel wall.

Another feature which appeared to be due in part to the TBM operation and in part to the rock excavated, was overbreak. Although appearing rather infrequently and to variable extent, Photo I-11 taken at Sta 15+62 is an example of what overbreak features were recorded by the photo documentation. To the right of the photo is the intact bedrock surface as excavated by the TBM. To the left is an overbreak approximately 18-in.-long, 9-in.-wide, and perhaps 6-in.-deep occurring in red shale. The overbreak has an apparent dip of 12° to the east. To the upper left muck appears to have been smeared by the front shield against the bedrock surface.

Functional Features

The third subdivision in the interpretation of the exposures can be described as features associated with the system used, or functional features. The most notable feature appearing in the exposures was the water spray (see Photo I-12). Attempts were made to correct this problem which only resulted in completely terminating the water spray. This was undesirable because the detail concealed by the water spray is by far less than the detail concealed by the muck. For instance, in Photo I-12, taken at Sta 72+44, the third and fifth water jets are spraying the lower portion of the exposed bedrock surface under each nozzle and what appears to be a bed of sandstone is exposed. However, the left and upper portions of exposed bedrock appear to be coated with a thin film of muck which concealed the detail underneath. The near-vertical shaded feature just to the left of the center of the exposure is a streak of dirt on the window between the camera and tunnel wall. The exposed bedrock which has been sprayed appears to be an interbedded sandstone with an apparent dip of 16° to the east with calcite-healed cross bedding fractures at an apparent dip of 69° to the west in contact with what appears to be red shale.

Another characteristic contained in the exposures related to the system used was condensation on the window between the exposed bedrock and the camera, see Photo I-13. It is uncertain whether this moisture was groundwater occasionally present and leaking into the forward portion of the housing or due entirely to the water spray. The exposure appears distorted or out of focus but certain features such as the apparent dip of the bedding are evident. The rock appears to be interbedded with an apparent dip of 15° to 17° to the east; however, the rock type cannot be distinguished.

An additional characteristic related to the system was exposures of the hydraulically operated door protecting the camera between exposures. In two separate instances the mount providing the reaction for the cylinder which opens the door (see Fig. 34) failed for unknown reasons and resulted in a series of exposures of the door (see Photo I-14B). The first occurrence went undetected for approximately one week and was repaired shortly thereafter. In the process of repairing the mount, the hydraulic hoses were disconnected from the cylinder and later connected in reverse order resulting in the door remaining open between exposures and closing immediately before an exposure was taken. This error produced a series of exposures of the door and was detected and repaired in approximately one week. Another circumstance involving an electrical malfunction also produced a series of exposures of the door. When the contact sheets for the last roll of film were examined, an interesting occurrence was observed (see Photo I-14C). Apparently, when the TBM encountered the fault zone at approximately Sta 96+64 (heading station), the thrusting force of the TBM extruded some of the clay around the cutterhead and against the protective door. The clay then pushed the door down and fractured the glass between the camera and tunnel wall.

Of the nearly 8800 ft of TBM advanced tunnel, approximately 25% of the coverage is not useful (repeat photographs of door) or was otherwise not obtained for any of the following reasons:

1. Occasionally after reloading the film and advancing exposed film into the winding cassette, the film would jam in the camera and remain undetected until the next reference exposure was taken (usually the next day), thereby losing coverage of any advancement which had taken place in the interim.

2. Occasionally too many exposures were taken at a given interval resulting in the film running out before reloading.

3. The flash units not fully synchronized with shutter thereby producing a dark exposure. This was due to the synchronization plug from the flash to the camera loosening from the camera because of the extreme vibrations transmitted from the cutterhead.

4. Difficulty remounting the camera after reloading.

5. Approximately 300 ft at the start of the TBM tunnel was not taken due to installation delays and debugging of the system.

6.5 Recommendations and Conclusions

Based upon viewing approximately 4500 contact prints, most of which were of the exposed bedrock surface at the springline as excavated by the TBM, the following conclusions can be made in regard to the geologic photo documentation system used for this project:

1. Automated photo documentation is better suited toward identifying structural characteristics of exposed bedrock rather than identifying specific or even general lithology.

2. Certain structure can be distinguished and apparent dip angles of joints, joint sets, fractures and faults can be measured.

3. The color of the exposed bedrock can be misleading and is particularly vulnerable to inconsistencies in development processes.

4. Water spray jets are an effective means to rid the exposed bedrock surface of fines which may conceal structural features.

5. The use of photo documentation to record geologic features encountered during advancement of machine bored tunnels is feasible.

The following recommendations concerning the use of an automated photo documentation system to record bedrock units can be made based upon experience gained in the process of executing this project:

1. A combination of sampling and photos of exposed bedrock should be used to document lithological units.

2. Any protective device (door) should be designed to, or be constructed of materials which would permit photos to be taken even when in place.

3. Stationing systems should be designed into shutter actuating mechanisms, perhaps by means of a wheel in contact with the bedrock surface calibrated to actuate the system and provide a record of the actual advancement which has taken place.

4. In order to view from left to right (i.e., in the direction of tunnel advance) the finished contact prints of continuous photographic coverage, the camera must be situated such that the film travels through the camera in the same relative direction as advancement. This would have been accomplished if it had been possible to mount the camera on the other side of the TBM as originally intended.

5. Water spray nozzles should be designed to wash the entire surface of exposed bedrock to be photographed.

6. The system should be as isolated as possible in terms of dependence upon the TBM for sources of water, hydraulic pressure, etc.

TABLES

TABLE 1 - INSTRUMENTATION SUMMARY
PARK RIVER AUXILIARY TUNNEL

Test Section No.	Station	Instruments Used
1	8+20	1 - MPBX 12 - Rock Bolt Load Cells
2	11+32	1 - MPBX 3 - Surface Vibrating Wire Strain Gages 4 - Embedded Vibrating Wire Strain Gages 1 - Vibrating Wire Piezometer 4 - Tape Extensometer Points
3	15+25	Same as 2
4	24+00	Same as 2
5	27+00	Same as 2
6	43+50	Same as 2
7	58+55	Same as 2
8	61+19	Same as 2
9	91+00	Same as 2
10	95+54	Same as 2 except for 4 - Surface Vibrating Wire Strain Gages

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TABLE 2 - STRAIN GAGE LOCATIONS (1)
PARK RIVER AUXILIARY TUNNEL

Ring No.	Leading Edge Station	Instrument Station	℄A* (Seg. #4, Crown)	℄B* (Seg. #2, Right)	℄C* (Seg. #3, Left)	HI*	Invert Elevation
15	11+31.37	11+29.08	1°-50'RT	70°-24'RT	66°-37'LT	6.42'	-147.20
81	15+31.09	15+28.28	1°-10'RT	64°-37'RT	67°-39'LT	5.15'	-144.60
225	24+02.00	23+99.71	0°-57'RT	63°-34'RT	64°-13'LT	5.34'	-138.90
275	27+04.14	27+01.85	0°-45'RT	65°-21'RT	65°-42'LT	5.14'	-137.01
548	43+53.83	43+51.54	0°-52'RT	65°-36'RT	60°-48'LT	5.20'	-126.27
797	58+59.07	58+56.78	1°-10'LT	64°-13'RT	60°-56'LT	5.11'	-117.69
840	61+19.01	61+16.72	0°-20'RT	65°-07'RT	60°-51'LT	5.20'	-116.11
1324	91+02.98	91+00.69	2°-05'RT	63°-13'RT	60°-05'LT	5.08'	- 98.91
1399	95+57.47	95+55.18	1°-37'RT	74°-28'RT	69°-16'LT	7.31'	- 96.30

*Refer to Fig. T-1 for angle and dimension locations. Data provided by Roger J. Au & Son, Inc.
(1) All invert embedded gages located 1.2' left of center of segment, 3.3' from leading edge.

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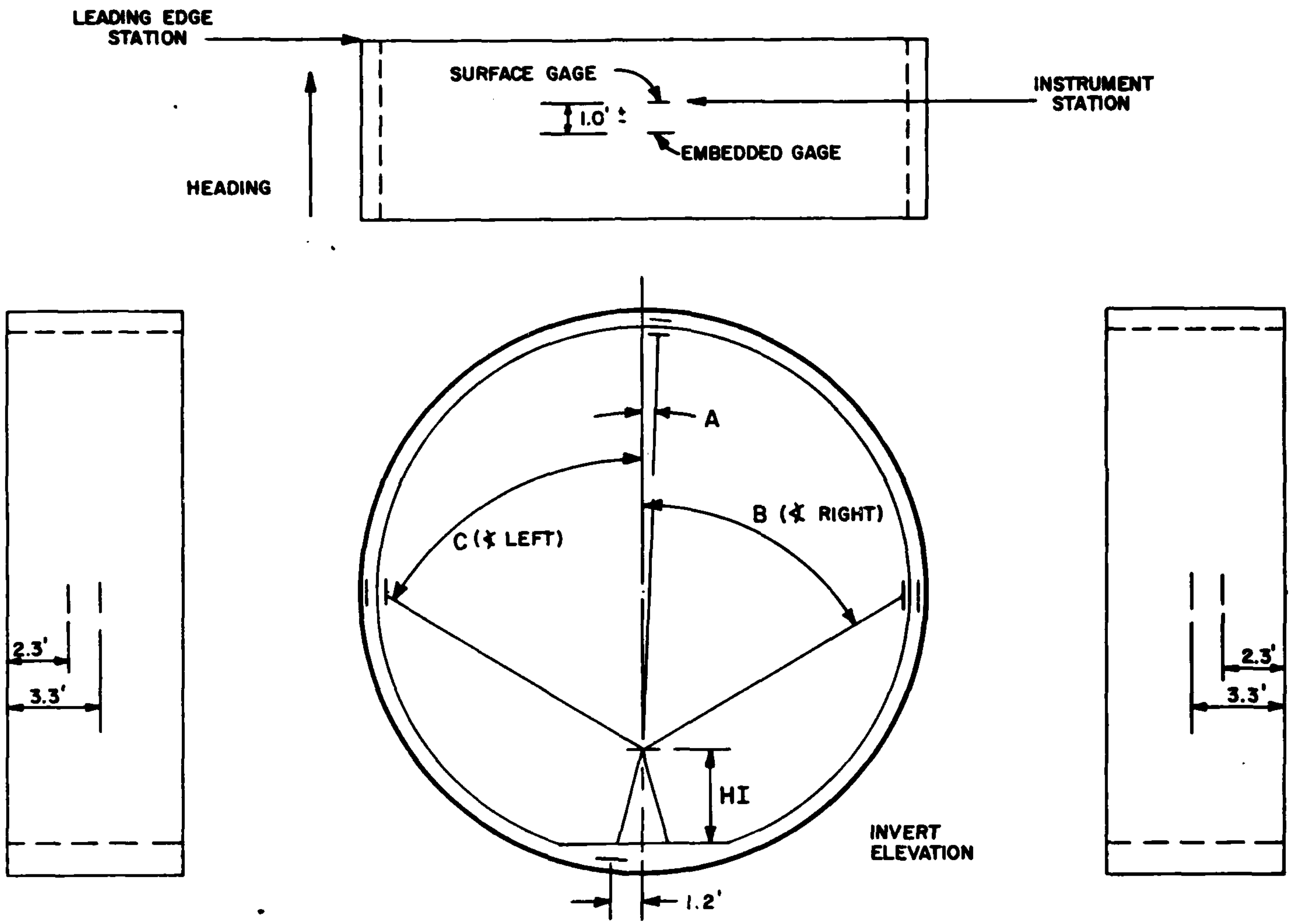

NOTES

NOT TO SCALE

REFER TO TABLE 2 FOR ANGLES, DIMENSIONS & STATIONS OF EACH INSTRUMENTED RING.

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	STRAIN GAGE LOCI
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. T-1

TABLE 3 - MAXIMUM MEASURED MPBX MOVEMENT
PARK RIVER AUXILIARY TUNNEL

MPBX No.	Maximum Downward Movement					
	Anchor 1	Anchor 2	Anchor 3	Anchor 4	Anchor 5	Anchor 6
1 (TS-1)[1)]	0.029	0.026	0.019	0.016	0.005	-
2 (TS-2)[1)]	0.262	0.234	0.208	0.121	0.022	-
3 (TS-3)[1)]	0.162	0.163	0.146	0.132	0.018	-
4 (TS-4)[2)]	0.065	0.031	0.025	0.018	0	0
5 (TS-5)[2)]	-0.033	-0.033	-0.025	-0.025	-0.015	0.010
6 (TS-6)[2)]	0.067	0.001	0.025	0.023	0.015	0.005
7 (TS-7)[2)]	0.180	0.117	0.071	0.077	0.043	0.013
8 (TS-8)[2,3)]	-0.015	-0.013	-0.004	0.011	0.013	-0.006
9 (TS-9)[2)]	0.207	0.179	0.008	-0.004	-0.018	-0.014
10 (TS-10)[2)]	0.624	0.281	0.182	0.065	0.042	0.019

NOTES:
1) Relative to Anchor #6
2) Relative to Reference Plate
3) Installation completed and initial reading obtained after passage of tunnel heading

__Anchor Locations Above Crown__

Anchor #1:	2.0 to 3.7 ft	Anchor #4:	10.9 to 15.7'
Anchor #2:	3.9 to 5.3 ft	Anchor #5:	19.8 to 26.7'
Anchor #3:	6.8 to 9.0 ft	Anchor #6:	46.7 to 61.6'

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TABLE 4 - MAXIMUM MEASURED STRAIN (μ IN./IN.)[5]
PARK RIVER AUXILIARY TUNNEL

Ring No.	Strain Gage Designation[1]						
	1E	2S	2E	3S	3E	4S	4E
15 (TS-2)	-19 +52	+157	-57 +41	-126 + 18	- 66 + 23	+538	- 26 + 54
81 (TS-3)	+109	- 45 + 31	+237	- 37 + 75	+155	- 13 +168	+ 87[2]
225 (TS-4)	+ 28	- 50	- 3 + 22	- 53 +136	- 13 + 18	-[4]	- 5 + 54
275 (TS-5)	-351	- 26 + 53	- 29	- 31 + 24	- 32	+129	+ 44 (+229)[3]
548 (TS-6)	- 68 + 34	- 42 + 34	-[4]	+ 52	+105	+514	+165
797 (TS-7)	- 38 + 91	- 56 + 50	+ 78	- 14 +162	- 4 + 61	+152	+ 95
840 (TS-8)	- 6 + 14	+164	- 49 + 5	+131	+ 23	+ 99	+ 43
1324 (TS-9)	-142 + 56	- 66	- 31 + 9	- 59	- 51	+ 90	- 37
1399 (TS-10)	-429	- 43 + 10	- 18 + 16	- 5	- 22	+1609	-[4]

NOTES: 1) Gages designated as follows: 1 - Invert; 2 - Right; 3 - Left; 4 - Crown; S - Surface; E - Embedded; (2E - Embedded gage, right segment).
2) Gage damaged, not monitored for complete period.
3) Questionable reading.
4) Gage damaged.
5) Approximate equivalence 4 psi/microstrain

+ Compression
- Tension

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Revised November 1980

TABLE 5 - CONCRETE MODULUS TEST SUMMARY PARK RIVER AUXILIARY TUNNEL

Test Date	Cylinder Designation	Corresponding Ring Segment[1)]	Modulus of Elasticity (psi)	Compressive Strength (psi)
9-17-79	I	15-2, 15-4	3.7×10^6	7630
	II	15-1, 15-3	3.8×10^6	7990
10-19-79	I	81-2, 81-4	4.0×10^6	7780
	II	81-1, 81-3	4.0×10^6	7730
12-13-79	I	225-2, 225-4, 275-2, 275-4	4.1×10^6	8200
	II	225-1, 225-3, 275-1, 175-3	4.0×10^6	8480
3-7-80	III	548-2, 548-4	3.8×10^6	8550
	IV	548-1, 548-3	3.9×10^6	8480
4-11-80	III	797-2, 797-4, 840-2, 840-4	3.4×10^6	8210
	IV	797-1, 797-3, 840-1, 840-3	4.0×10^6	7470
7-25-80	III	1324-2, 1324-4, 1339-2, 1399-4	4.5×10^6	9240
	IV	1324-1, 1324-3, 1399-1, 1399-3	3.9×10^6	8520

NOTES: 1) 81-2 signifies Ring 81, Segment 2

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TABLE 6 - SUMMARY OF OVERCORING TESTS ATTEMPTED
PARK RIVER AUXILIARY TUNNEL

Boring	Test No.	Depth of Measurement ft	in.	Length of Overcore in.	Comments
OC-1	1	14	4	17	Successful test
OC-1	2	21	0	18	Successful test
OC-2	1	18	11	11	Rock fractured; gage malfunctioned
OC-2	2	23	1	10	Test successful to moment of rock fracture.
OC-2	3	28	0	6	Water supply interrupted, bit jammed and rock fractured
OC-2	4	29	0	0	Gage malfuntioned

TABLE 7 - CALIBRATION OF BOREHOLE DEFORMATION GAGE PARK RIVER AUXILIARY TUNNEL

Gage No.	Gage Factor	$K_1 \times 10^{-6}$ in.	$K_2 \times 10^{-6}$ in.	$K_3 \times 10^{-6}$ in.
40	1.00	2.55	2.64	2.70

TABLE 8 - OVERCORING TEST RESULTS

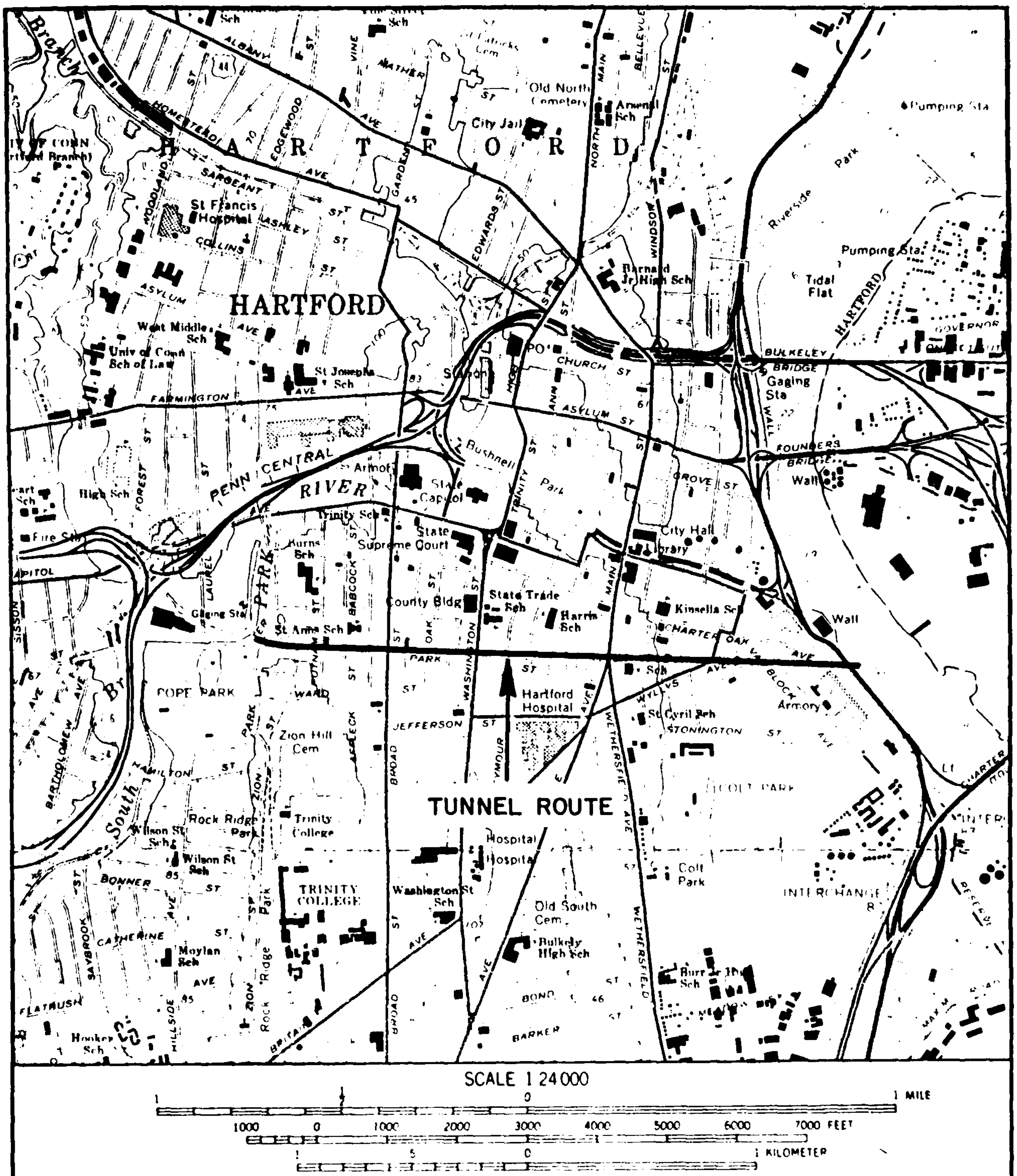
PARK RIVER AUXILIARY TUNNEL

Boring	Test No.	Depth from Ground Surface ft	Distance from Tunnel Wall ft	in.	Dia. Deformation x 10^{-6} in. U_1	U_2	U_3	Average Elastic Modulus x 10^6 psi	Maximum Prin-cipal Stress P_c psi	Minimum Prin-cipal Stress Q_c psi	Bearing of P_c Counterclockwise from Vertical* degrees
OC-1	1	184	14	4	176	396	270	2.91	334	210	17
OC-1	2	183	21	0	405	150	-205	4.02	236	-81	73
OC-2	2	195	23	1	219	660	-554	1.97**	304	-162	41

*The bearing of P_c is in a plane perpendicular to the axis of the boring, counterclockwise as observed from the tunnel.

**Due to fracture of the rock, the elastic modulus was measured on a sample from 10 inches beyond the point of overcore relief measurement.

FIGURES



Plan prepared from U.S.G.S. quadrangle maps Hartford North, Conn and Hartford South, Conn., both photo revised 1972.

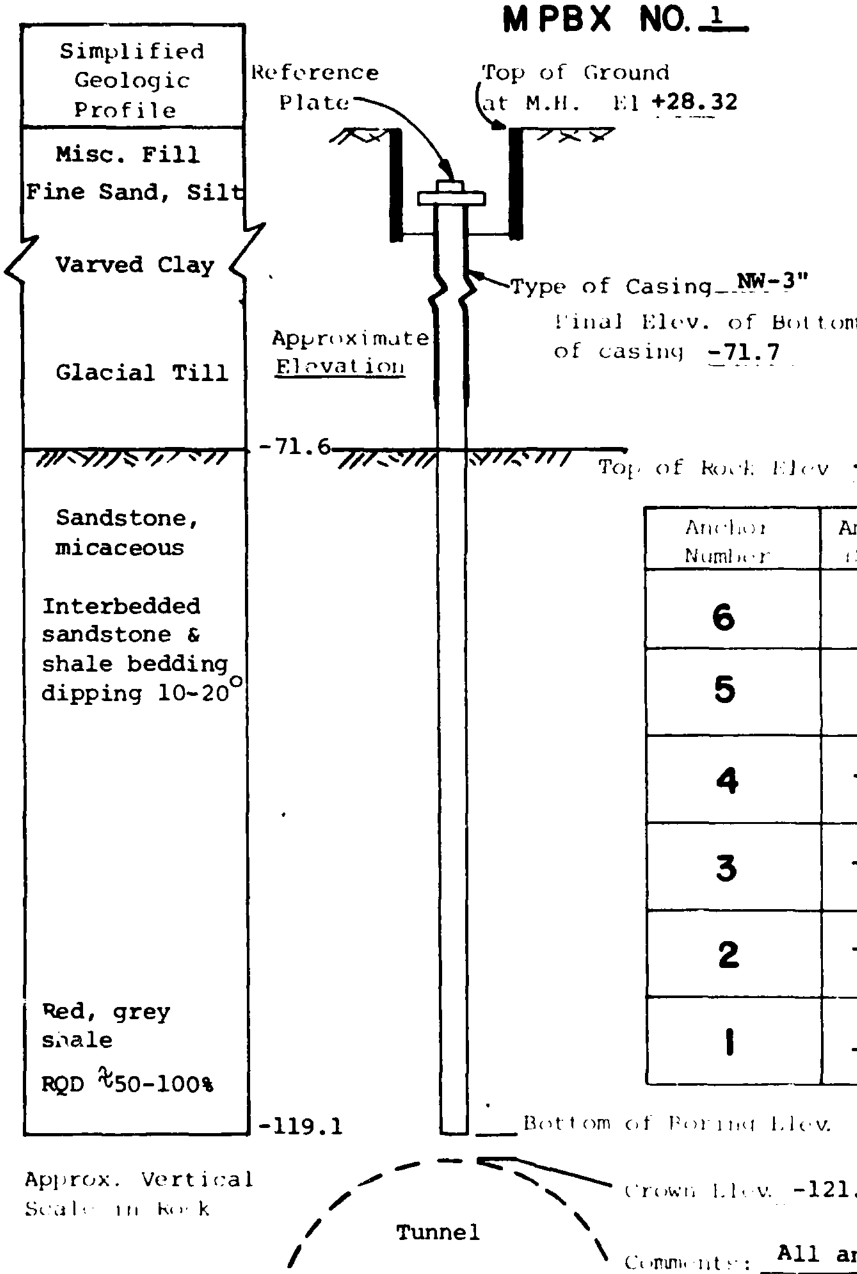
Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	PROJECT LOCATION PLAN
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project [illegible]	August 1989 Fig. [illegible]

SEE VOLUME I PLATE NO. 22

FIGURE 2

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 1

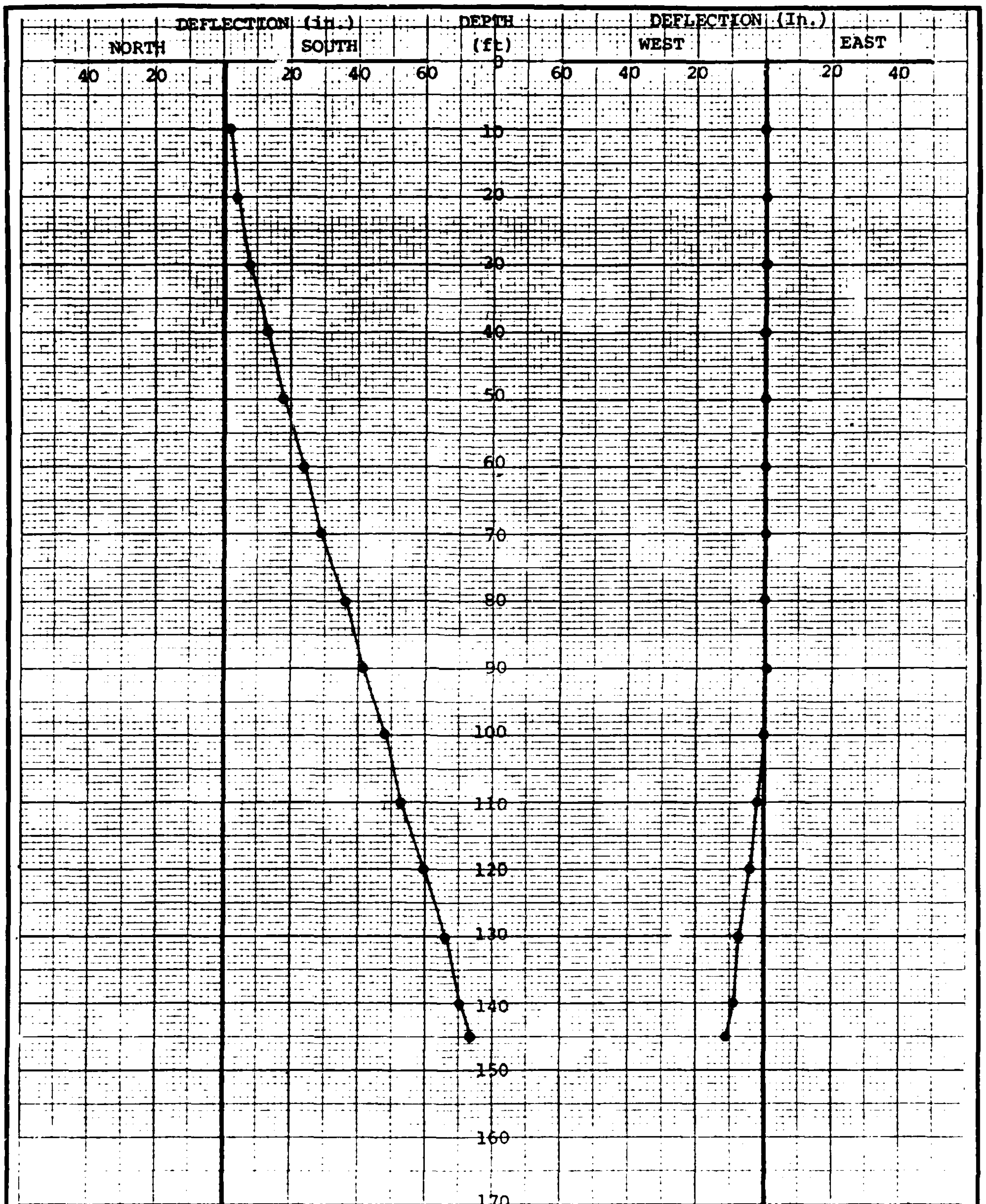


Reference Plate Data	
Date	10/20/78
Sta	8+20
Offset	Centerline
Elev	28.031

Anchor Number	Anchor Tip Elevation	Distance Above Crown
6	-74.7	46.7'
5	-97.6	23.8'
4	-108.5	12.9'
3	-112.4	9.0'
2	-116.6	4.8'
1	-118.6	2.8'

Comments: All anchor elevations assume a vertical borehole

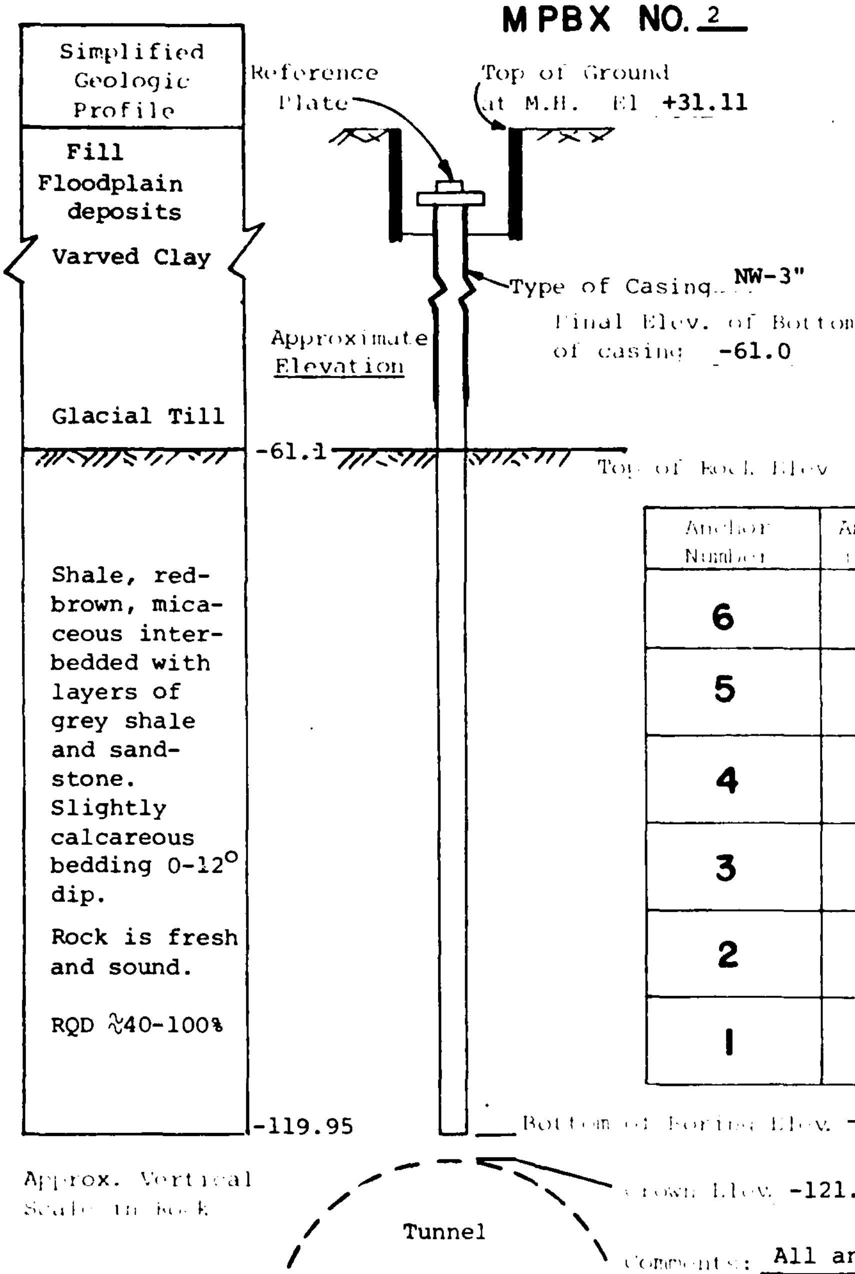
Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX NO. 1
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 3



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX- 1 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 4

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 2

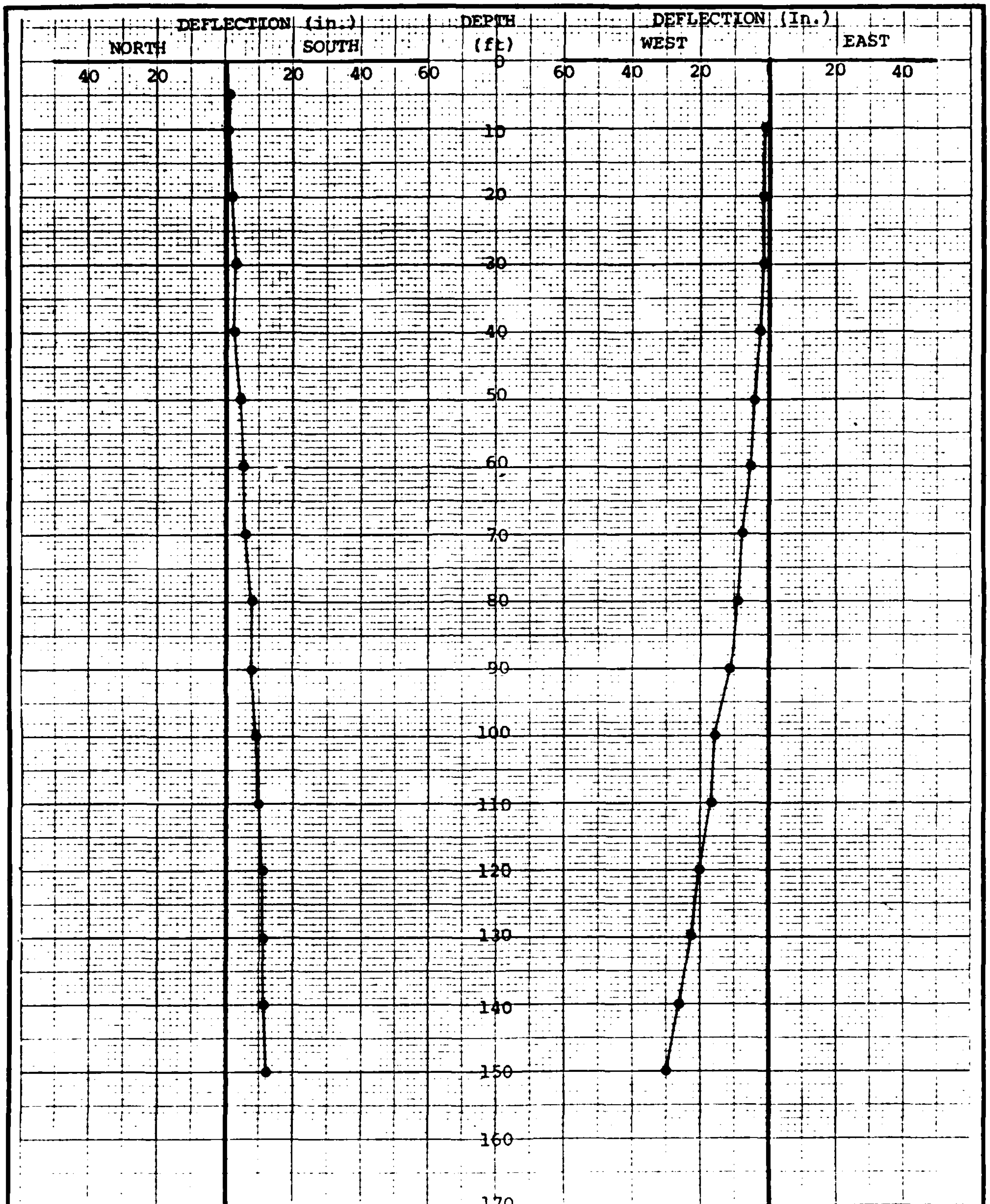


Reference Plate Data	
Date	6-7-79
Sta	11+32
Offset	Centerline
Elev	30.415'

Anchor Number	Anchor Tip Elevation	Distance Above Crown
6	-70.2	51.8'
5	-102.2	19.8'
4	-108.6	13.4'
3	-114.6	7.4'
2	-117.6	4.4'
1	-119.6	2.4'

Comments: All anchor elevations assume a vertical borehole.

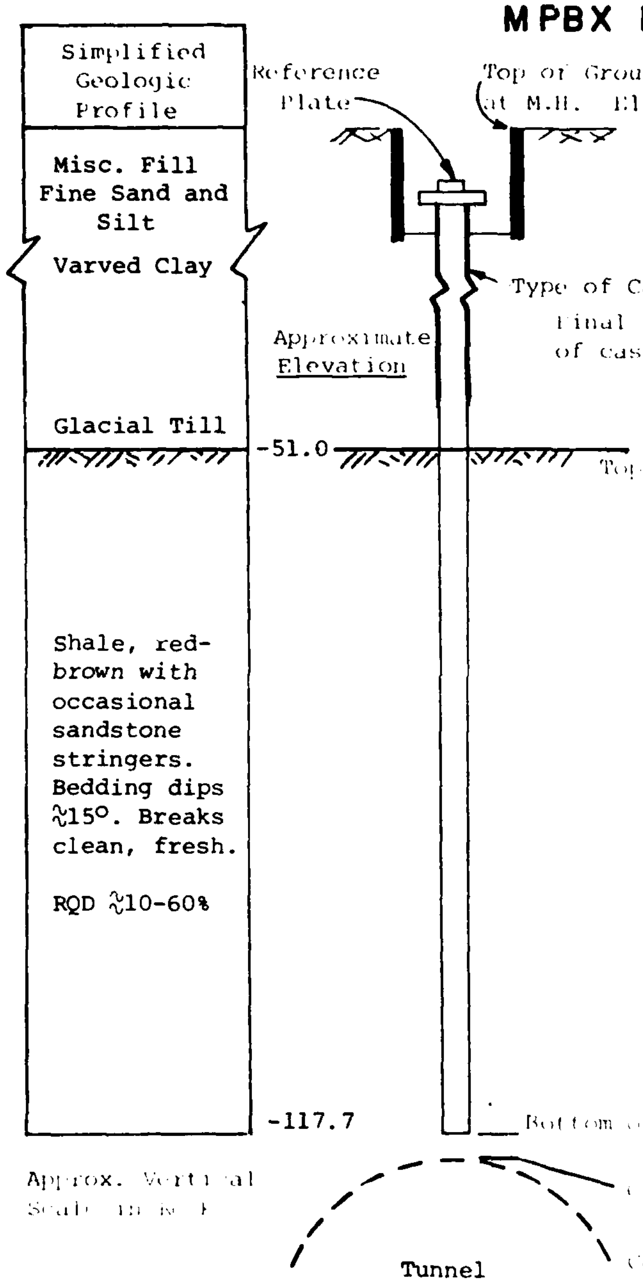
Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX NO. 2
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 5



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX- 2 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig.6

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 3

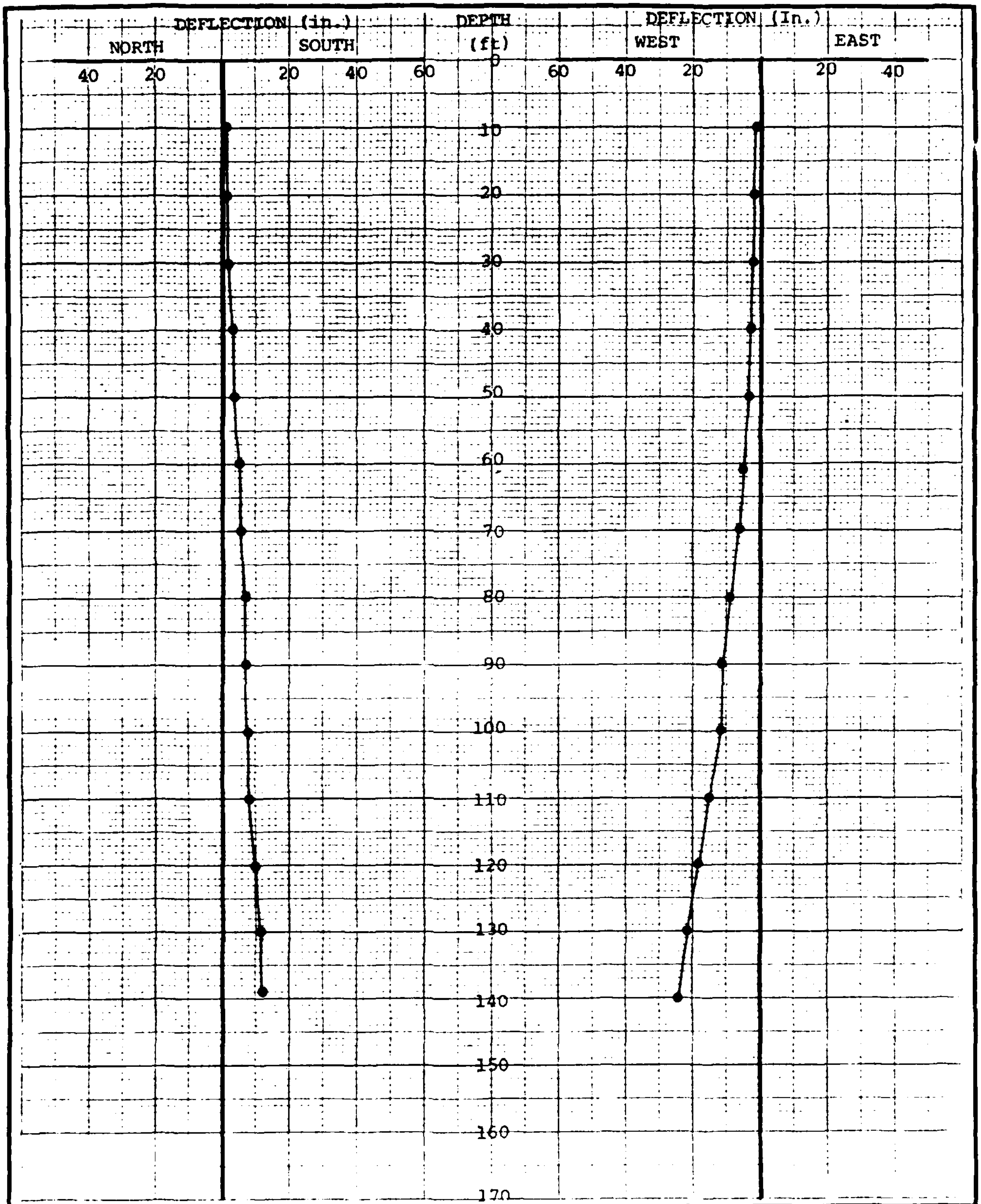


Reference Plate Data	
Date	9/27/79
Sta	15+25
Offset	Centerline
Elev	20.227'

Anchor Number	Anchor Tip Elevation	[illegible]
6	-72.0	47.7'
5	-93.0	26.7'
4	-108.0	11.7'
3	-112.9	6.8'
2	-115.8	3.9'
1	-117.3	2.4'

Comment: All anchor elevations assume a vertical borehole.

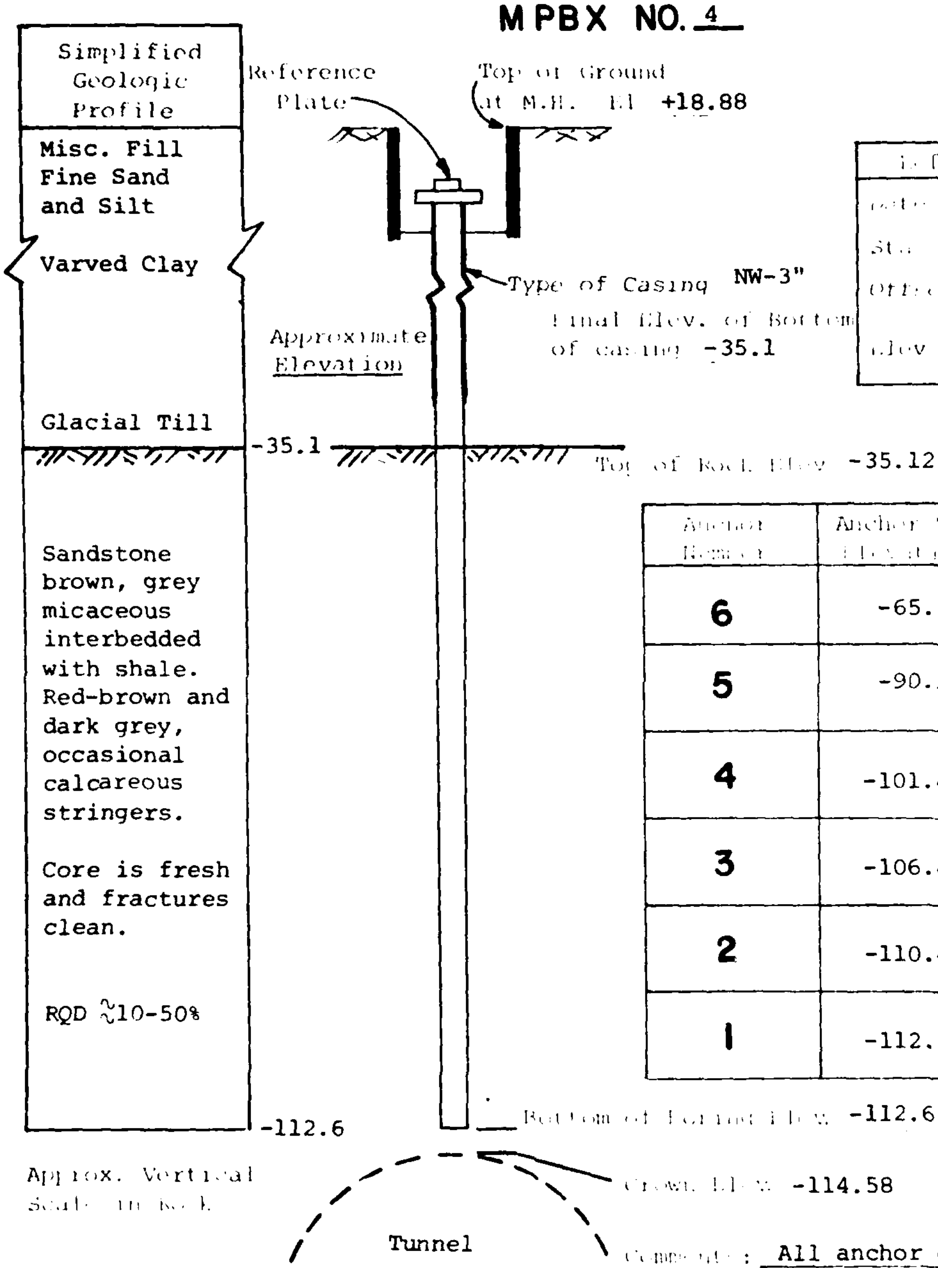
Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	Multiple Position Borehole Extensometer Installation Data MPBX No. 3
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 7



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX-3 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 8

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 4

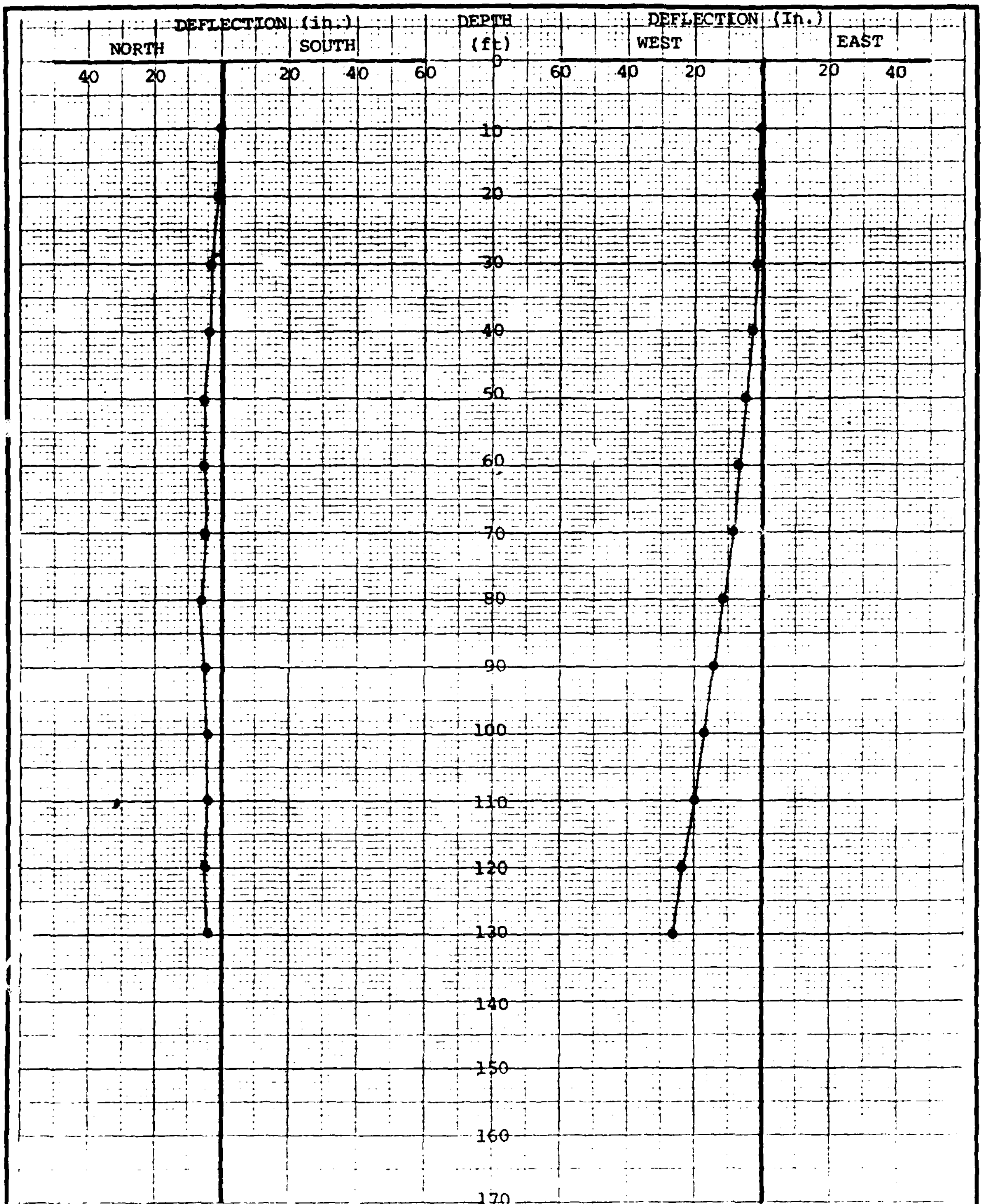


Reference Plate Data	
Date	5/3/79
Sta	24+00
Offset	Centerline
Elev	17.981

Anchor Number	Anchor Tip Elevation	[illegible]
6	-65.1	49.5'
5	-90.1	24.5'
4	-101.4	13.2'
3	-106.4	8.2'
2	-110.4	4.2'
1	-112.1	2.5'

Comments: All anchor elevations assume a vertical borehole.

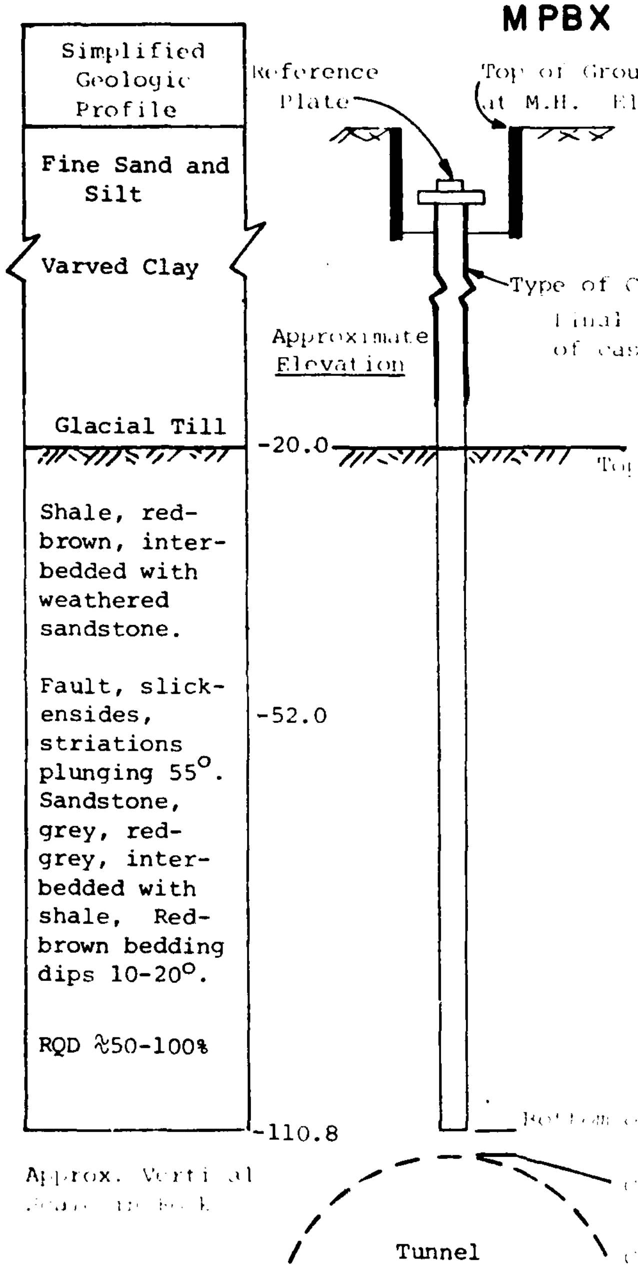
Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION MPBX NO. 4
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 9



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX-4 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig.1[illegible]

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 5



Reference Plate Data	
Date	9/27/79
Sta	27+00
Offset	Centerline
Elev	17.548'

Top of Rock Elev -20.01

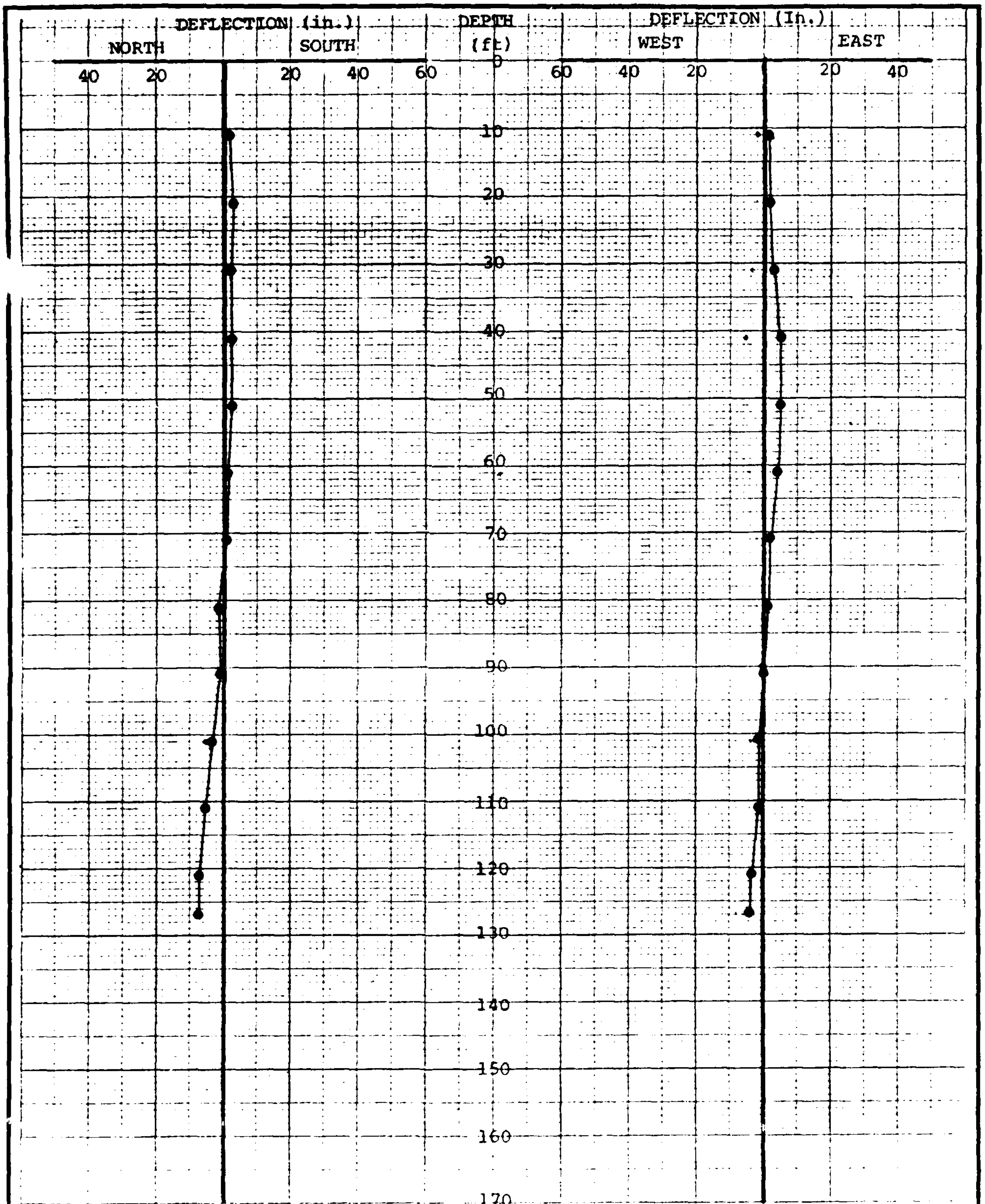
Anchor Number	Anchor Tip Elevation	[illegible] Above Crown
6	-51.2	61.6'
5	-88.6	24.2'
4	-98.6	14.2'
3	-104.1	8.7'
2	-108.6	4.2'
1	-110.6	2.2'

Bottom of Borehole Elev. -110.8'

Crown Elev -112.84

Comments: All anchor elevations assume a vertical borehole.

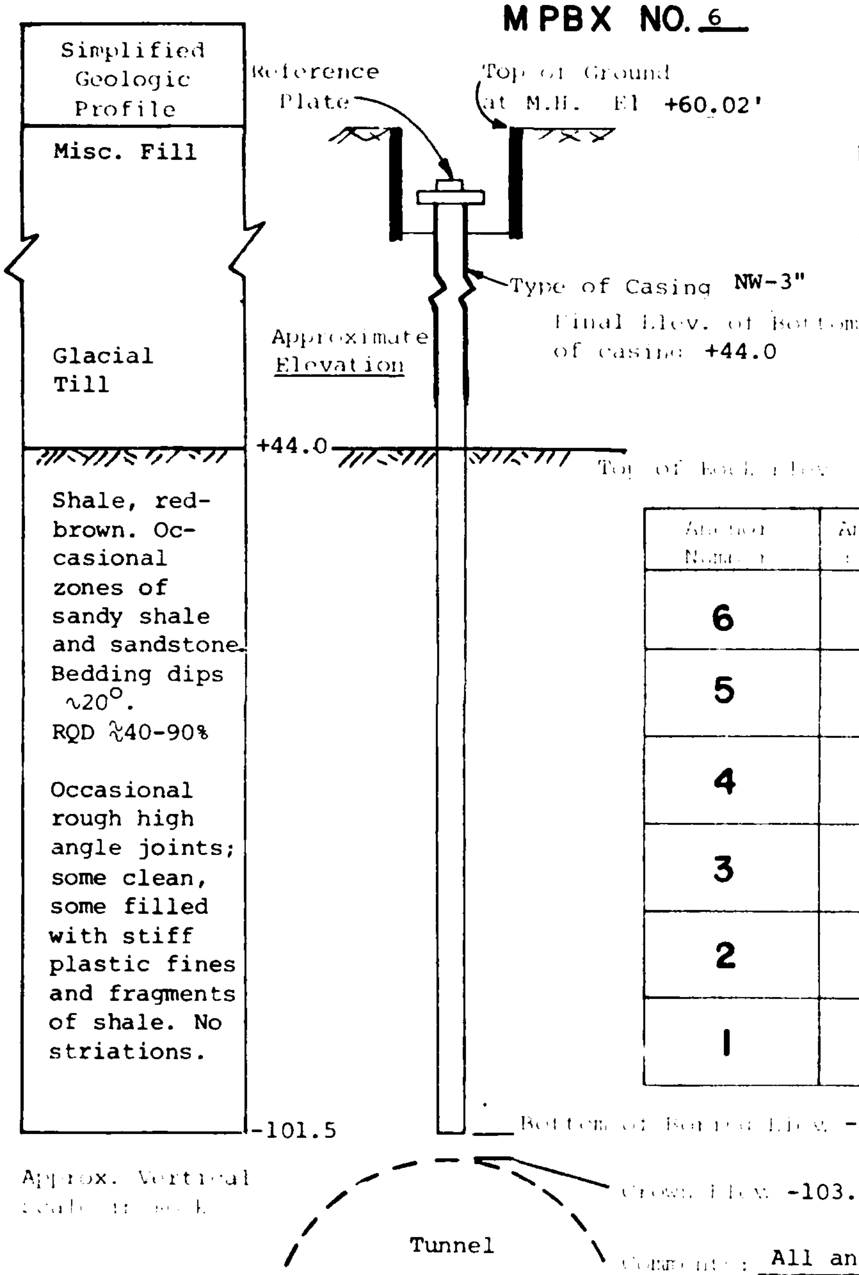
Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX NO. 5
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 11



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX-5 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig.12

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 6

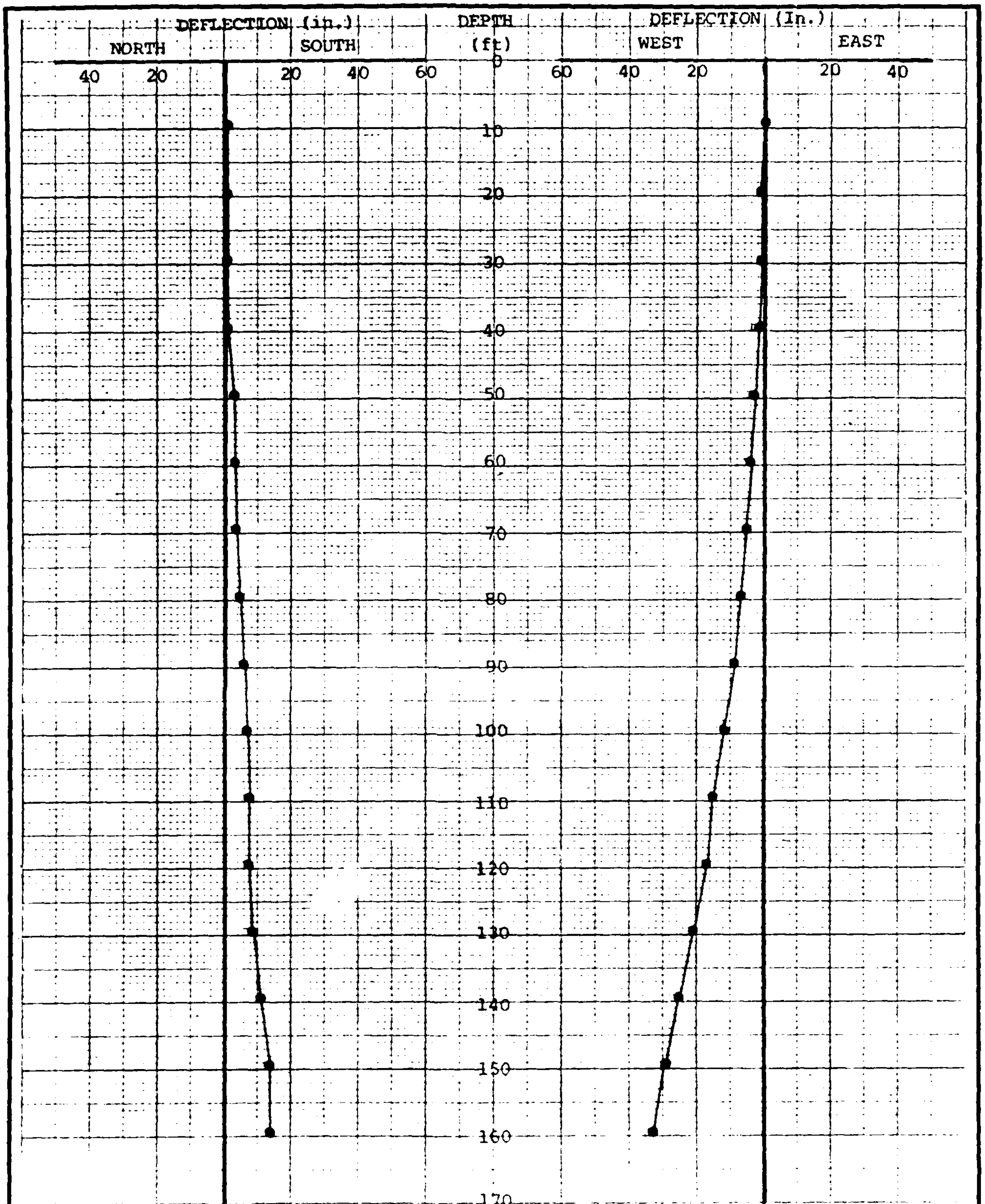


Reference Plate Data	
Date	8/18/80
Sta	43+50
Offset	Centerline
Elev	+58.924

Anchor Number	Anchor Tip Elevation	[illegible]
6	-55.2	48.2'
5	-78.6	24.9'
4	-92.6	10.9'
3	-95.5	8.0'
2	-99.0	4.5'
1	-101.0	2.5'

Comments: All anchor elevations assume a vertical borehole.

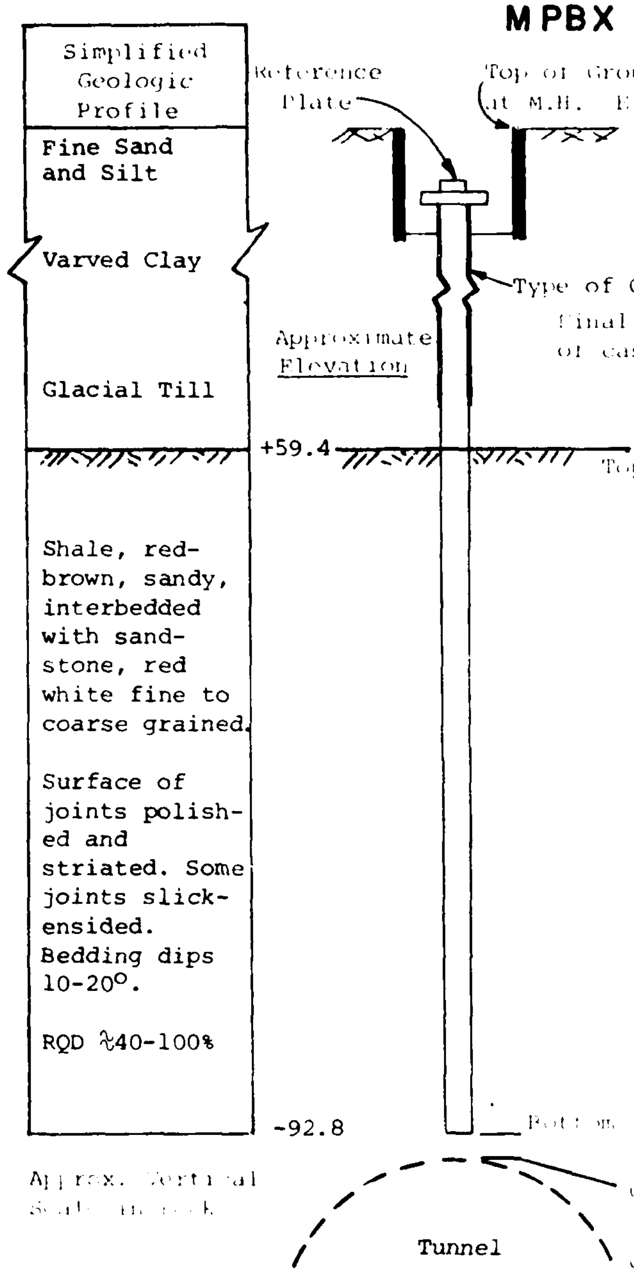
Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX No. 6
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 13



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX-6 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig.14

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 7



Reference Plate Data	
Date	8/18/80
Sta	58+55
Offset	Centerline
Elev	+73.932

Anchor Number	Anchor Tip Elev.	[illegible]
6	-46.7	48.1'
5	-70.5	24.3'
4	-82.5	12.3'
3	-87.0	7.8'
2	-90.6	4.2'
1	-92.1	2.7'

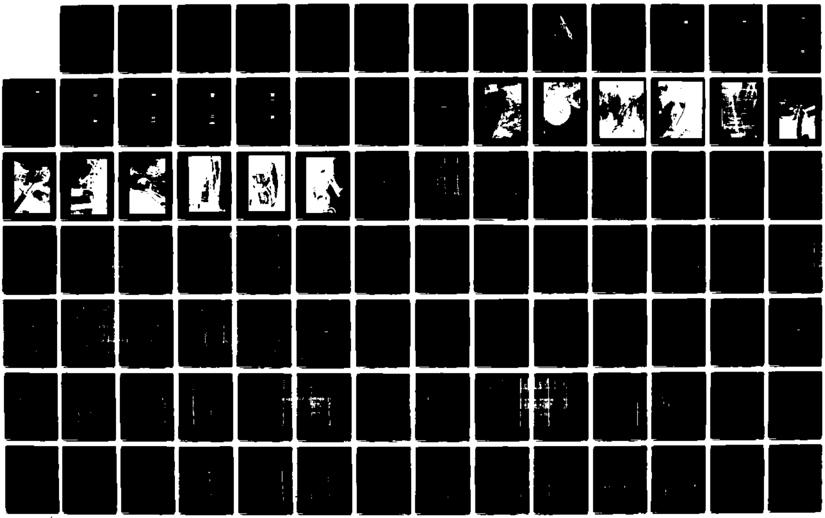
Comments: All anchor elevations assume a vertical borehcle.

Roger J. Au & Son, Inc. Mansfield, Oh	Park River Auxiliary Tunnel Hartford, CT	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX No. 7
GEOTECHNICAL ENGINEERS INC	Project 77382	August 1980 Fig. 15

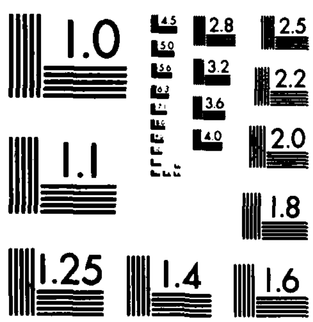
AD-A125 752 PARK RIVER LOCAL PROTECTION AUXILIARY CONDUIT TUNNEL AS-BUILT FOUNDATION..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV DEC 82

2/4

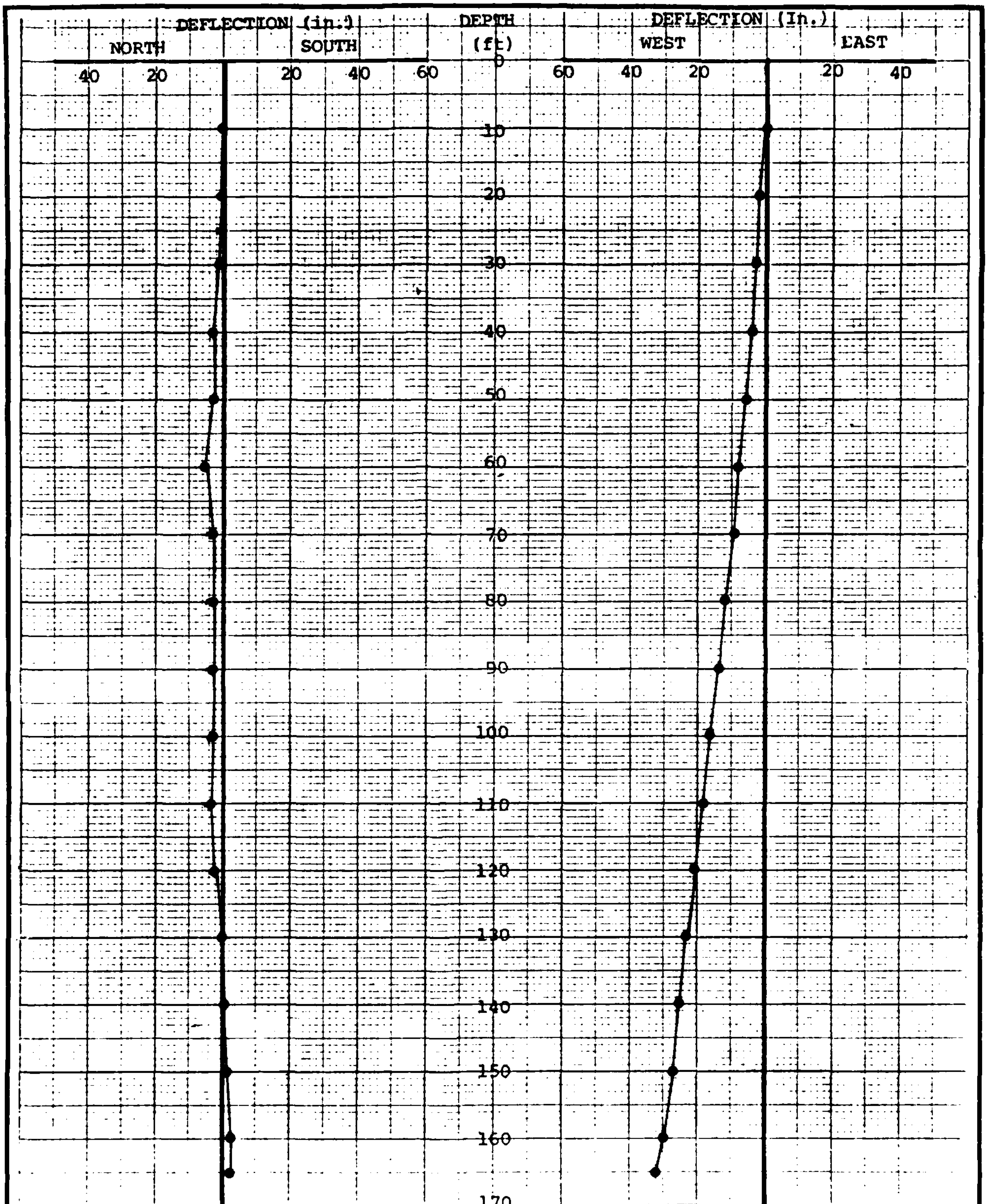
UNCLASSIFIED F/G 13/2 NL



M-2



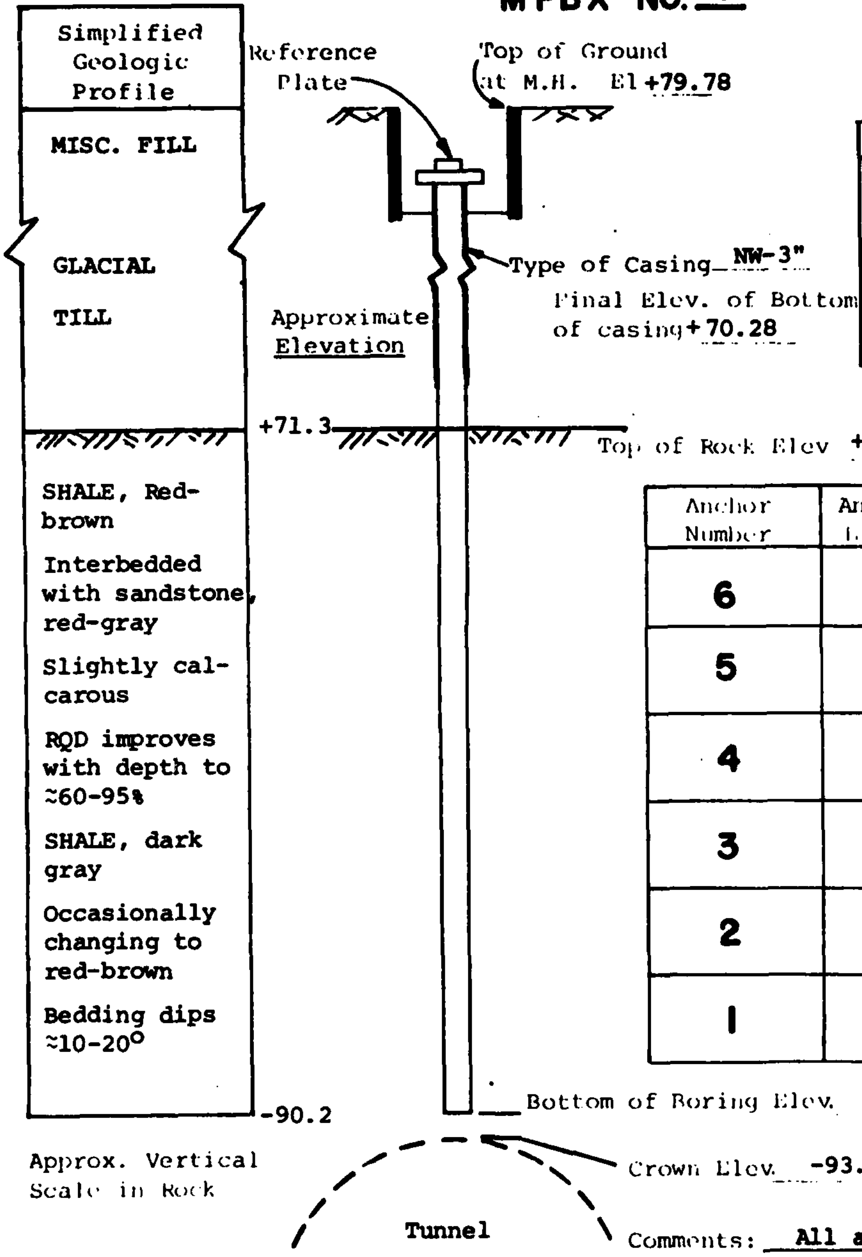
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX- 7 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig.16

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 8

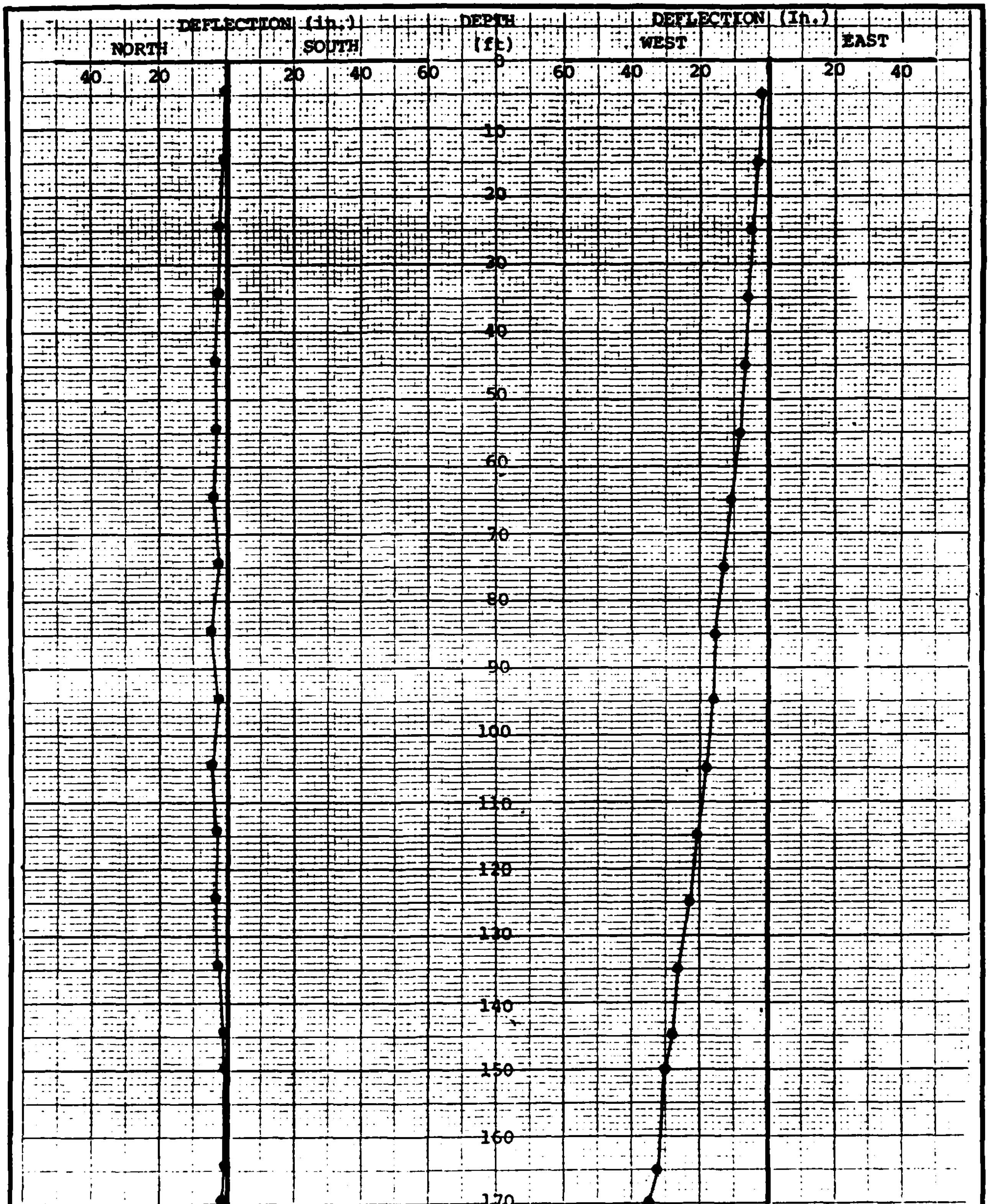


Reference Plate Data	
Date	3-24-80
Sta	61+19
Offset	Centerline
Elev	+78.86

Anchor Number	Anchor Tip Elevation	Distance Above Crown
6	-43.9	49.3'
5	-67.9	25.3'
4	-79.9	13.3'
3	-85.9	7.3'
2	-87.9	5.3'
1	-89.9	3.3'

Comments: All anchor elevations assume a verticle borehole.

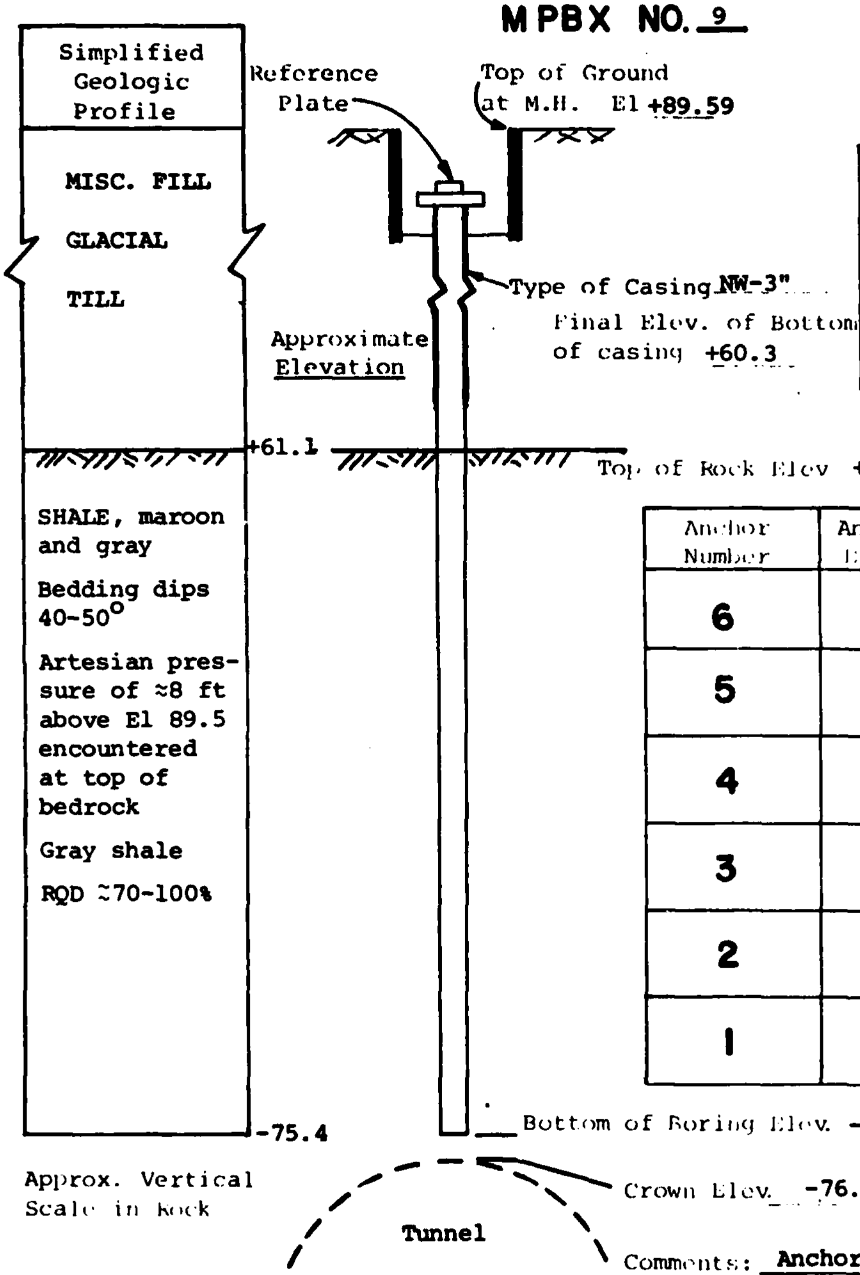
Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, Connecticut	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX NO. 8
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 17



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX-8 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig.18

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 9

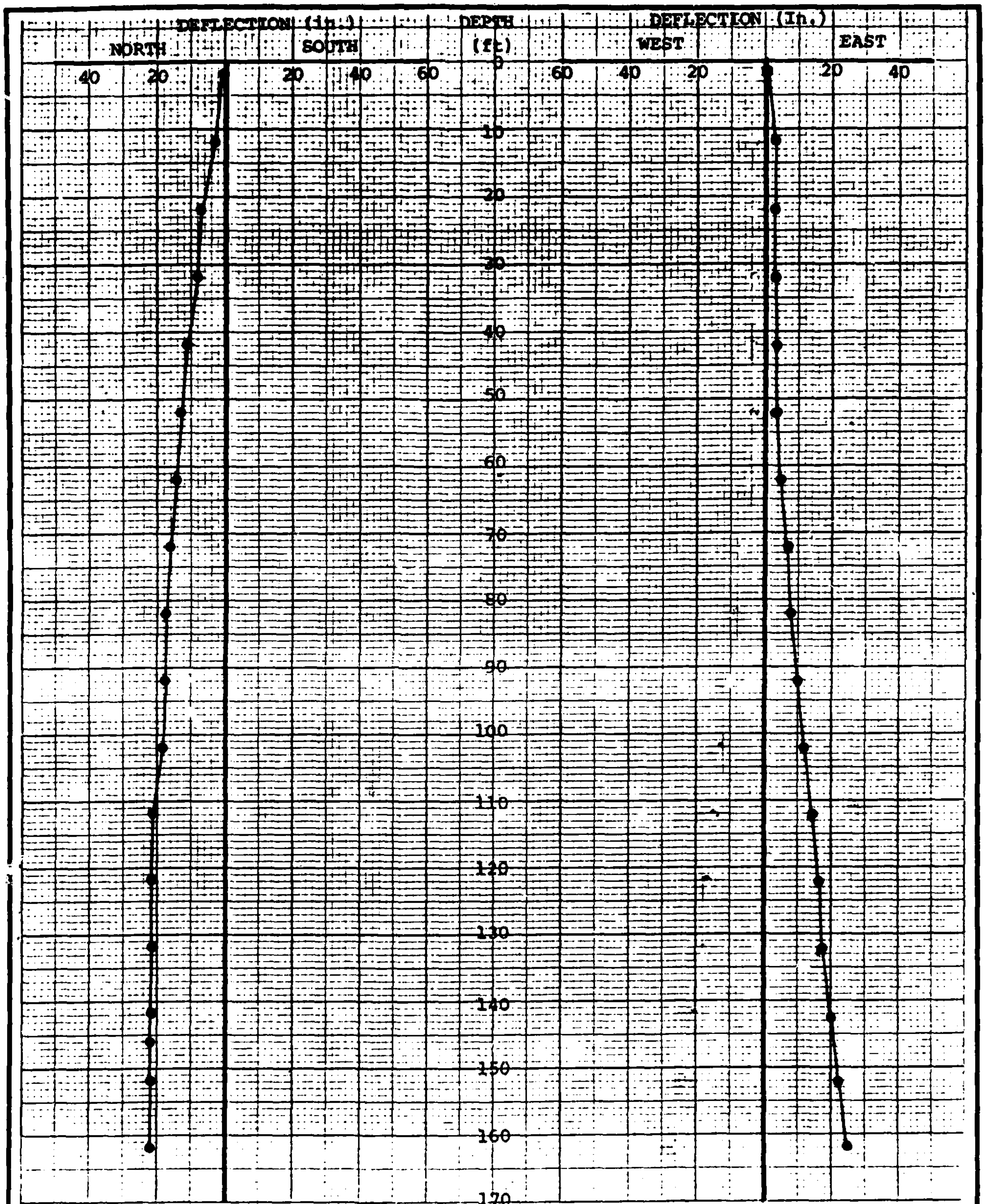


Reference Plate Data	
Date	8/18/80
Sta	91+00
Offset	Centerline
Elev	+88.973

Anchor Number	Anchor Tip Elevation	Distance Above Crown
6	-28.8	47.7'
5	-52.8	23.7'
4	-60.8	15.7'
3	-68.8	7.7'
2	-72.3	4.2'
1	-72.8	3.7'

Comments: Anchor elevations assume a vertical borehole. *Approximately 2 ft of rock core left in borehole after last run.

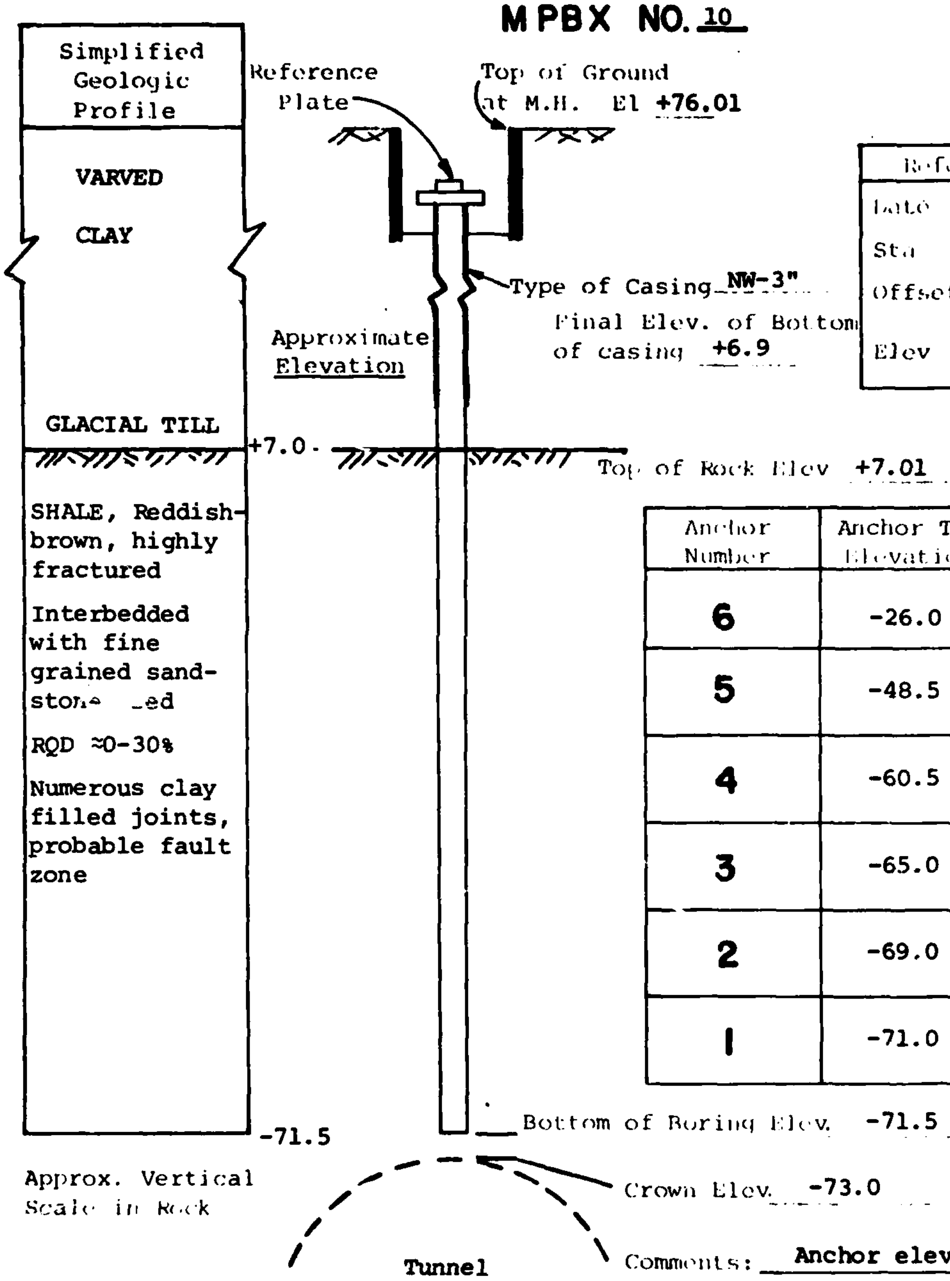
Roger J. Au & Son, Inc. Mansfield, Ohio GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Park River Auxiliary Tunnel Hartford, Connecticut Project 77382	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX NO. 9 August 1980 Fig. 19



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX- 9 BOREHOLE IN- CLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 20

MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA

MPBX NO. 10

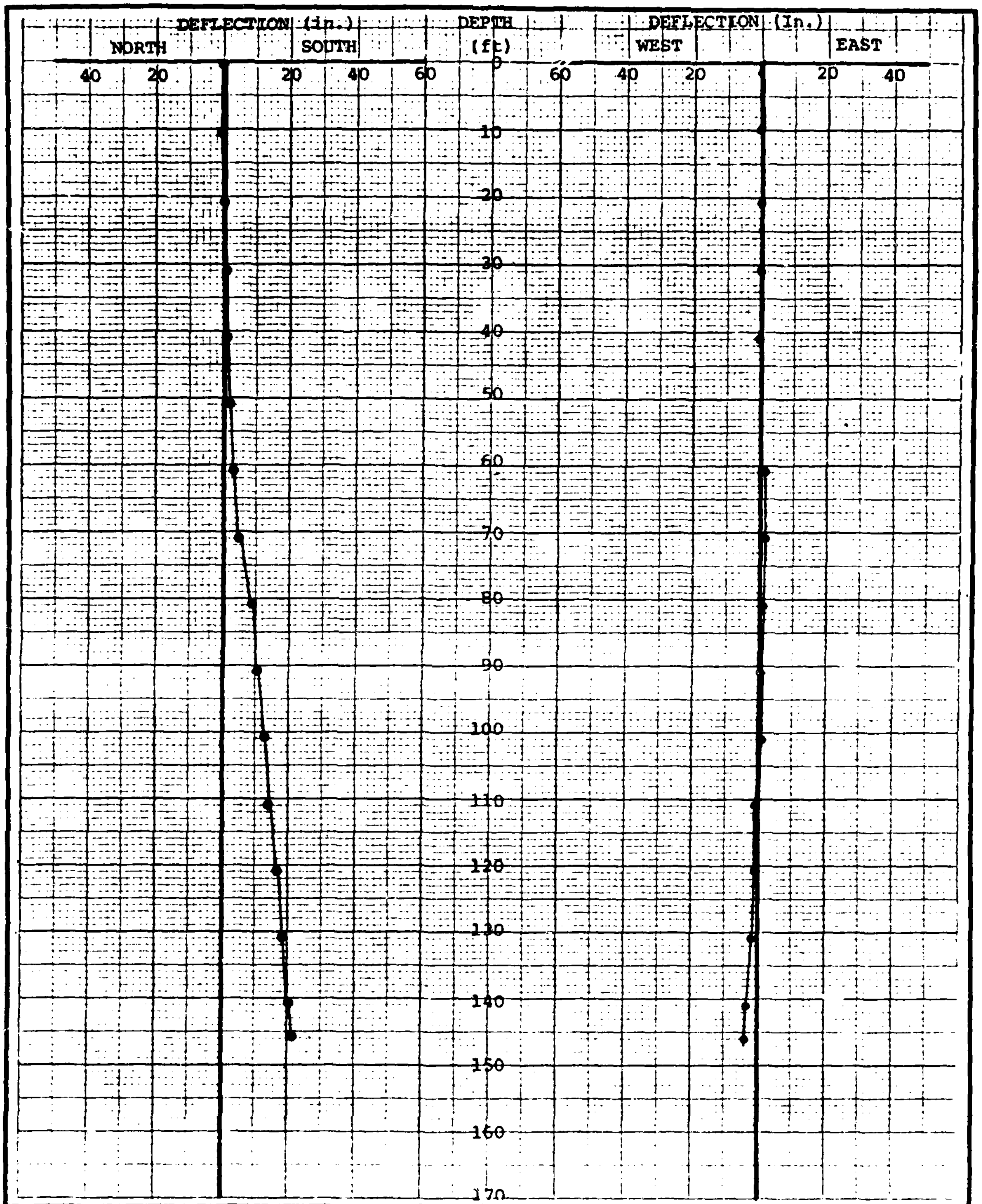


Reference Plate Data	
Date	8/18/80
Sta	95+54
Offset	Centerline
Elev	+75.045

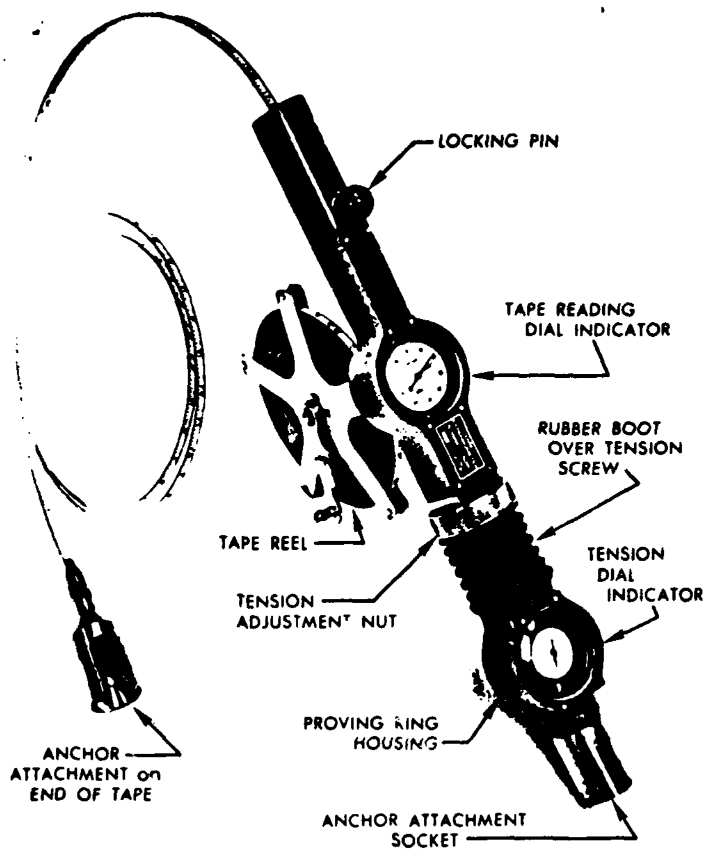
Anchor Number	Anchor Tip Elevation	Distance Above Crown
6	-26.0	47.0'
5	-48.5	24.5'
4	-60.5	12.5'
3	-65.0	8.0'
2	-69.0	4.0'
1	-71.0	2.0'

Comments: Anchor elevations assume a vertical borehole

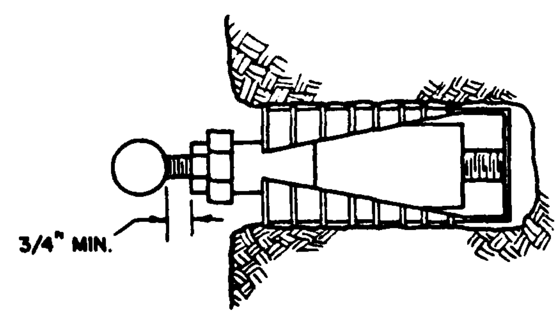
Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, Connecticut	MULTIPLE POSITION BOREHOLE EXTENSOMETER INSTALLATION DATA MPBX NO. 10
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 21



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	MPBX-10 BOREHOLE INCLINOMETER SURVEY DATA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 22

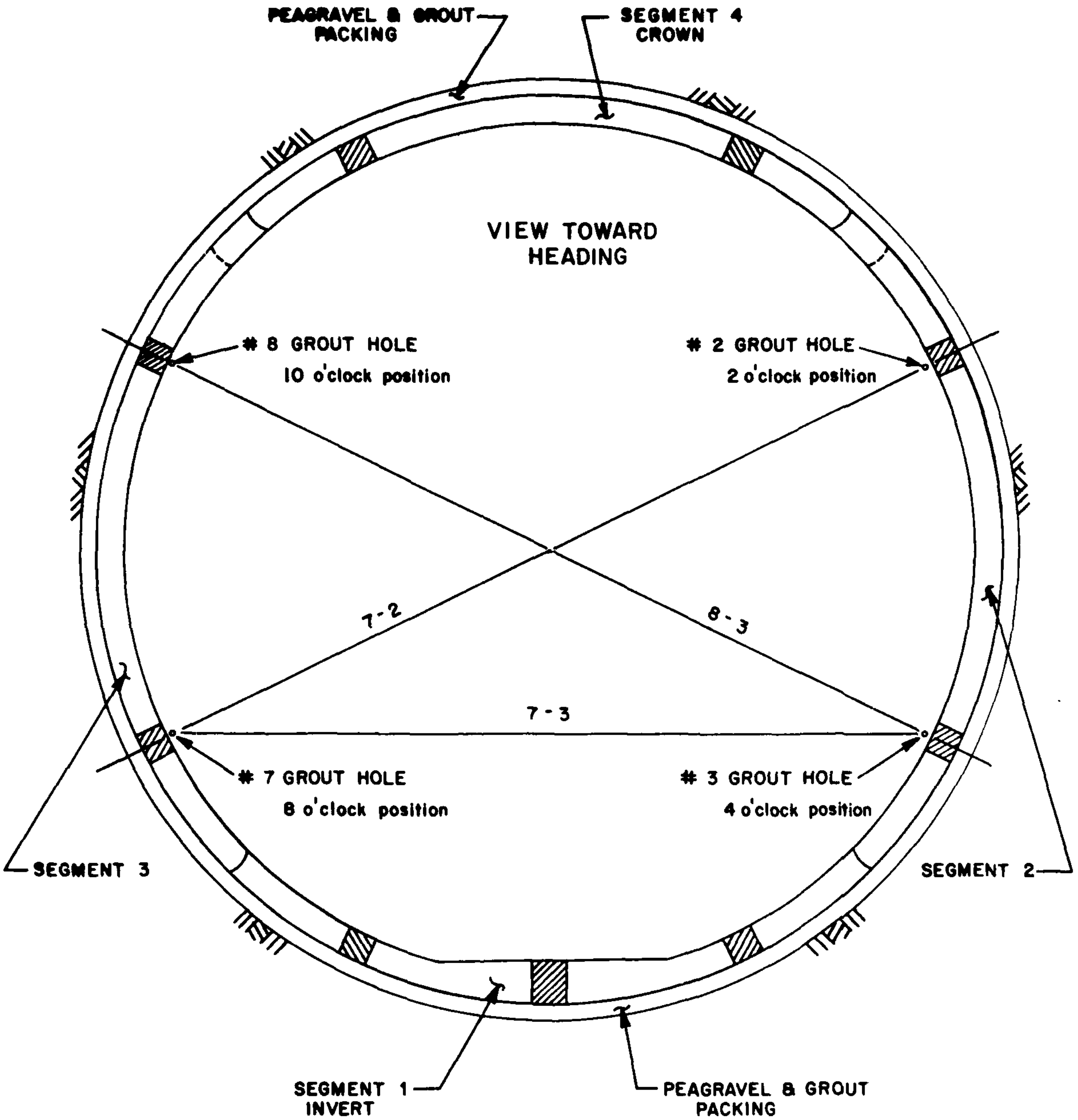


TAPE EXTENSOMETER
SINCo 51855



ANCHOR POINT AND STAR ANCHOR

Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	TAPE EXTENSOMETER DETAILS
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 23



Scale 1" = 4'

Roger J. Au and Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	TAPE EXTENSOMETER MEASUREMENT LOCATIONS
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 24

OBSERVATION WELL - 1

GROUND SURFACE ELEVATION - 22.6 FT

Depth ft	Sample No.	Blows Per 6 in.	Piezometer Installation Detail	Soil Description
	SS-1	1-1-2-3	Grout Bent.	Dk brown silty sand to fine sandy silt, topsoil
5	SS-2	1-2-2-3	Concrete sand 2.0"OD Solid Sch. 80 PVC	Dk brown fine sandy silt to silt
10	SS-3	2-3-2-2	2.0"OD Slotted Sch. 80 PVC	
15	SS-4	4-4-3-4	Ottawa Sand	Dk brown and gray brown fine sand and fine sandy silt. Stratified.
20	SS-5	3-3-3-4		
21.5				

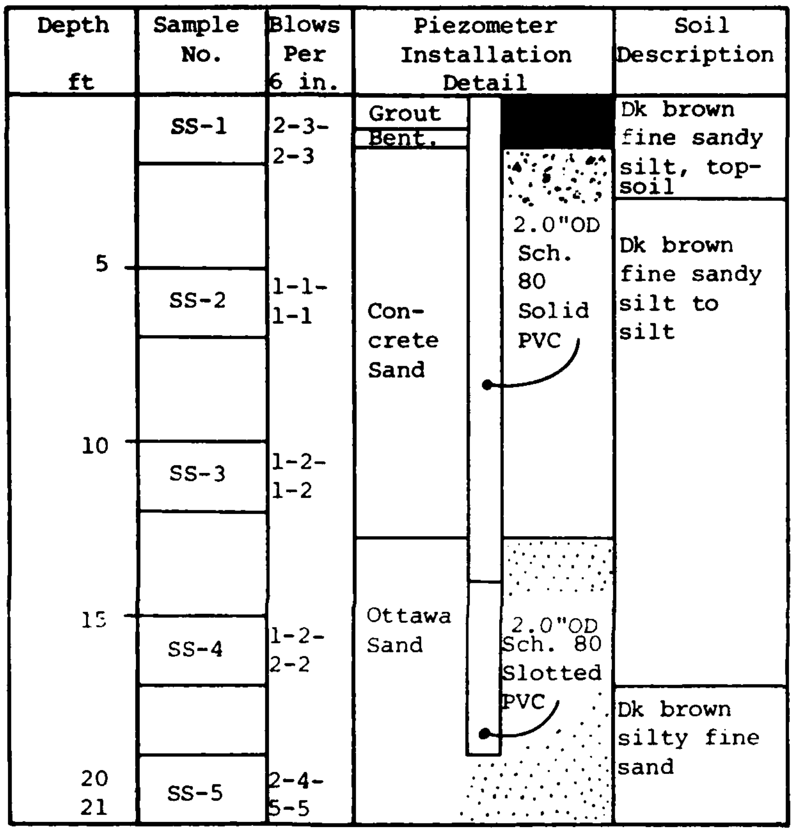
Depth To Top of Slotted PVC: 15.5'
Depth To Bottom of Slotted PVC: 20.5'
Depth To Groundwater: 10.5' on December 5, 1977

Date Installed-Nov. 28, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	SOIL PROFILE AND INSTALLATION DETAIL OF OBSERVATION WELL - 1
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. 25

OBSERVATION WELL - 2

GROUND SURFACE ELEVATION - 19.3 FT



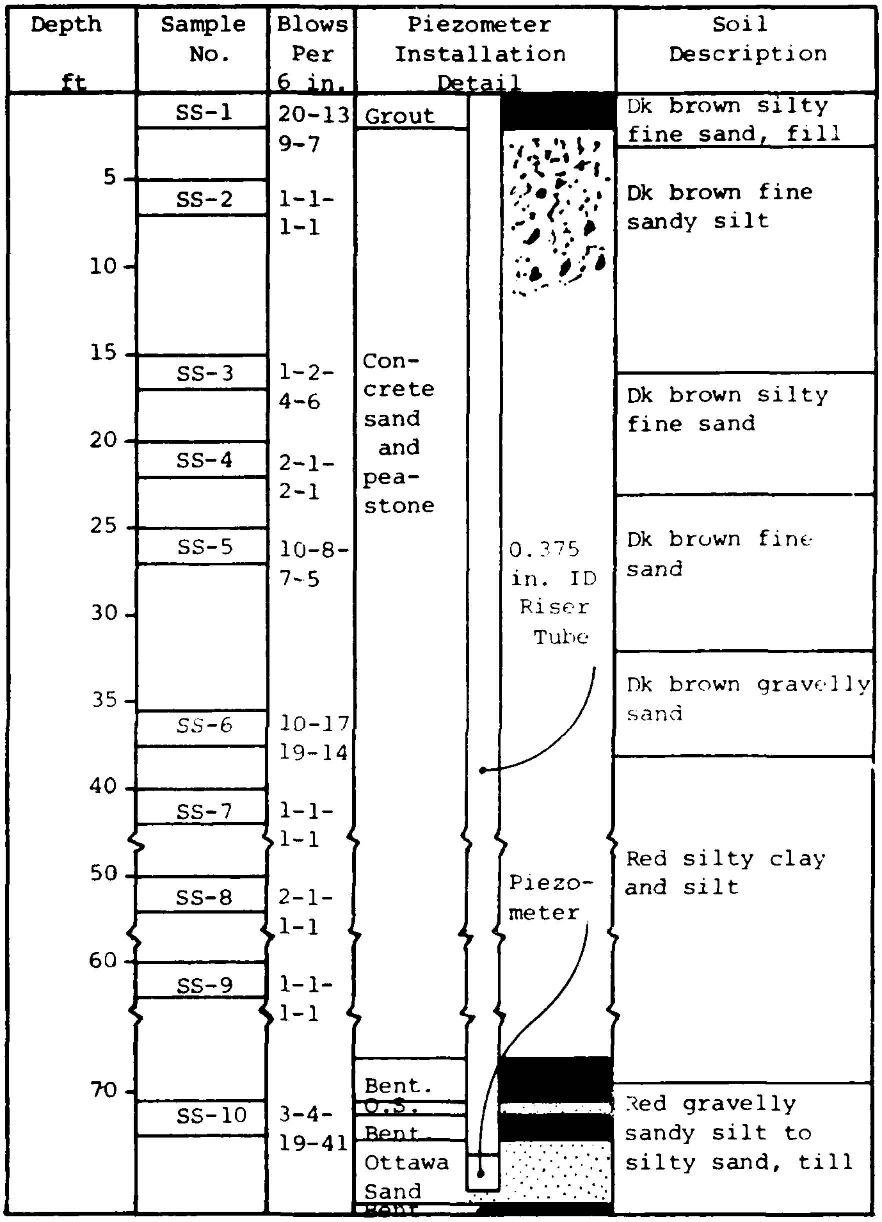
Depth To Top of Slotted PVC: 14.0'
Depth To Bottom of Slotted PVC: 19.0'
Depth To Groundwater: 6.7' on December 5, 1977

Date Installed - Nov. 29, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	SOIL PROFILE AND INSTALLATION DETAIL OF OBSERVATION WELL - 2
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. 26

PIEZOMETER NO. 1

GROUND SURFACE ELEVATION - 21.9 FT



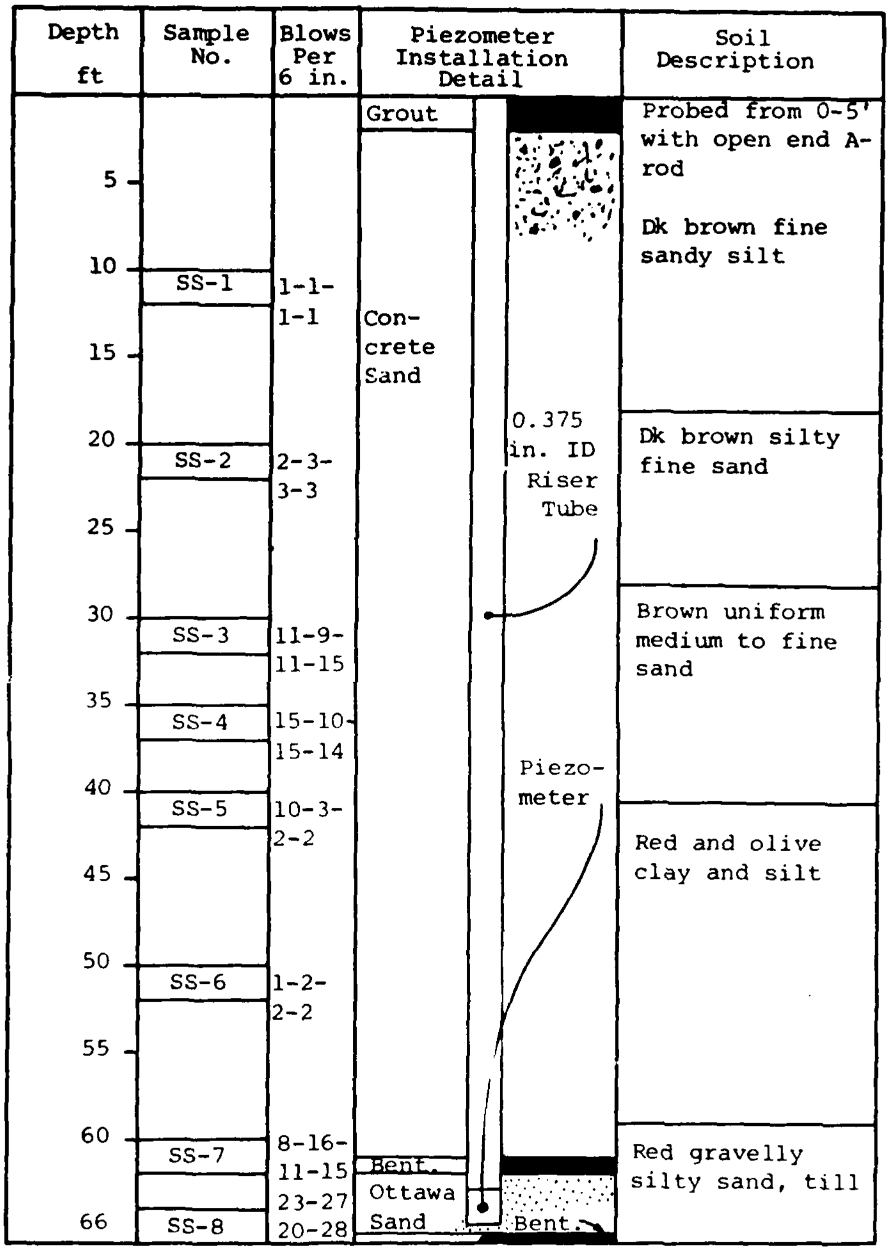
Depth To Piezometer: Top 73.6', Bottom 75.6'
Depth To First Seal: Top 76.3', Bottom 77.1'
Second Seal: Top 71.1', Bottom 72.8'
Depth to Groundwater: 12.2' on December 5, 1977

Date Installed-Dec. 2, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio Geotechnical Engineers Inc. Winchester, Massachusetts	Park River Auxiliary Tunnel Hartford, CT Project 77382	SOIL PROFILE AND INSTALLATION DETAIL OF PIEZOMETER NO. 1 August 1980 Fig. 27

PIEZOMETER NO. 2

GROUND SURFACE ELEVATION - 20.9 FT



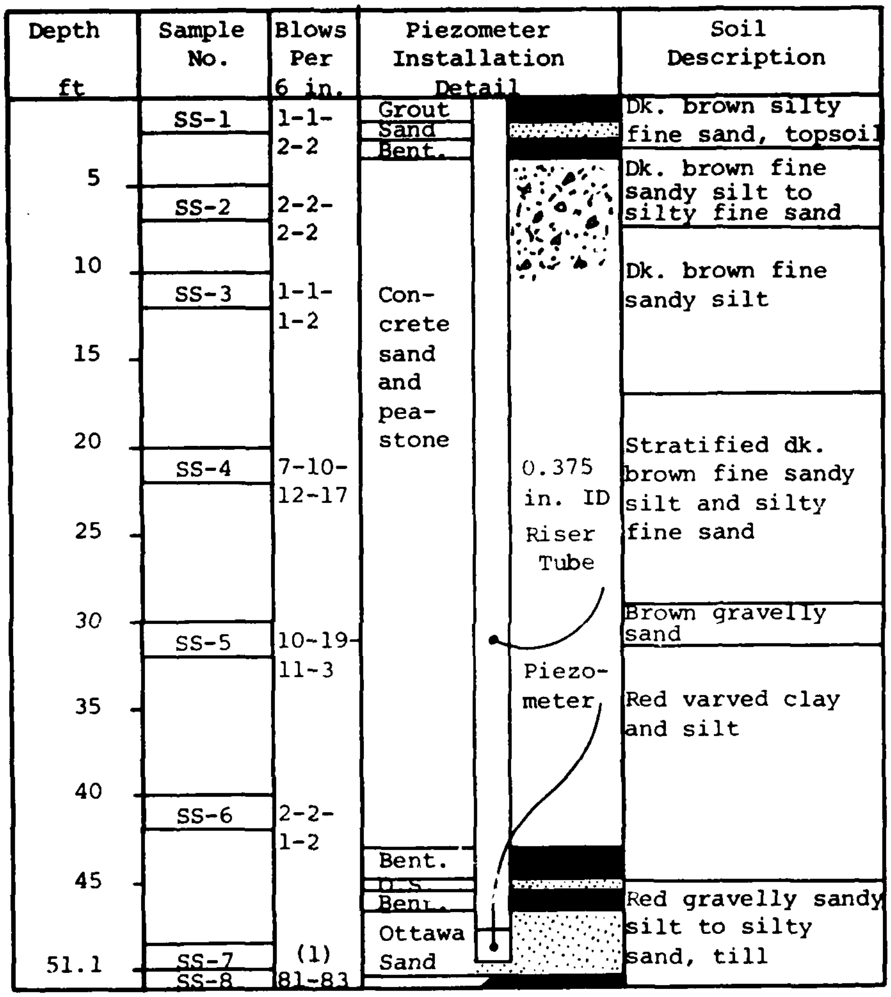
Depth To Piezometer: Top 62.8', Bottom 64.8'
Depth To First Seal: Top 65.3', Bottom 65.7'
Second Seal: Top 60.8', Bottom 61.8'
Depth To Groundwater: Not available

Date Installed-Dec. 5, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	SOIL PROFILE AND INSTALLATION DETAIL OF PIEZOMETER NO. 2
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. 28

PIEZOMETER NO. 3

GROUND SURFACE ELEVATION - 22.7 FT



Depth To Piezometer: Top 47.8', Bottom 49.8'
Depth To First Seal: Top 51.1', Bottom 50.6'
Second Seal: Top 45.6', Bottom 46.7'
Third Seal: Top 43.1', Bottom 44.9'
Depth To Groundwater: 11.1' on December 5, 1977

Notes:
(1) Blows of 100/3" with 140-lb hammer and 65/3"-60/6"-70/6" with 300-lb hammer using open end A-rod.

Date Installed-Nov. 30, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio Geotechnical Engineers Inc. Winchester, Massachusetts	Park River Auxiliary Tunnel Hartford, CT Project 77382	SOIL PROFILE AND INSTALLATION DETAIL OF PIEZOMETER NO. 3 August 1980 Fig. 29

PIEZOMETER NO. 4

GROUND SURFACE ELEVATION - 22.5 FT

Depth ft	Sample No.	Blows Per 6 in.	Piezometer Installation Detail	Soil Description
	SS-1	2-3-2-2	Grout Bent.	Dk brown fine sandy silt, topsoil
5	SS-2	3-1-2-3	Sand and pea-stone	Dk brown fine sandy silt to silt
10				
15	SS-3	3-4-6-8	0.375 in. ID Riser Tube	
20				
25	SS-4	20-8-11-12		Gravelly silty sand and stratified silty sands
30			Piezometer	
35	SS-5	2-1-2-2		Red silty clay and silt
40	SS-6	7-12-9-10	Bent. O.S. Bent. Ottawa Sand	Red gravelly sandy silt, till
45	SS-7	56-5 1/4"		
47.5			Bent.	

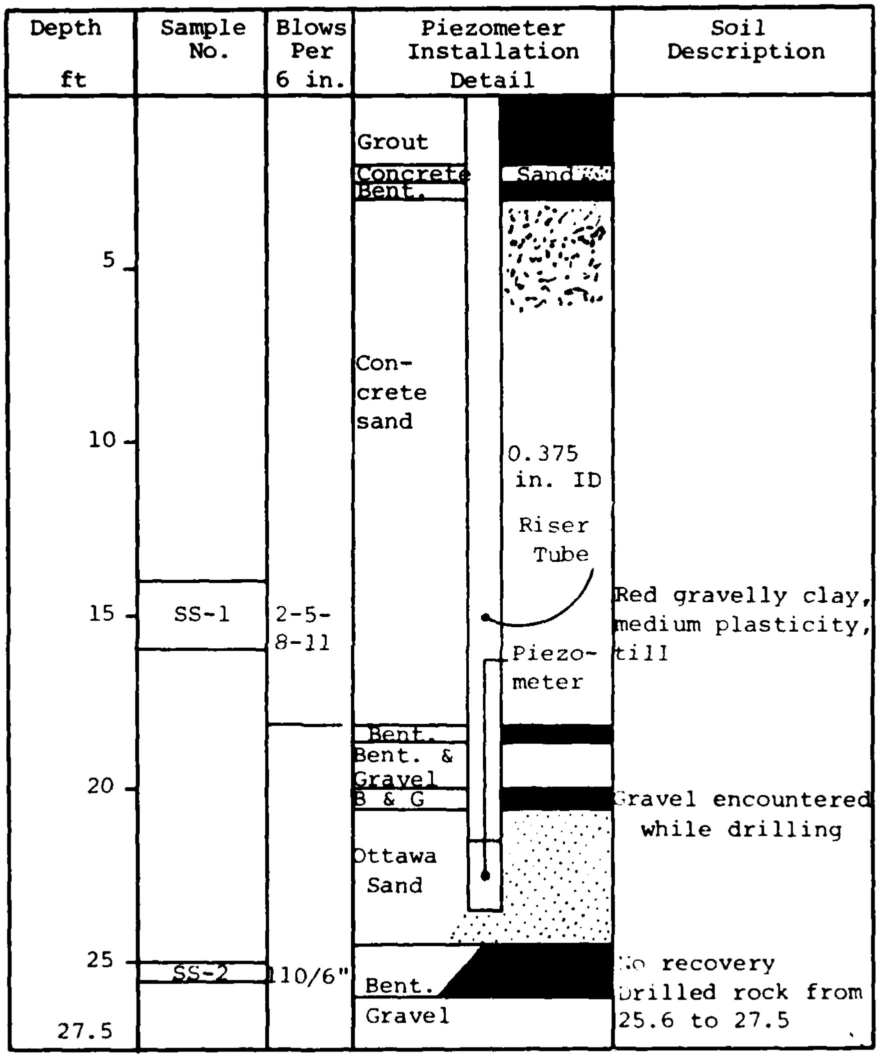
Depth To Piezometer: Top 43.2', Bottom 45.2'
Depth To First Seal: Top 46.2', Bottom 47.1'
Second Seal: Top 41.1', Bottom 42.2'
Third Seal: Top 38.9', Bottom 40.2'
Depth To Groundwater: 11.1' on December 5, 1977

Date Installed-Nov. 29, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	SOIL PROFILE AND INSTALLATION DETAIL OF PIEZOMETER NO. 4
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. 30

PIEZOMETER NO. 5

GROUND SURFACE ELEVATION - 69.7 FT



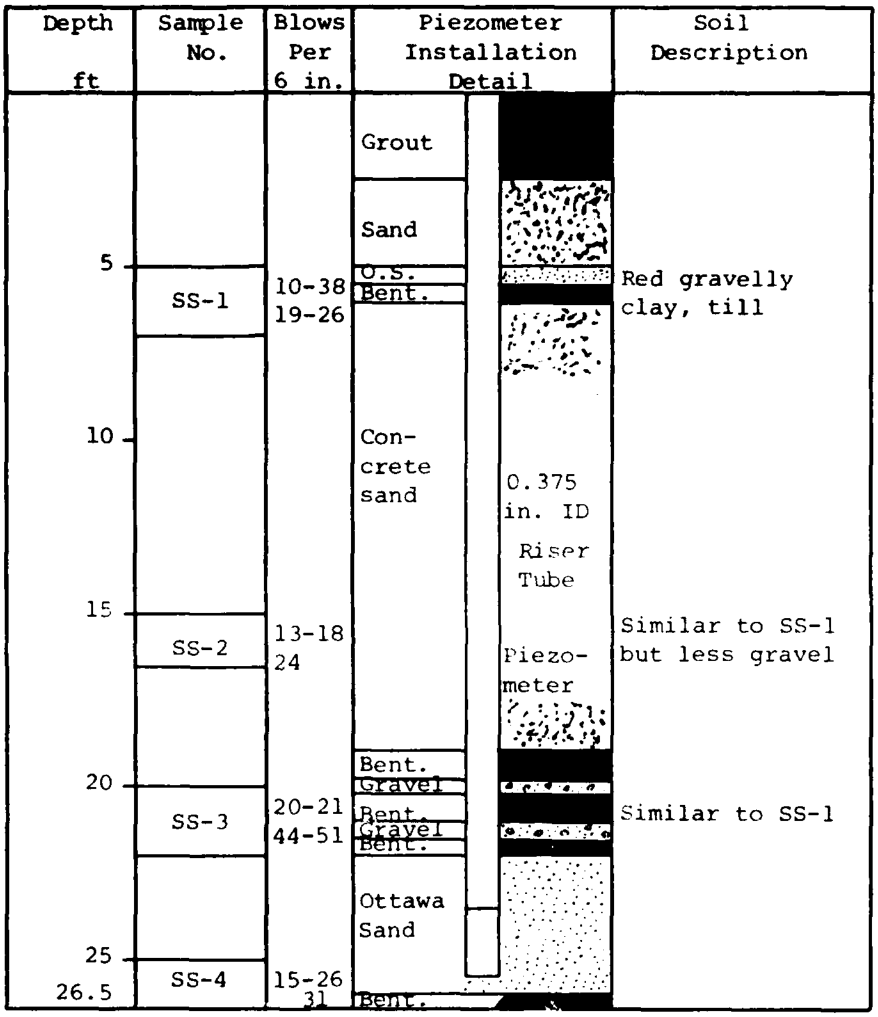
Depth To Piezometer: Top 21.5', Bottom 23.5'
Depth To First Seal: Top 24.2', Bottom 26.0'
Second Seal: Top 20.0', Bottom 20.8'
Third Seal: Top 18.2', Bottom 18.7'
Depth To Groundwater: 5.0' on December 5, 1977

Date Installed-Dec. 2, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	SOIL PROFILE AND INSTALLATION DETAIL OF PIEZOMETER NO. 5
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. 31

PIEZOMETER NO. 6

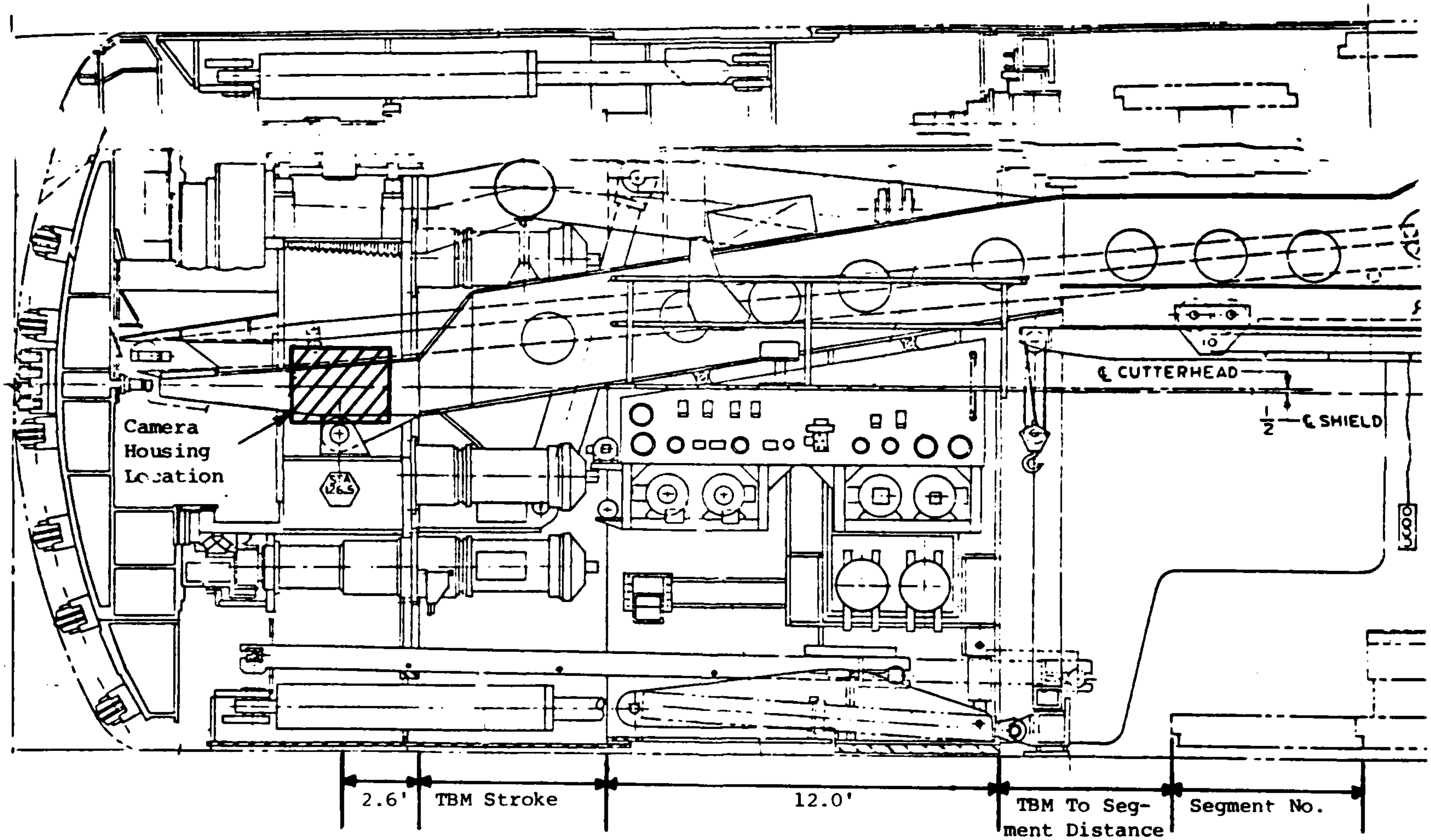
GROUND SURFACE ELEVATION - 81.0 FT



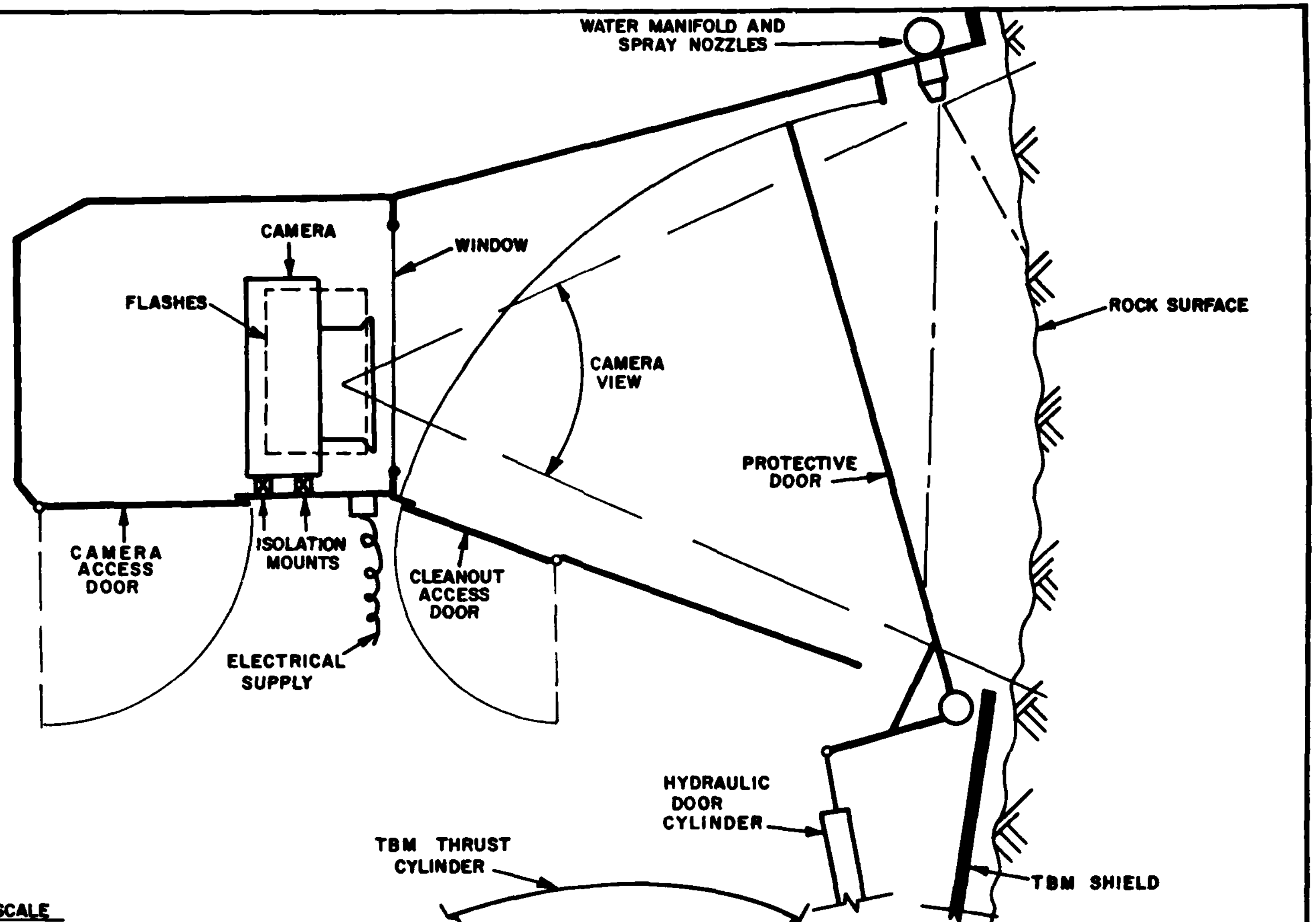
Depth To Piezometer: Top 23.5', Bottom 25.5'
Depth To First Seal: Top 26.0', Bottom 26.5'
Second Seal: Top 21.5', Bottom 22.0'
Third Seal: Top 20.5', Bottom 21.0'
Depth To Groundwater: 0.5' on December 5, 1977

Date Installed - Dec. 3, 1977

Roger J. Au & Son, Inc. Mansfield, Ohio Geotechnical Engineers Inc. Winchester, Massachusetts	Park River Auxiliary Tunnel Hartford, CT Project 77382	SOIL PROFILE AND INSTALLATION DETAIL OF PIEZOMETER NO. 6 August 1980 Fig. 32



Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	CAMERA LOCATION AND STATIONING METHOD
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 33



APPROXIMATE SCALE
1" = 6"

HOUSING MOUNTED ON RIGHT SIDE OF
TBM APPROXIMATELY AT SPRINGLINE.

Roger J. Au & Son, Inc. Mansfield, OH	Park River Auxiliary Tunnel Hartford, CT	CROSS SECTION CAMERA HOUSING
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. 34

PHOTOGRAPHS

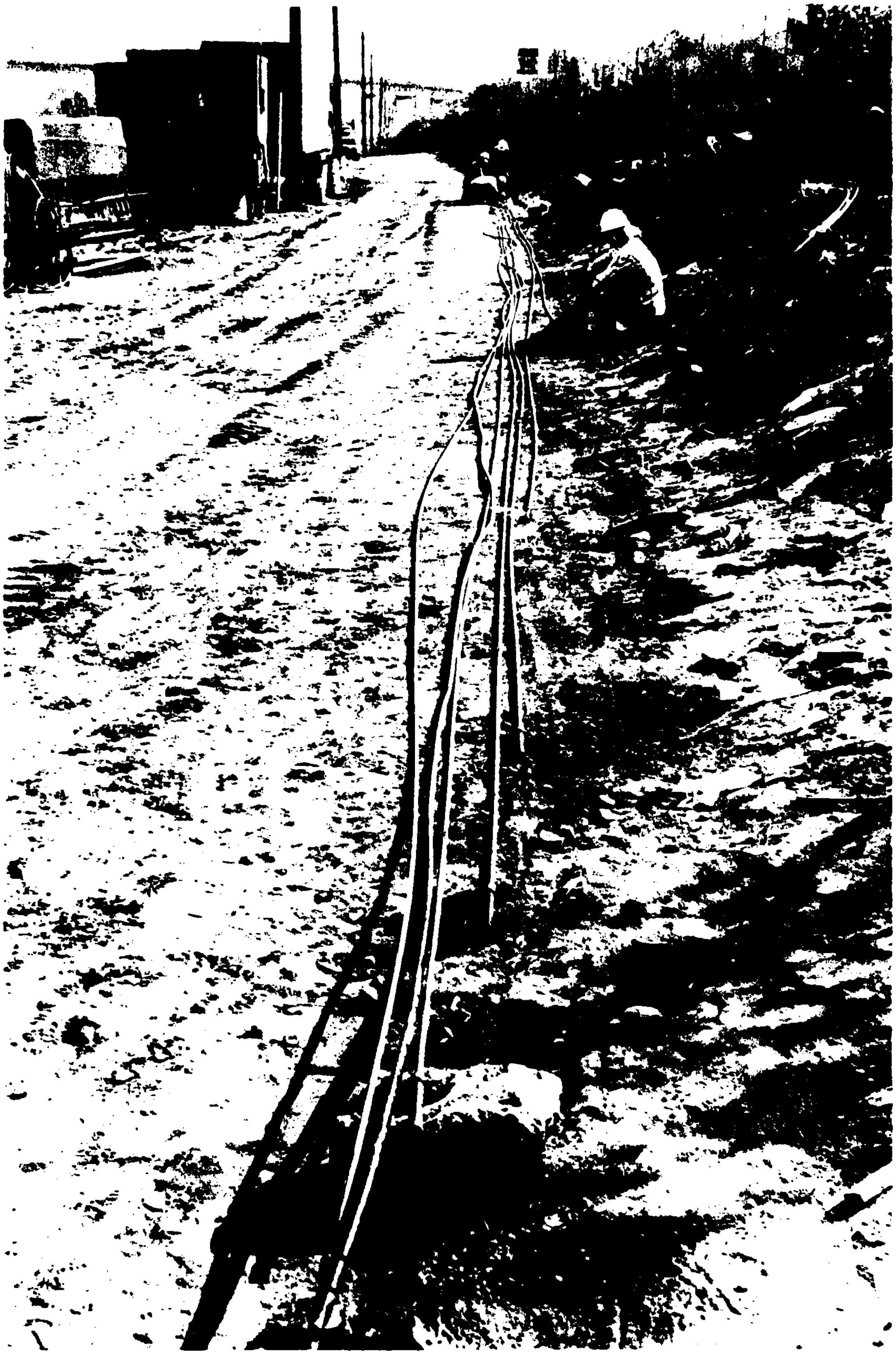


PHOTO #1 - MPBX anchors, rod and grout tube.

PHOTO #2 - MPPX reference head.

PHOTO #3 - Typical rock bolt load cell installation.

PHOTO #4 - Typical rock bolt load cell installation.

PHOTO #5 - Embedded strain gages and handy boxes mounted on rebar cage.

PHOTO #6 - Adjustment of surface strain gage.

PHOTO #7 - Attaching tape extensometer
to anchor point.

PHOTO #8 - Adjusting tape extensometer.

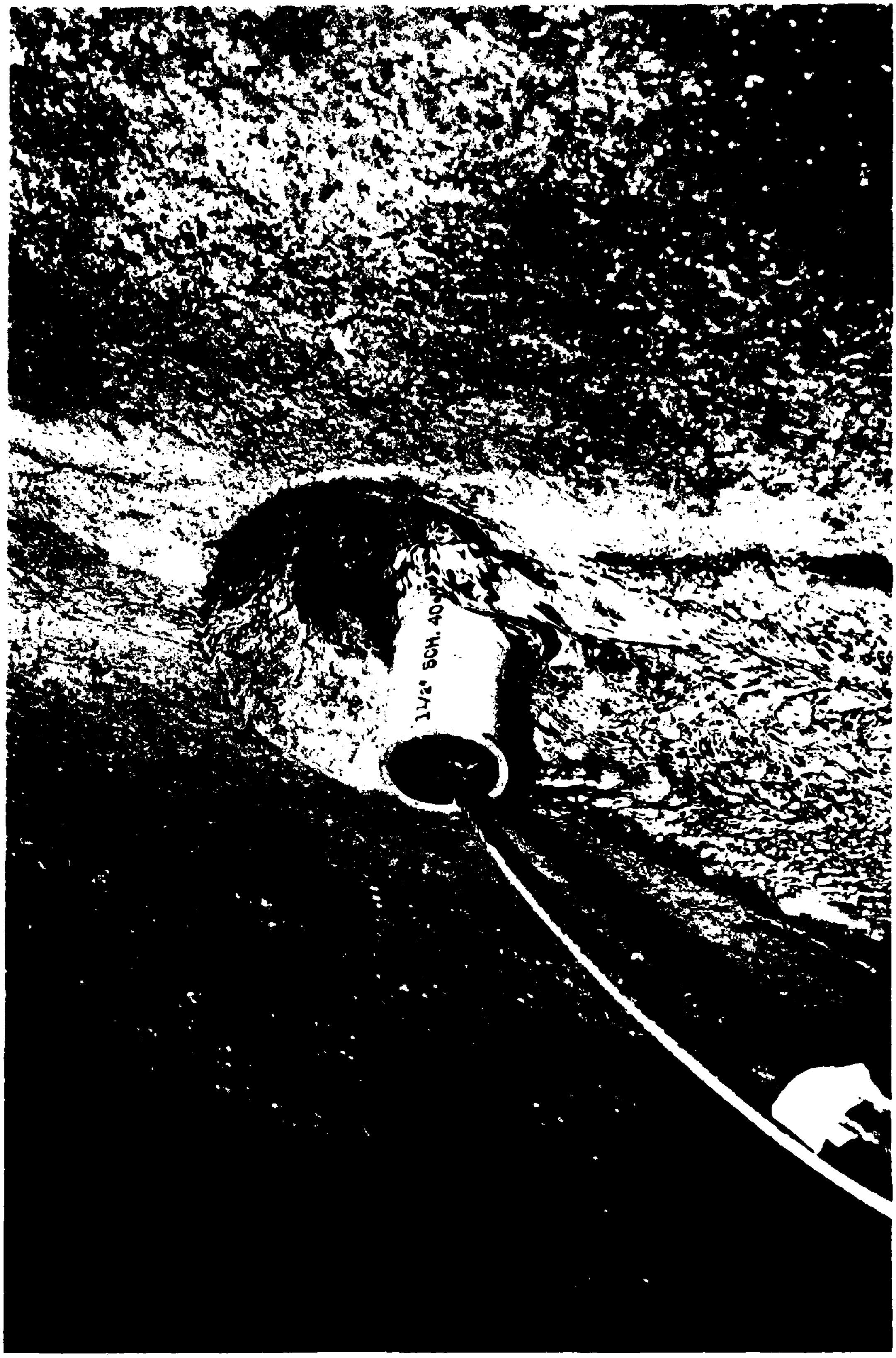


PHOTO #9 - Inserting tunnel piezometer
into borehole.

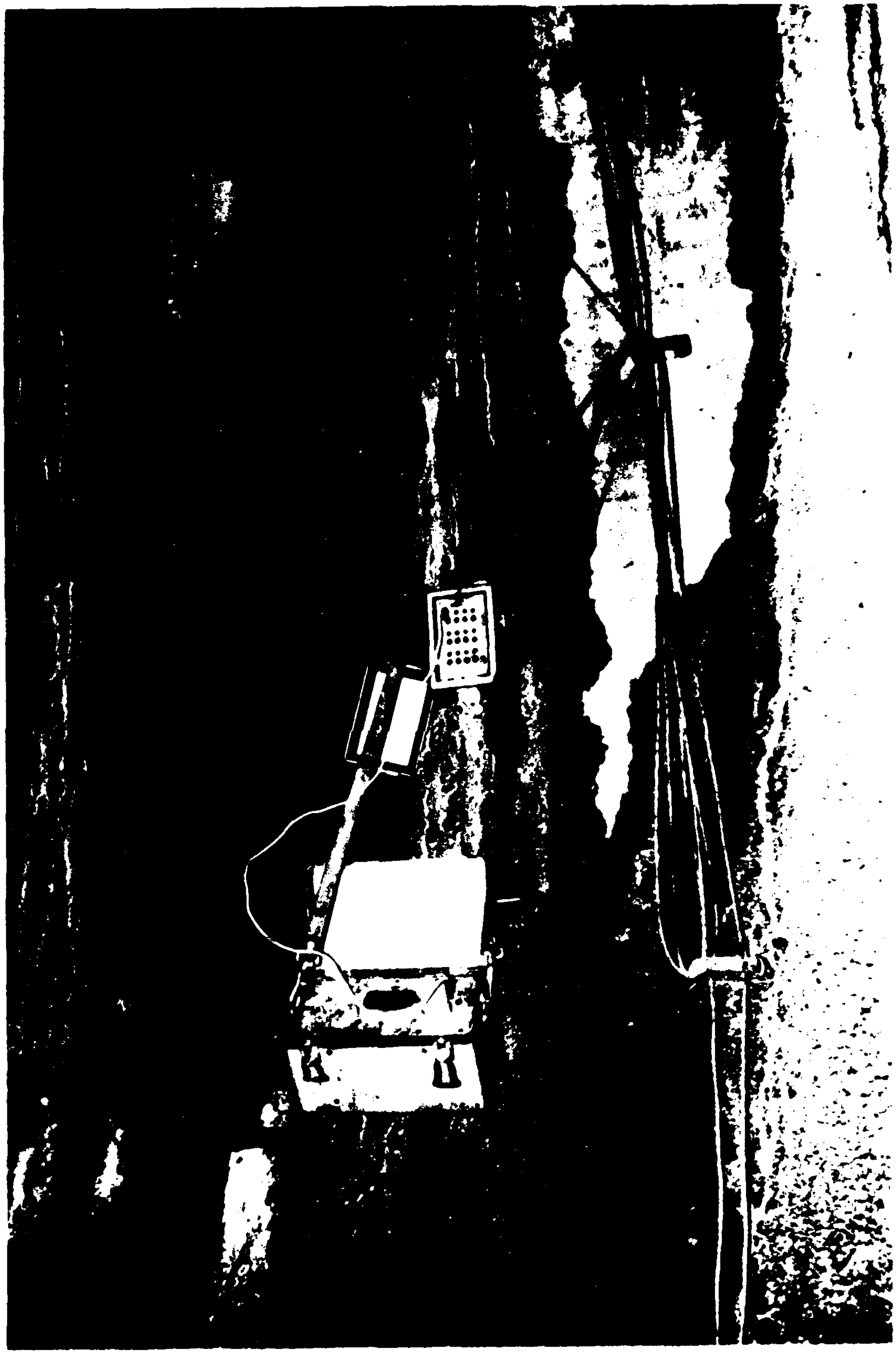


PHOTO #10 - Typical instrumented ring showing surface strain gage, terminal box, readout box, reference anchor and sealed piezometer hole.

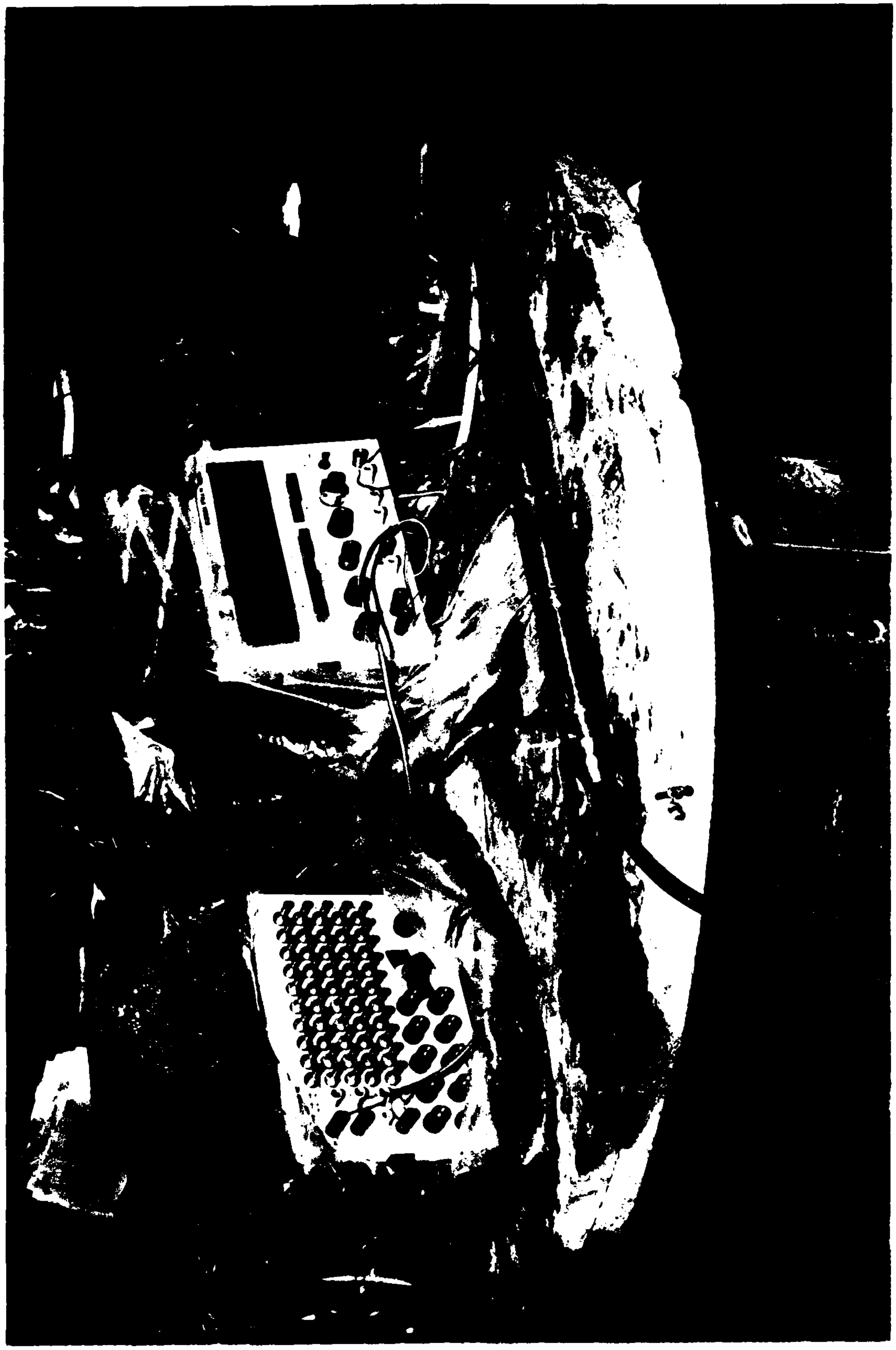
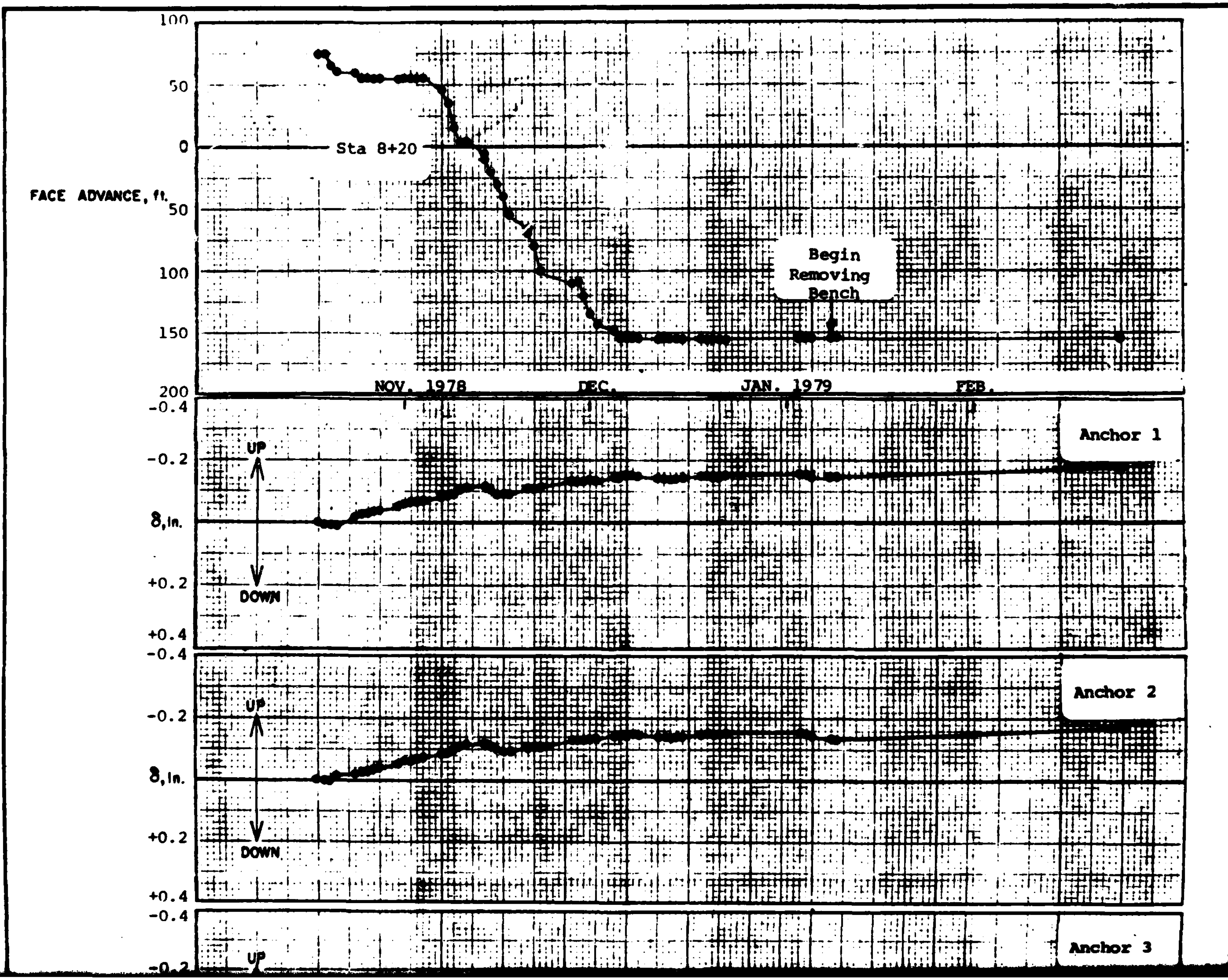
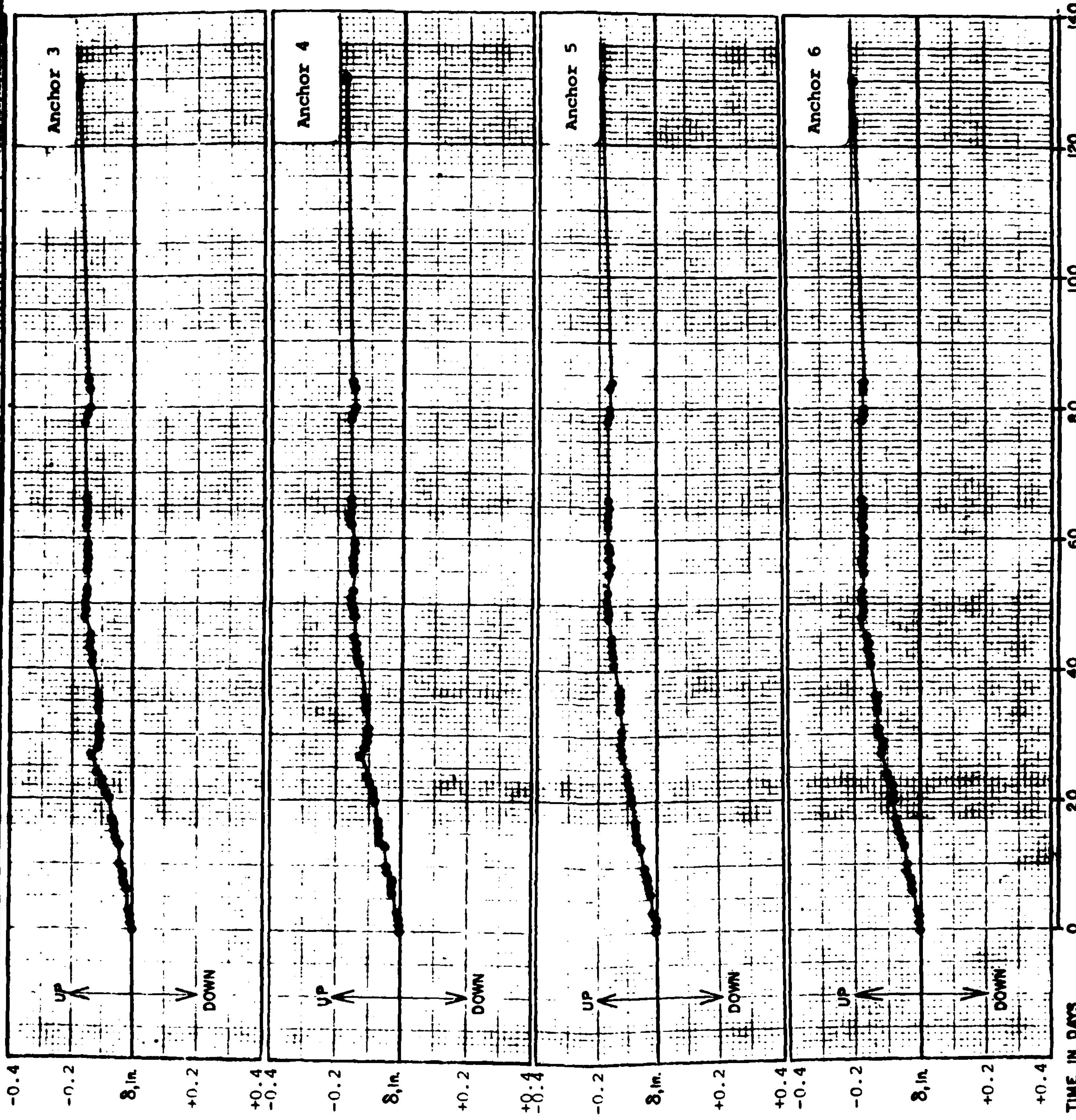


PHOTO #11 - USBM overcoring borehole gage, digital readout and switching unit.

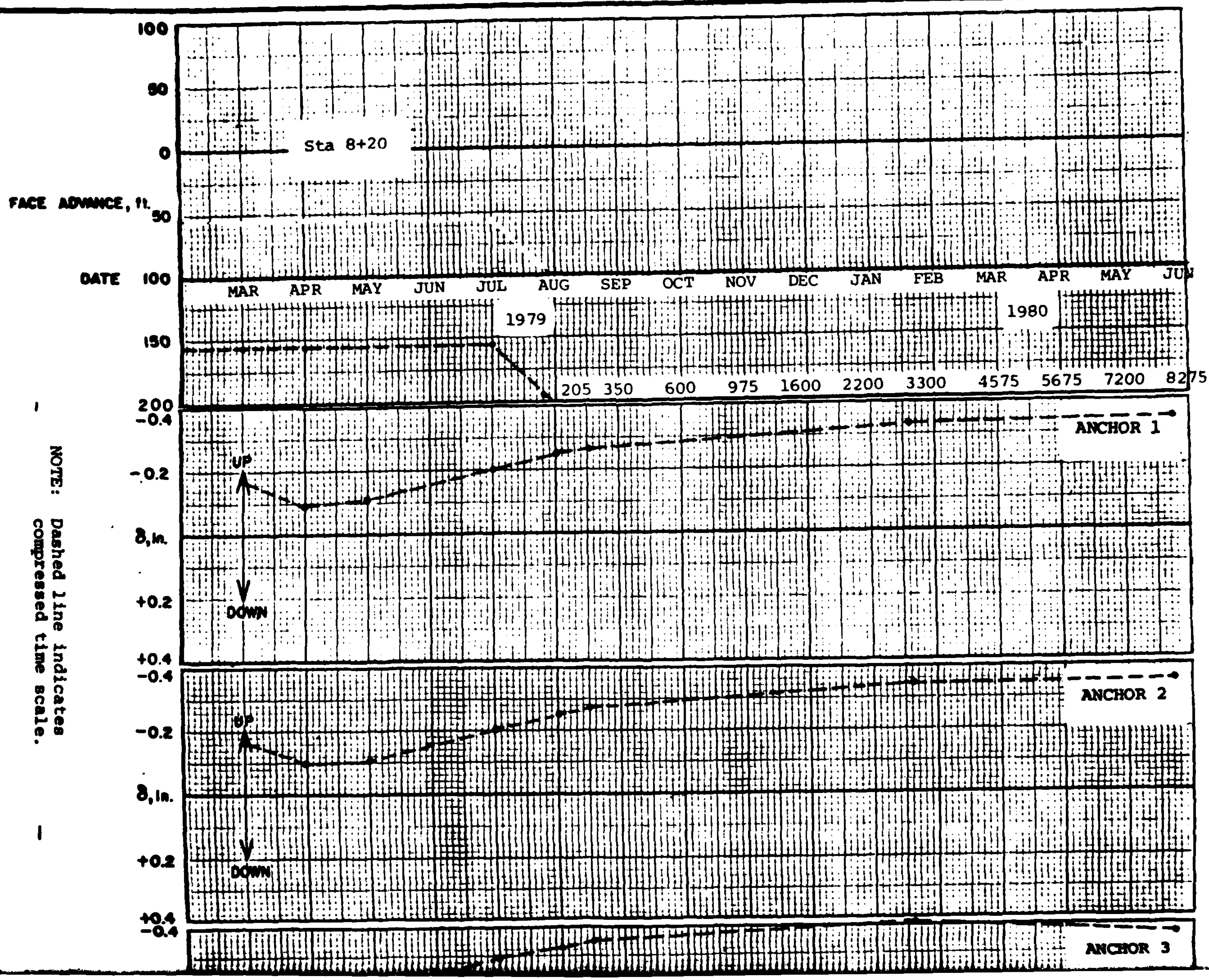
PHOTO #12 - Biaxial cell, borehole gage
and hydraulic pump.

APPENDIX A

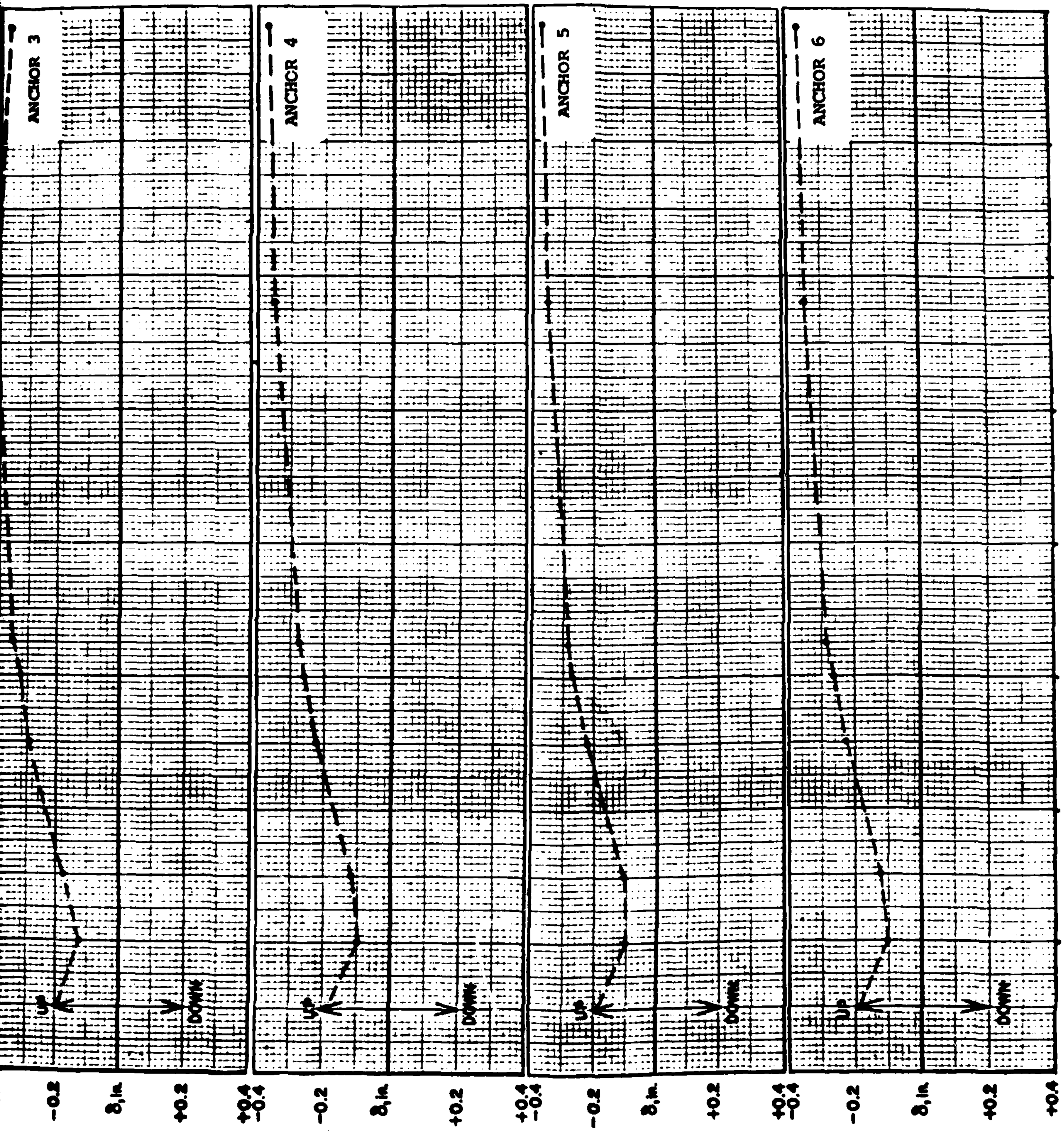

FACE ADVANCE, ft.
100
50
0
50
100
150
200
Sta 8+20
Begin Removing Bench
NOV. 1978
DEC.
JAN. 1979
FEB.
Anchor 1
-0.4
-0.2
δ, in.
+0.2
+0.4
UP
DOWN
Anchor 2
-0.4
-0.2
δ, in.
+0.2
+0.4
UP
DOWN
Anchor 3
-0.4
-0.2
UP



ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX-1 ANCHOR MOVEMENT RELATIVE TO REFERENCE PLATE TS.1 STA 08+20
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG A-1e



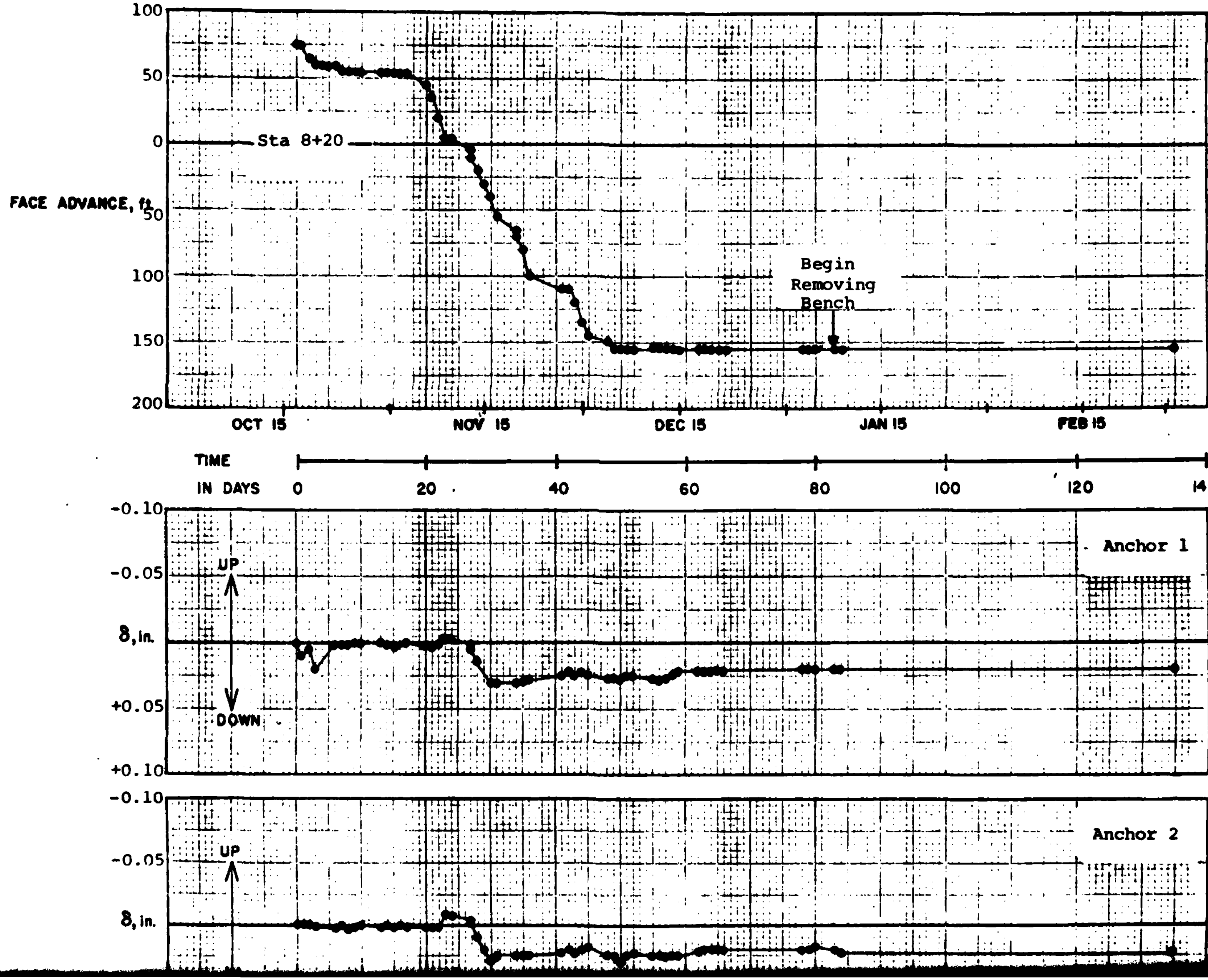
NOTE: Dashed line indicates compressed time scale.



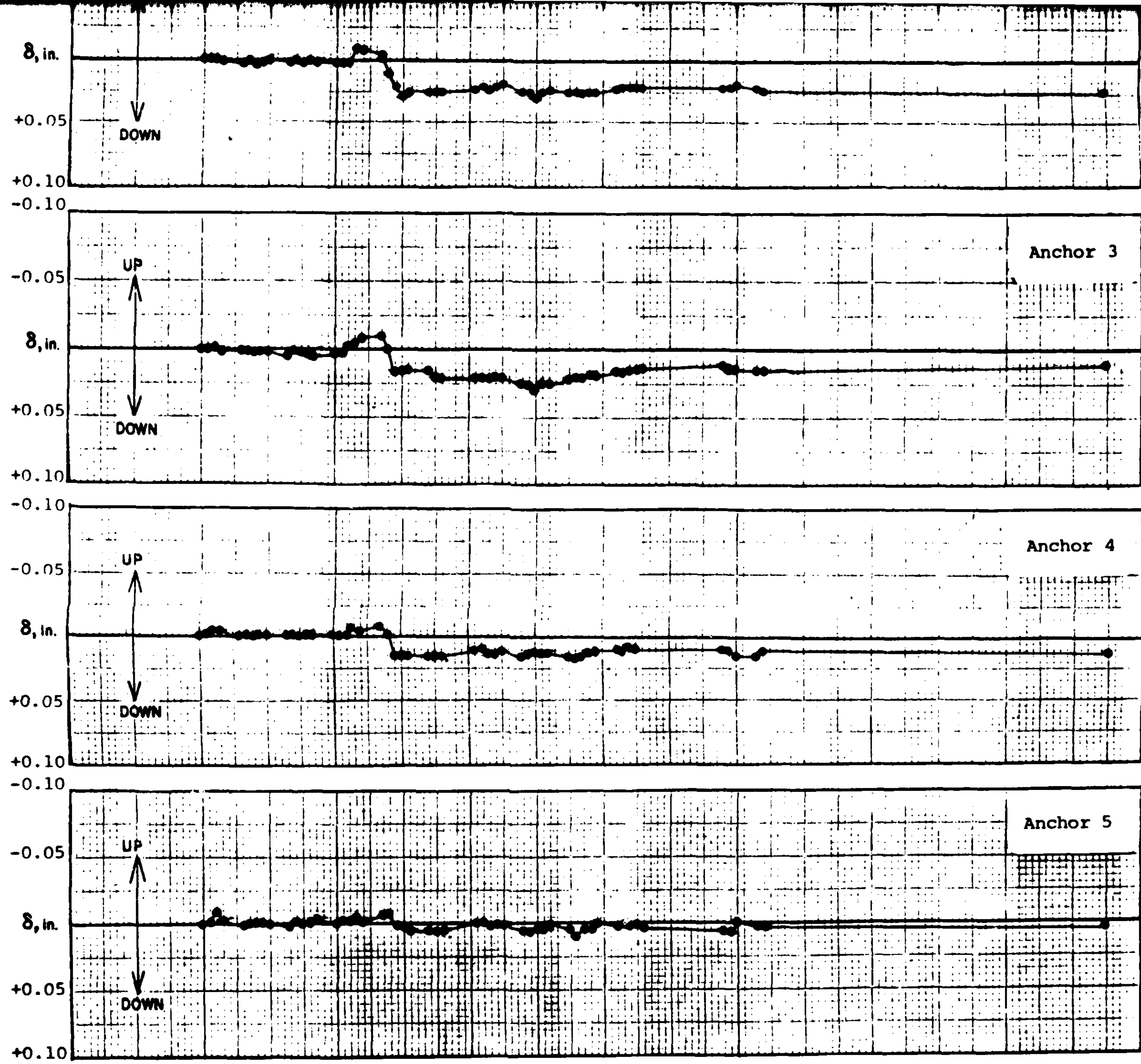
ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 1 ANCHOR MOVEMENT RELATIVE TO REFERENCE PL. TS I · STA 08 + 20
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A1e

2

(Continued)



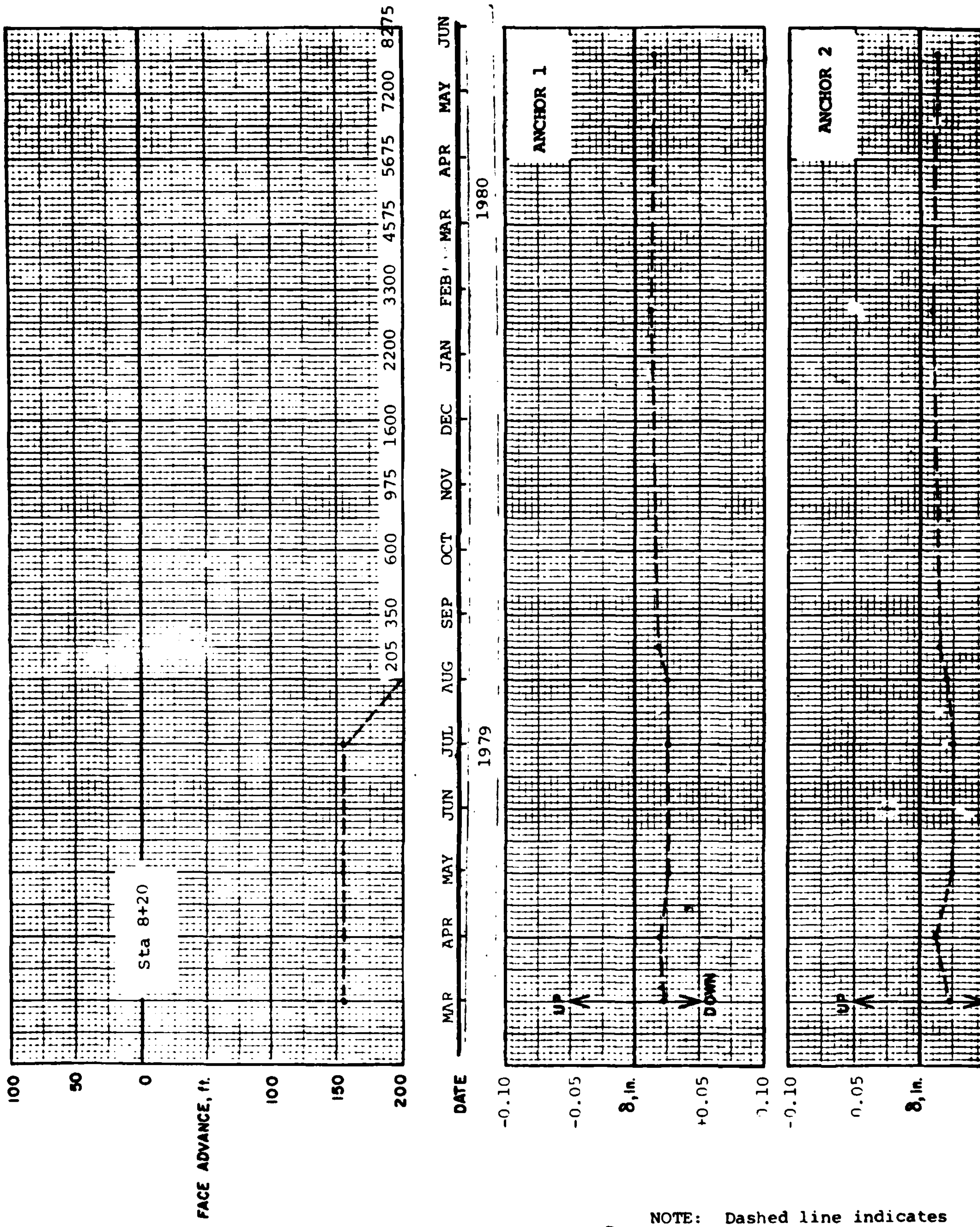
FACE ADVANCE, ft
100
50
0
50
100
150
200
Sta 8+20
Begin
Removing
Bench
OCT 15
NOV 15
DEC 15
JAN 15
FEB 15
TIME
IN DAYS
0
20
40
60
80
100
120
140
Anchor 1
-0.10
-0.05
δ, in.
+0.05
+0.10
UP
DOWN
Anchor 2
-0.10
-0.05
δ, in.
UP



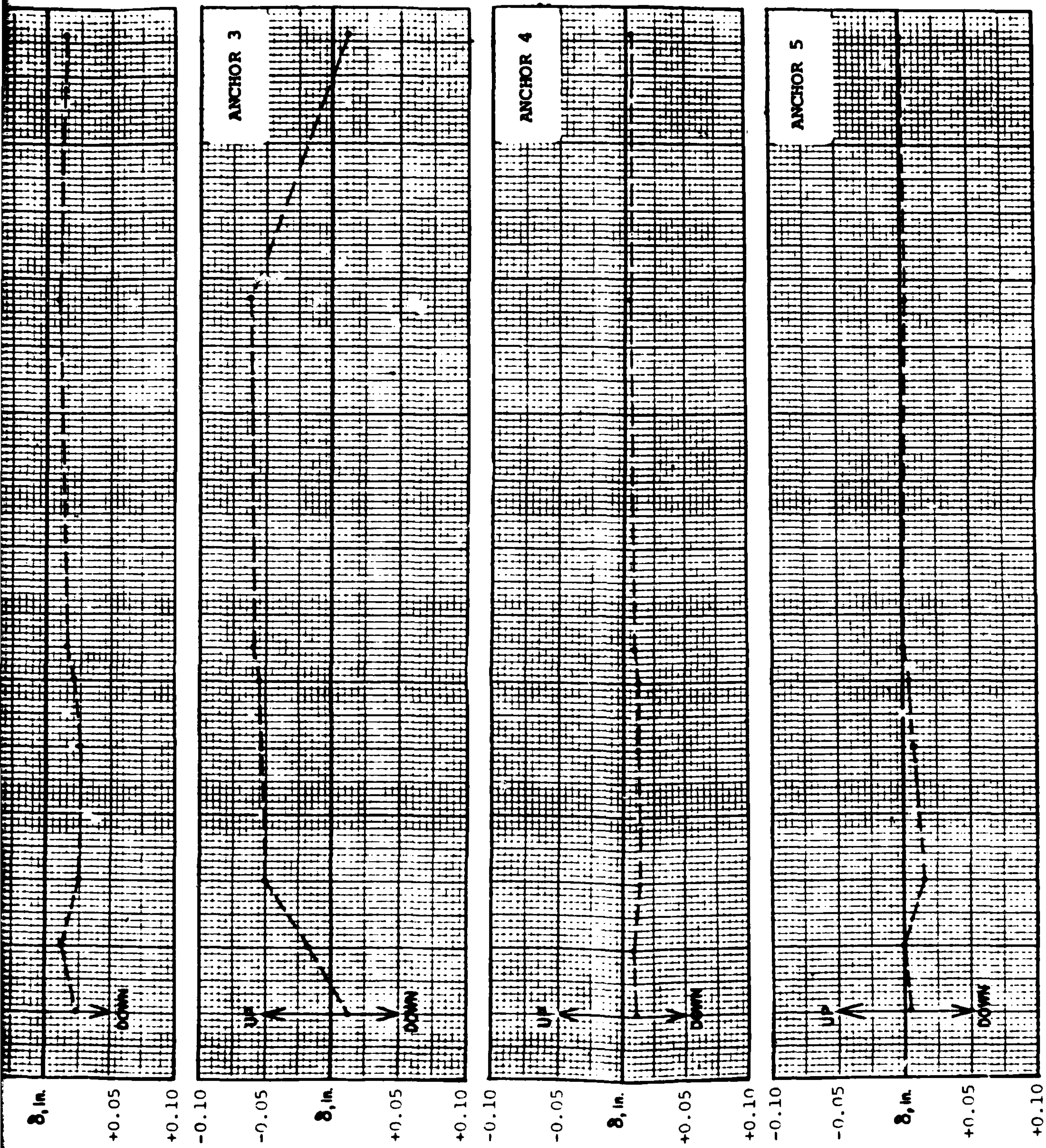
NOTE: δ = ANCHOR DEFLECTION RELATIVE TO ANCHOR 6.

ROGER J. AU & SON, INC, MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX-1 ANCHOR MOVEMENT RELATIVE TO REFERENCE PLATE TSI STA 8+20
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-1b

2



NOTE: Dashed line indicates compressed time scale.



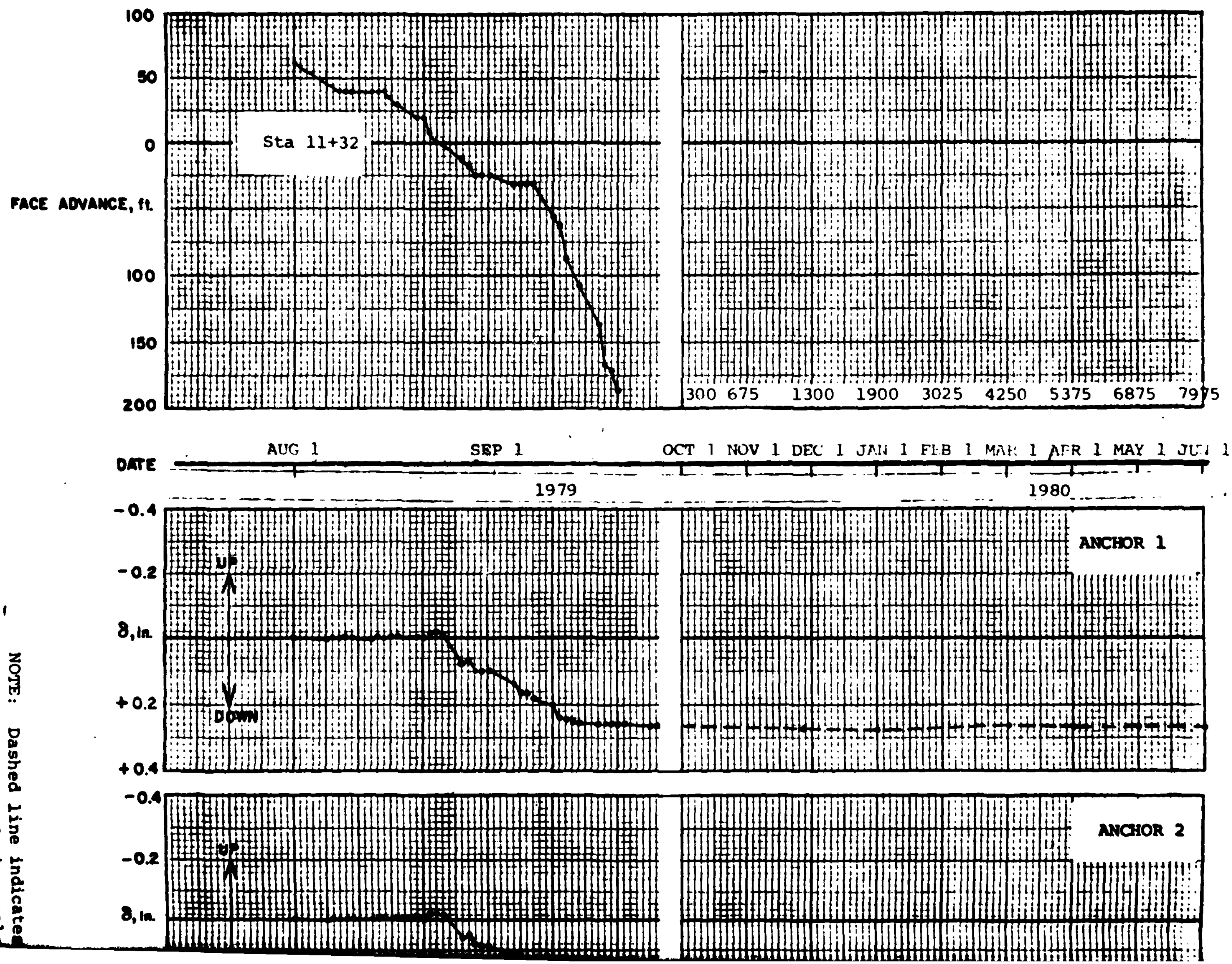
NOTE: δ = ANCHOR DEFLECTION RELATIVE TO ANCHOR 6.

icates
scale.

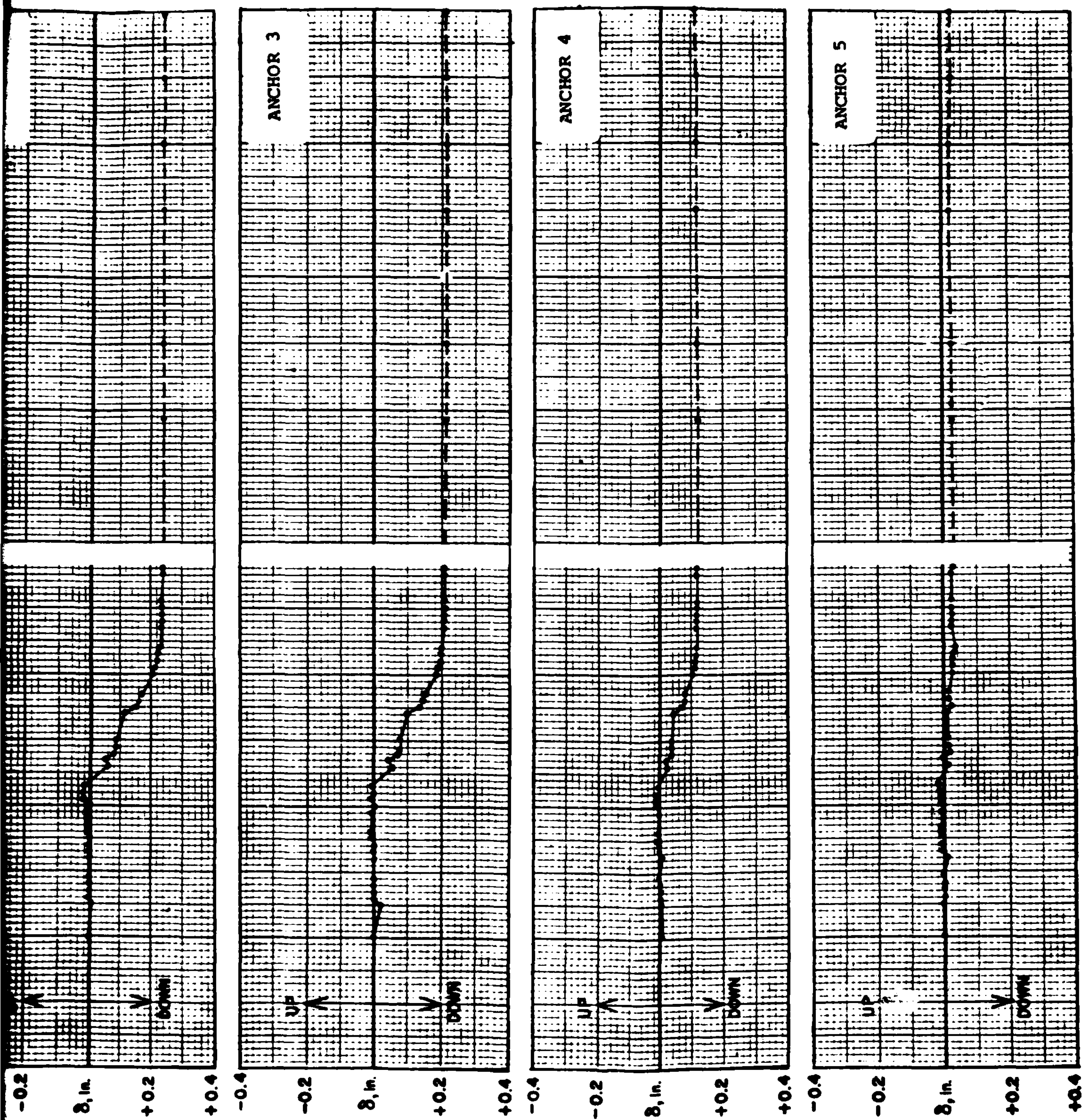
ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 1 ANCHOR MOVEMENT RELATIVE TO ANCHOR NO. 6 TS I STA 08 + 20
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-1b

(Continued)

2



NOTE: Dashed line indicate[s]
compressed time scal[e]

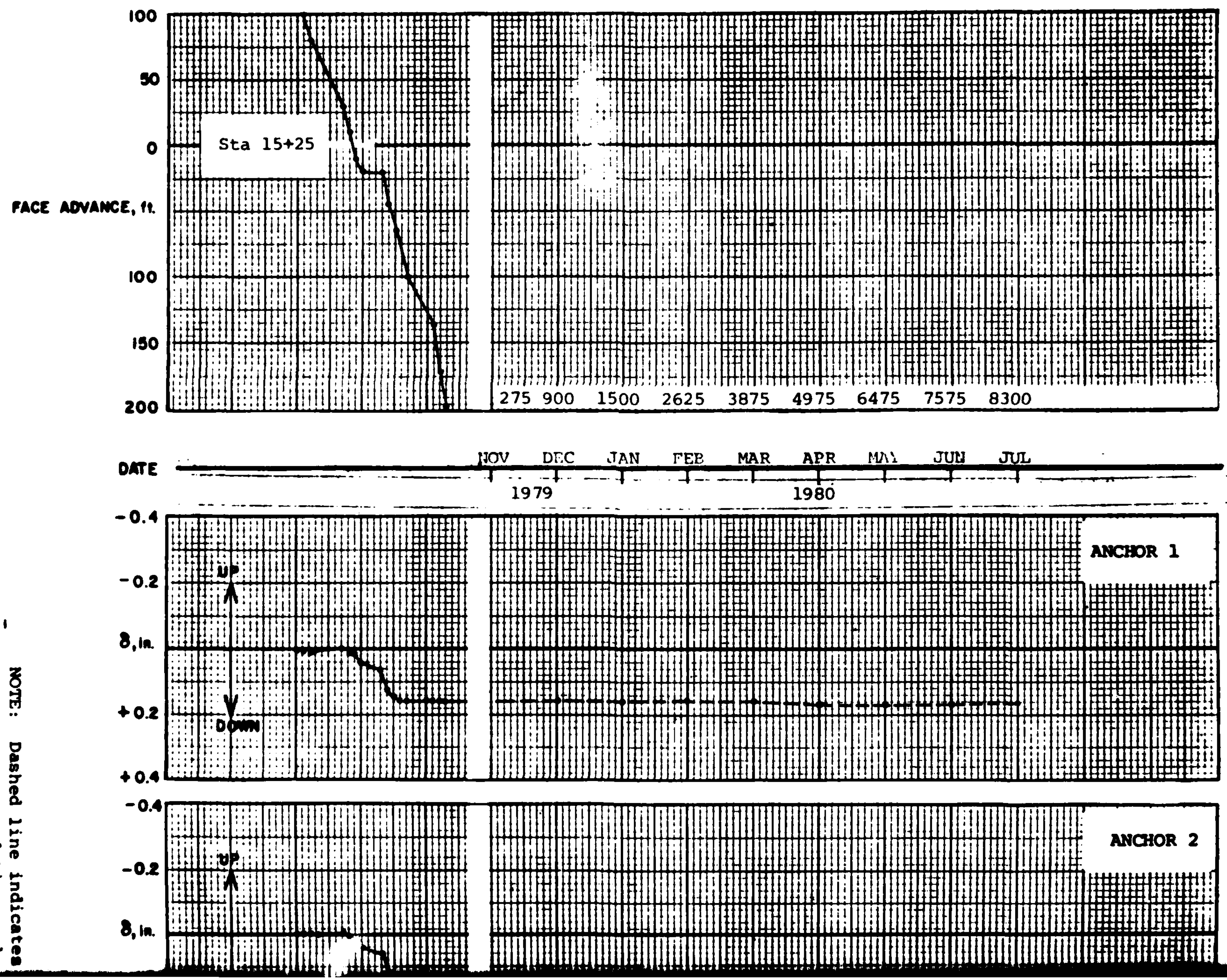


NOTE: δ = ANCHOR DEFLECTION RELATIVE TO ANCHOR 6.

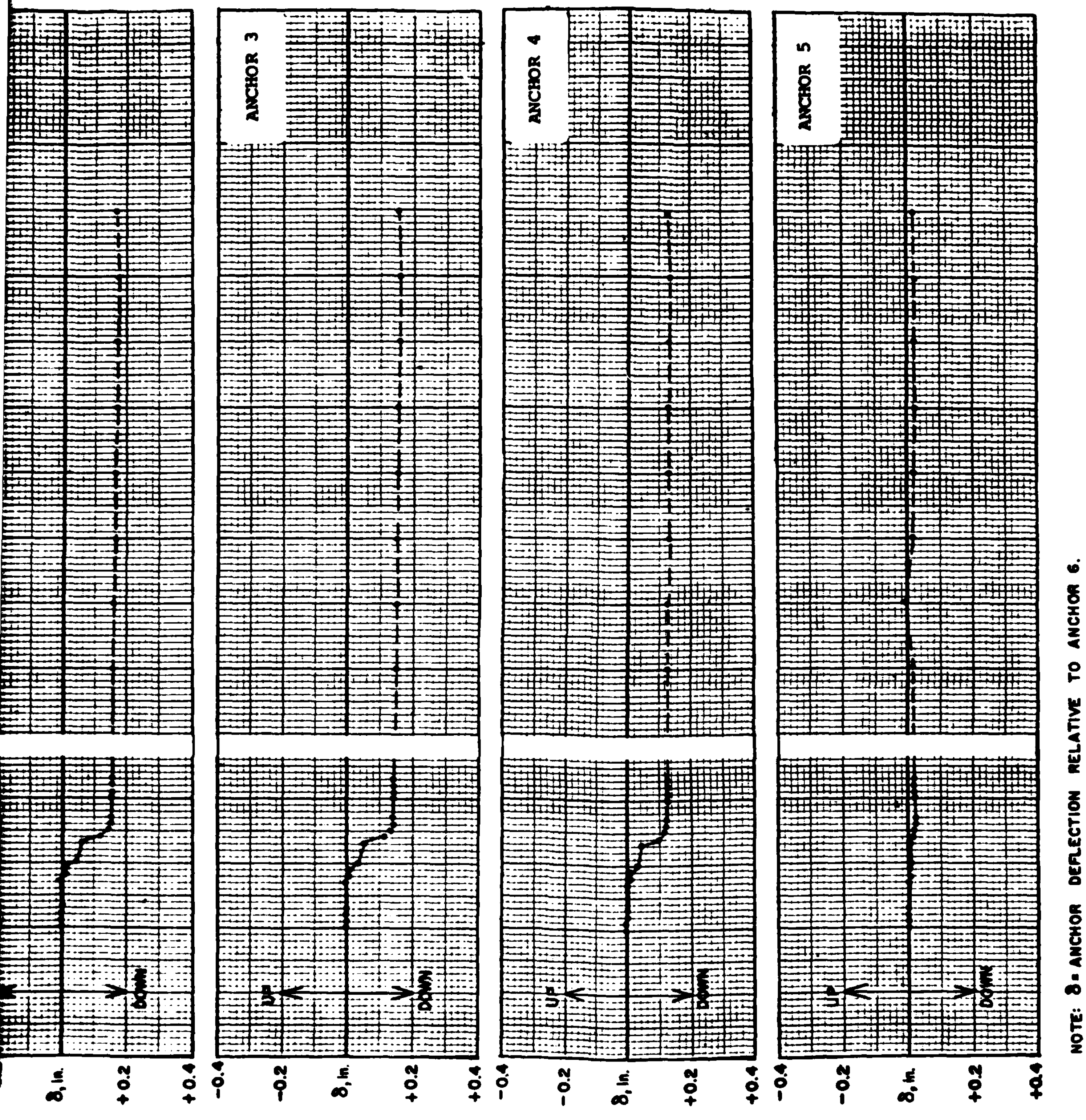
indicates
ame scale.

ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 2 ANCHOR MOVEMENT RELATIVE TO ANCHOR NO.6 TS 2 STA 11+32
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-2

2



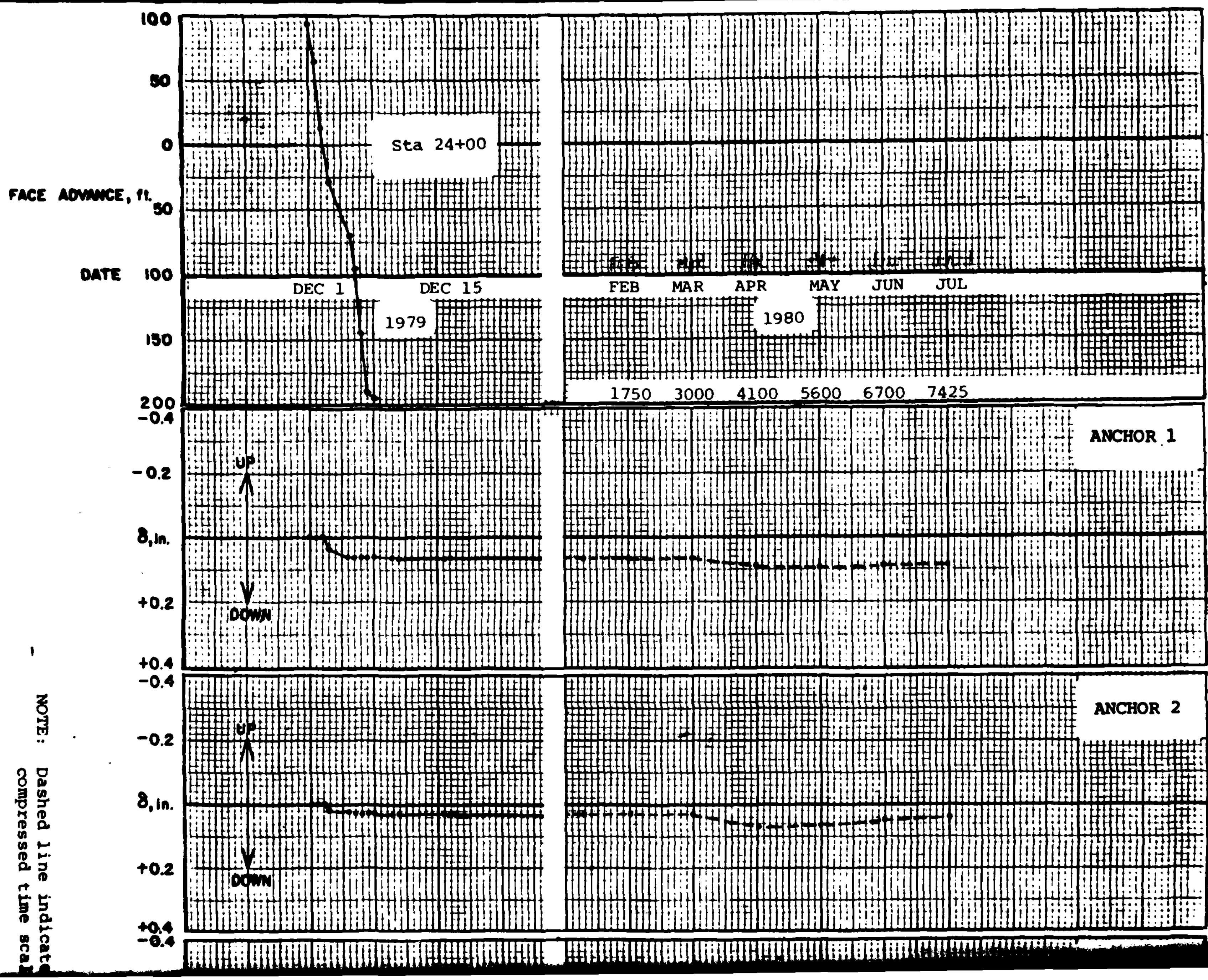
NOTE: Dashed line indicates
compressed time scale.



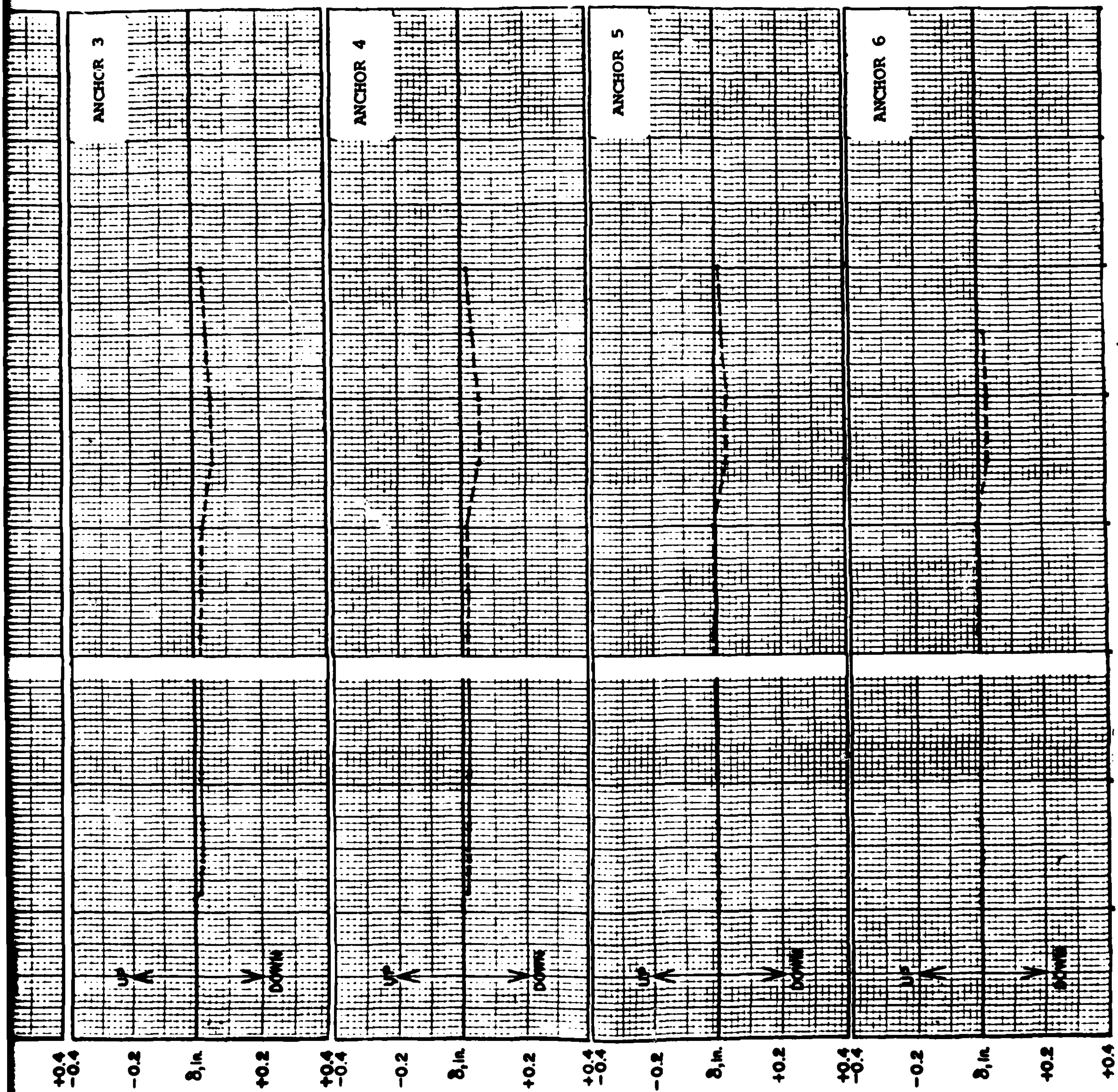
indicates
ime scale.

ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX- 3 ANCHOR MOVEMENT RELATIVE TO ANCHOR NO. 6 TS 3 STA. 15+25
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-3

2



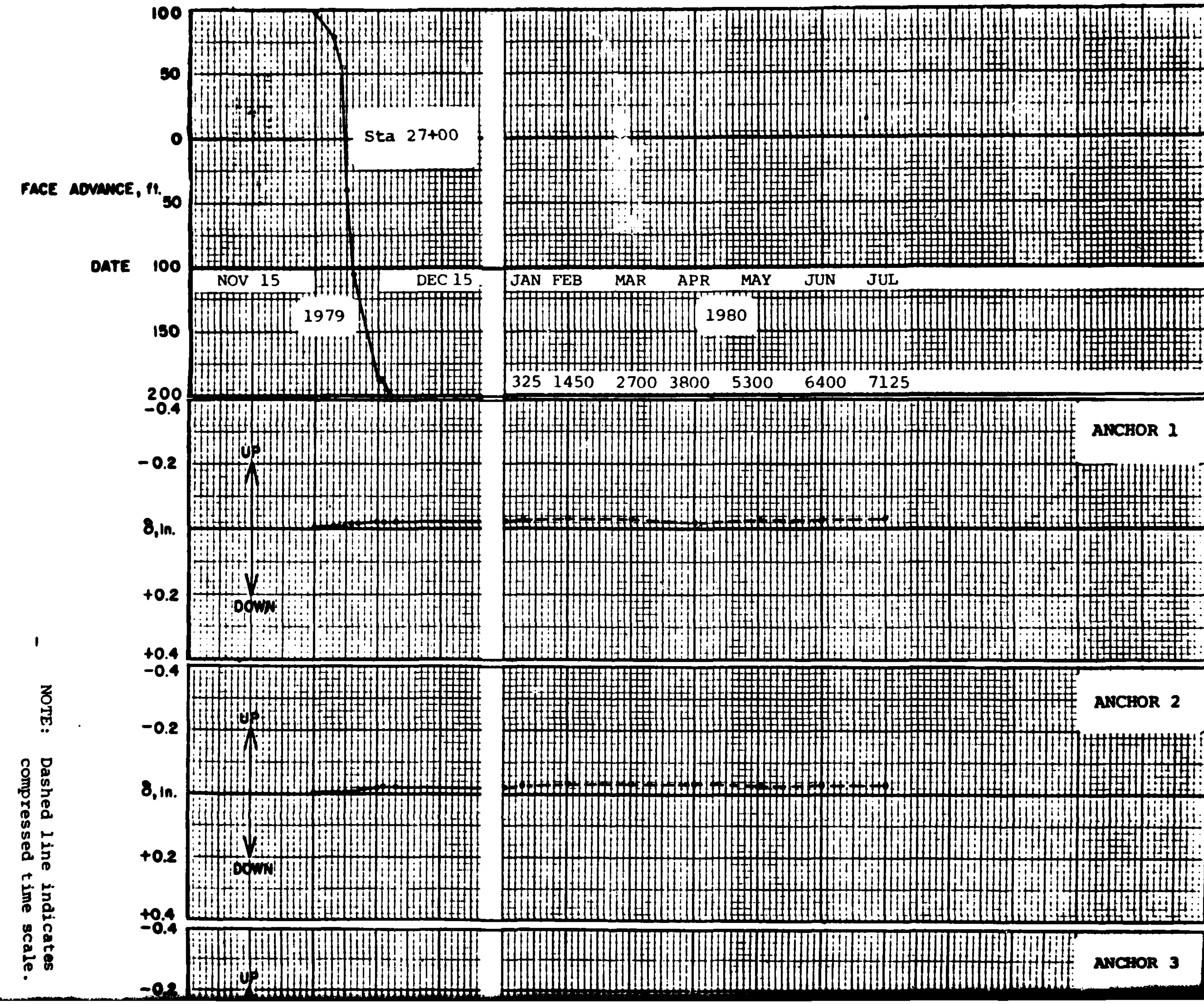
NOTE: Dashed line indicat
compressed time scal



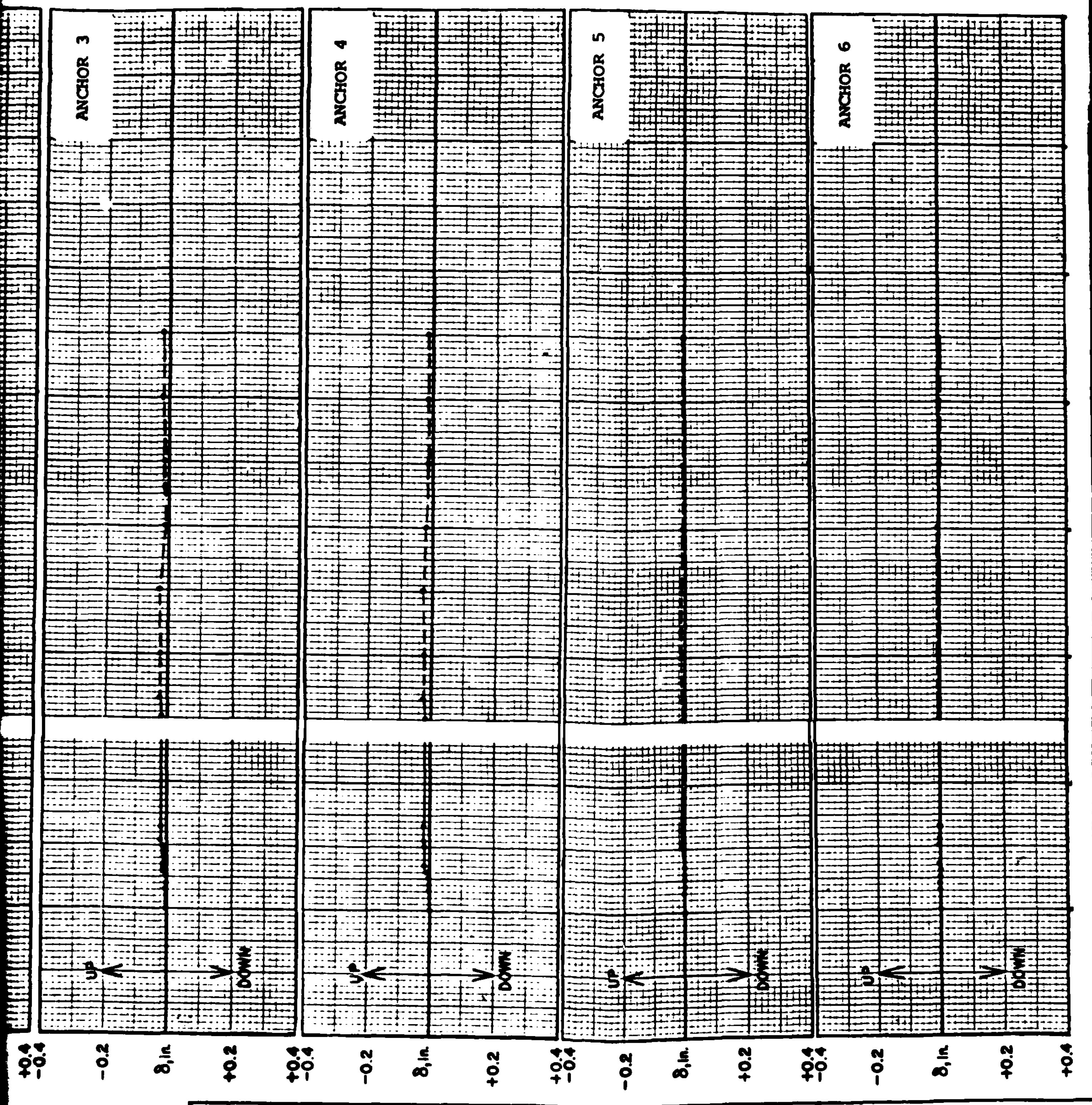
indicates
ime scale.

ROGER J. AU & SON, INC. MANSFIELD, OHIO GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PARK RIVER AUXILIARY TUNNEL PROJECT 77382	MPBX - 4 ANCHOR MOVEMENT RELATIVE TO REFERENCE PL. TS 4 , STA. 24 + 00 JULY 1980 FIG. A-4

2



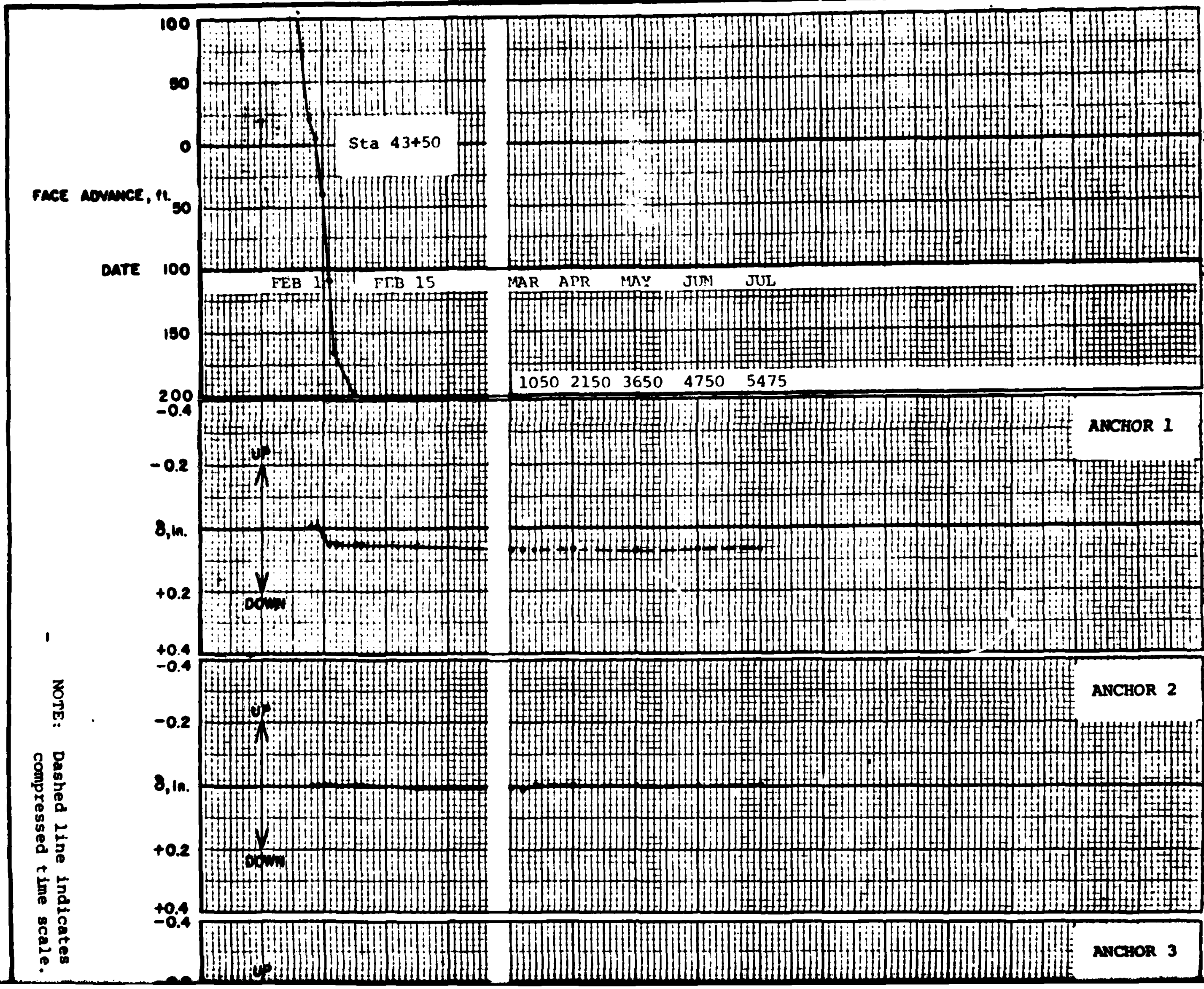
NOTE: Dashed line indicates compressed time scale.



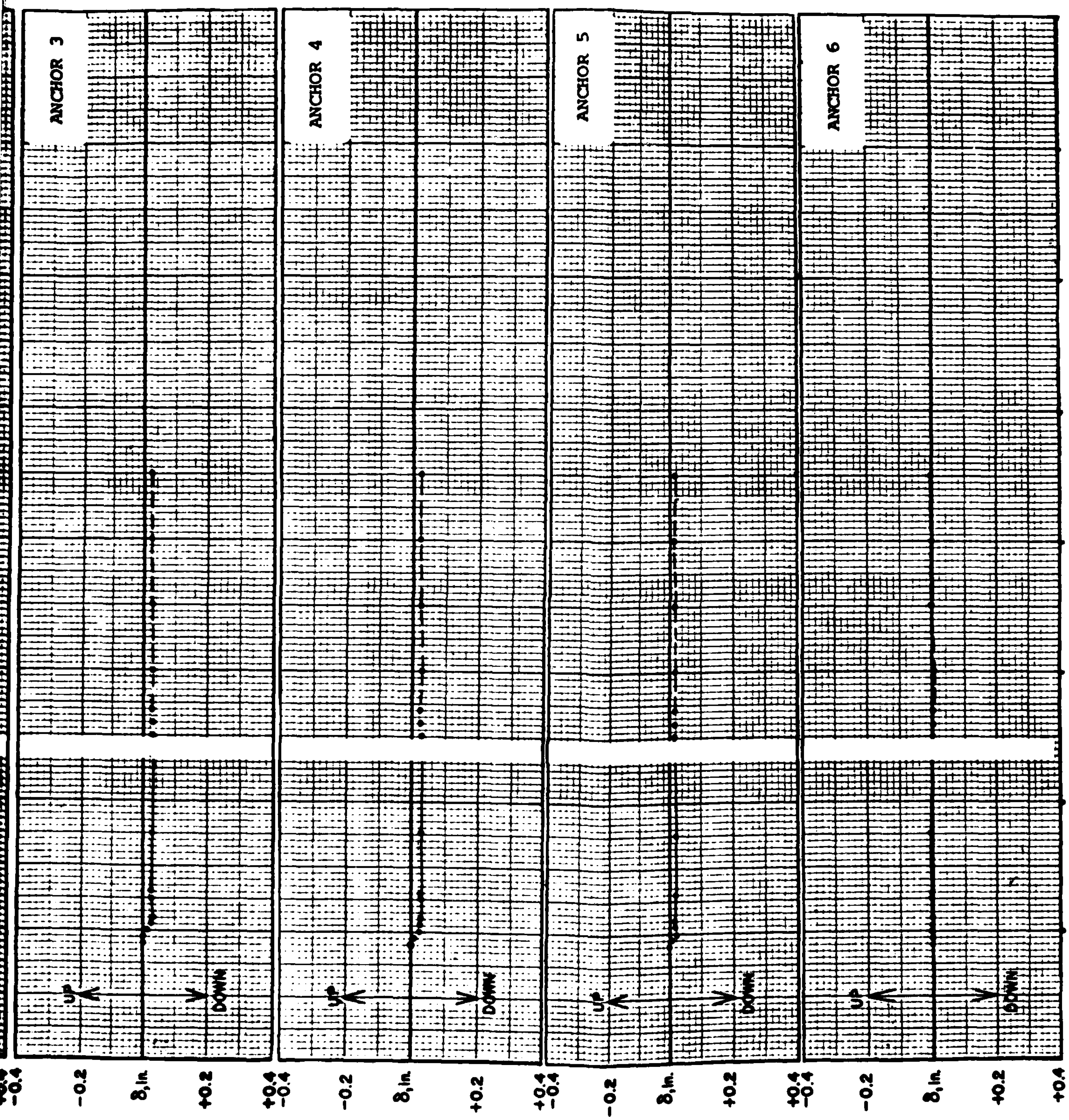
dicates
scale.

ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 5 ANCHOR MOVEMENT RELATIVE TO REFERENCE PL. TS 5 - STA 27 +00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-5

2



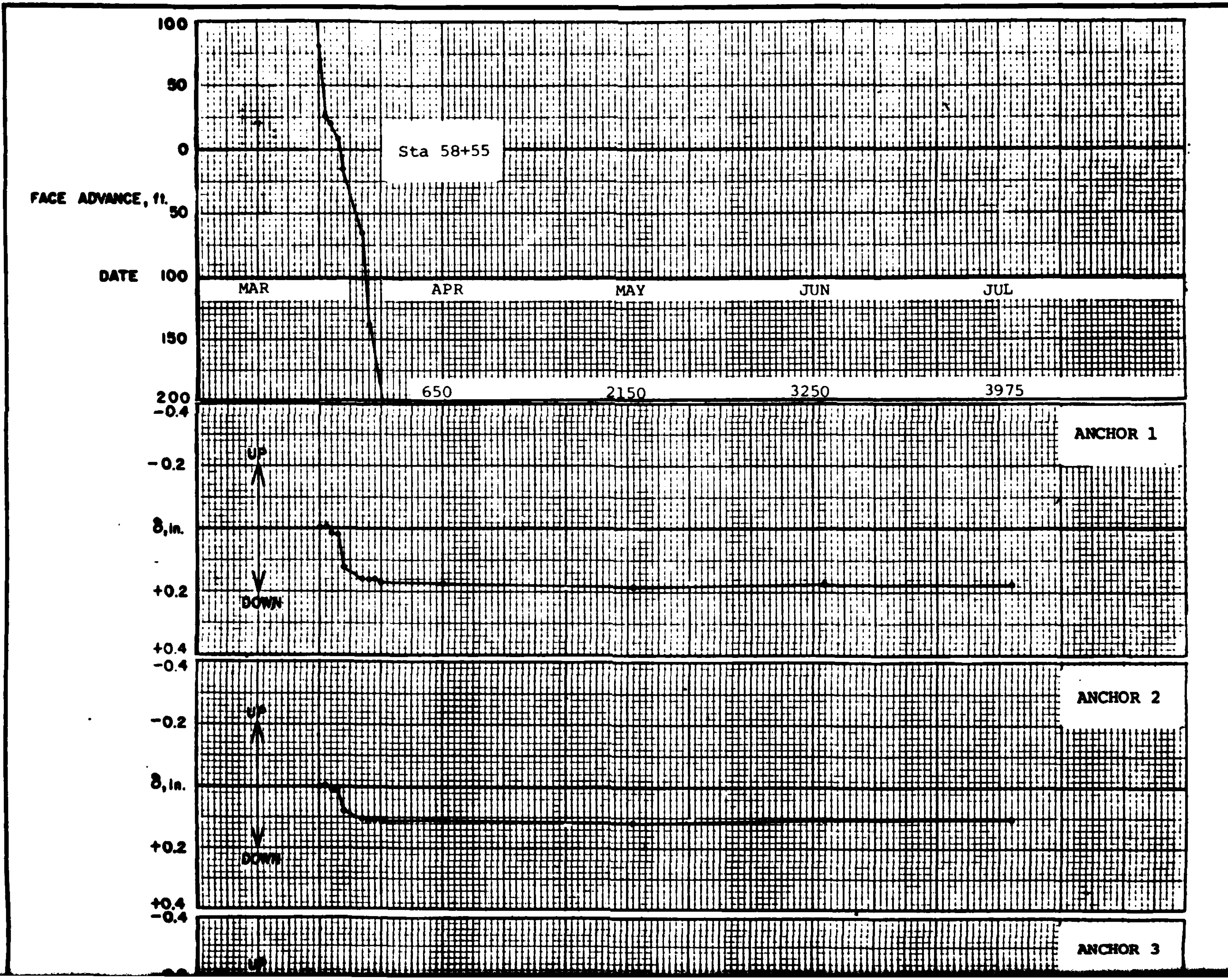
NOTE: Dashed line indicates compressed time scale.



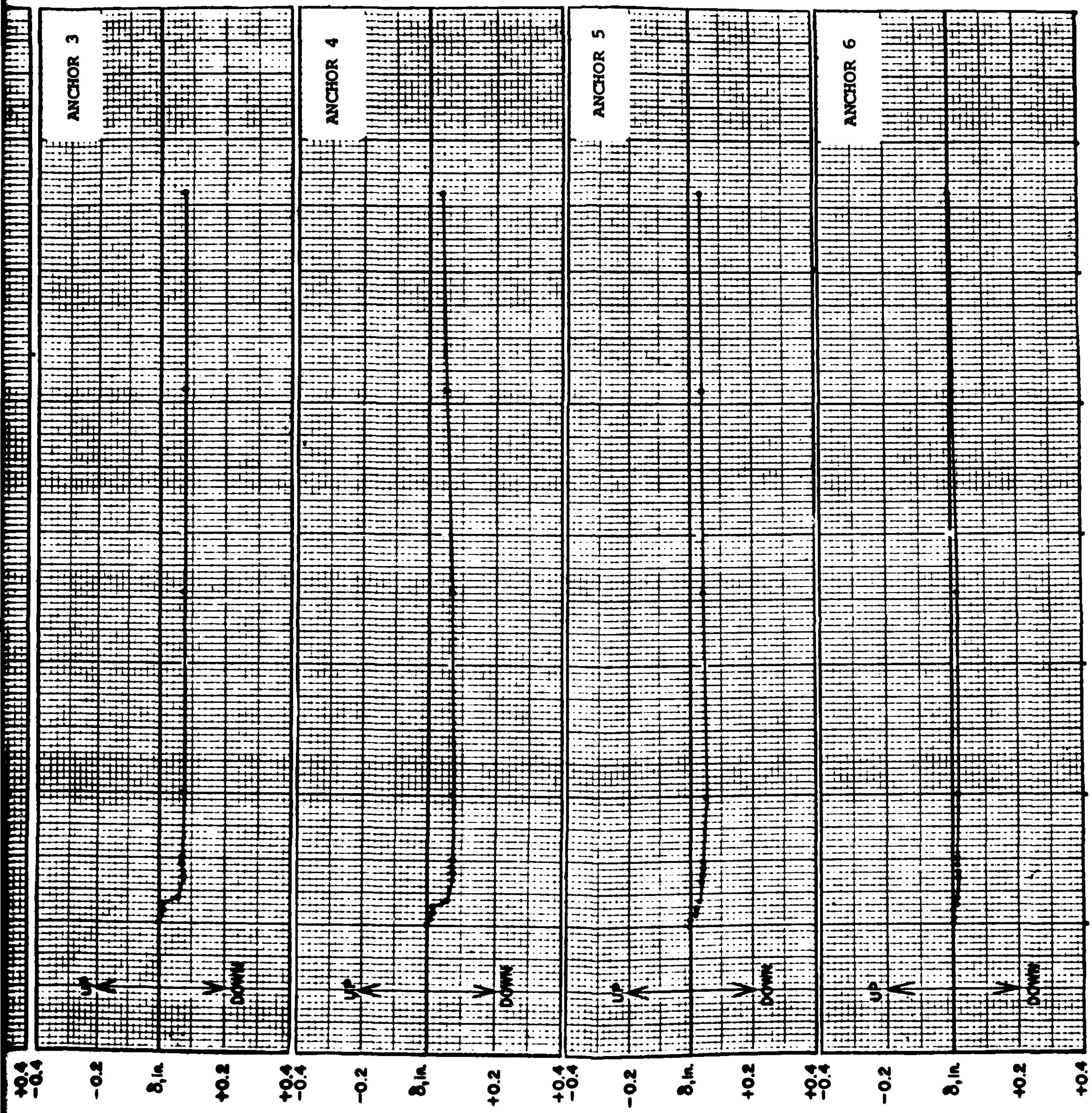
licates
) scale.

ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 6 ANCHOR MOVEMENT RELATIVE TO REFERENCE PL TS 6 STA. 43+50
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. ?-6

2

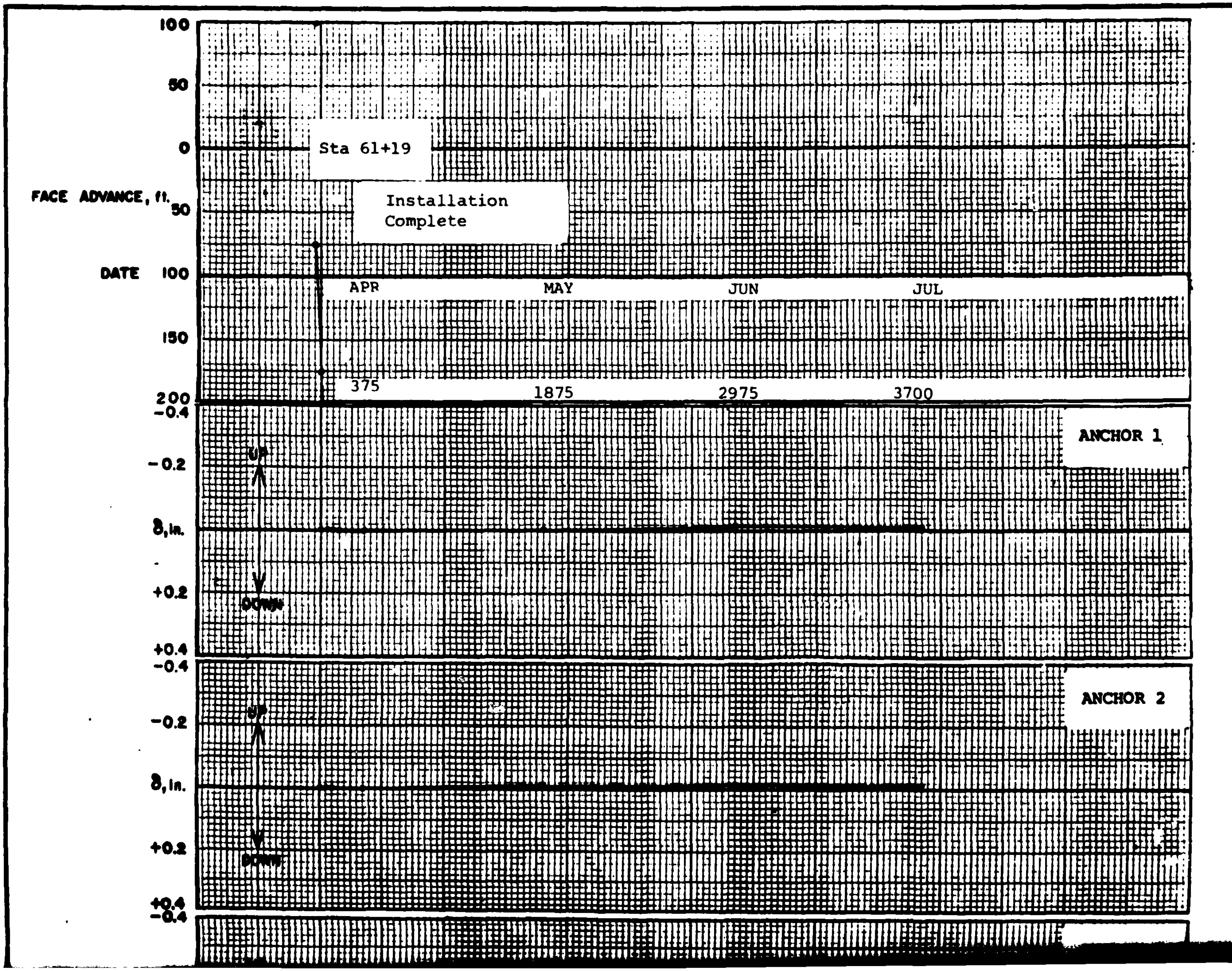


FACE ADVANCE, ft.
100
50
0
50
100
150
200
Sta 58+55
DATE
MAR
APR
MAY
JUN
JUL
650
2150
3250
3975
ANCHOR 1
-0.4
-0.2
δ, in.
+0.2
+0.4
UP
DOWN
ANCHOR 2
-0.4
-0.2
δ, in.
+0.2
+0.4
UP
DOWN
ANCHOR 3
-0.4
UP

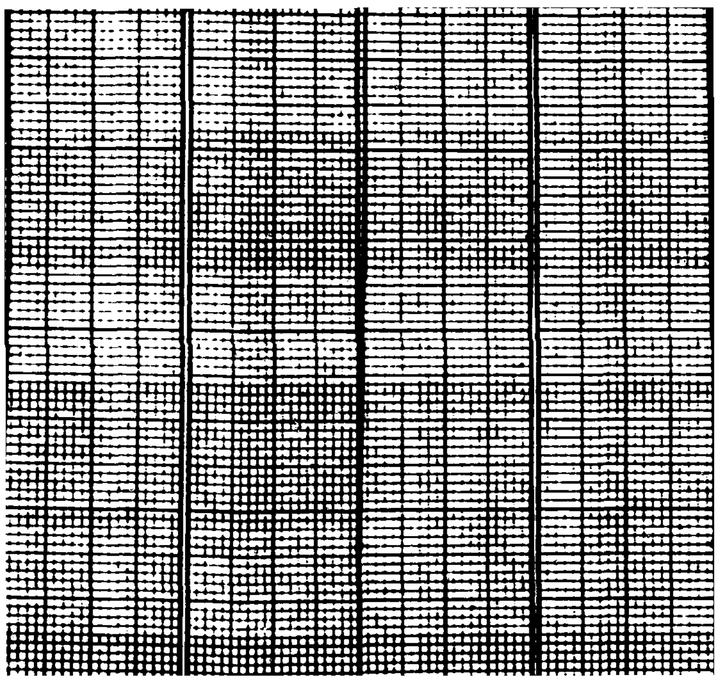


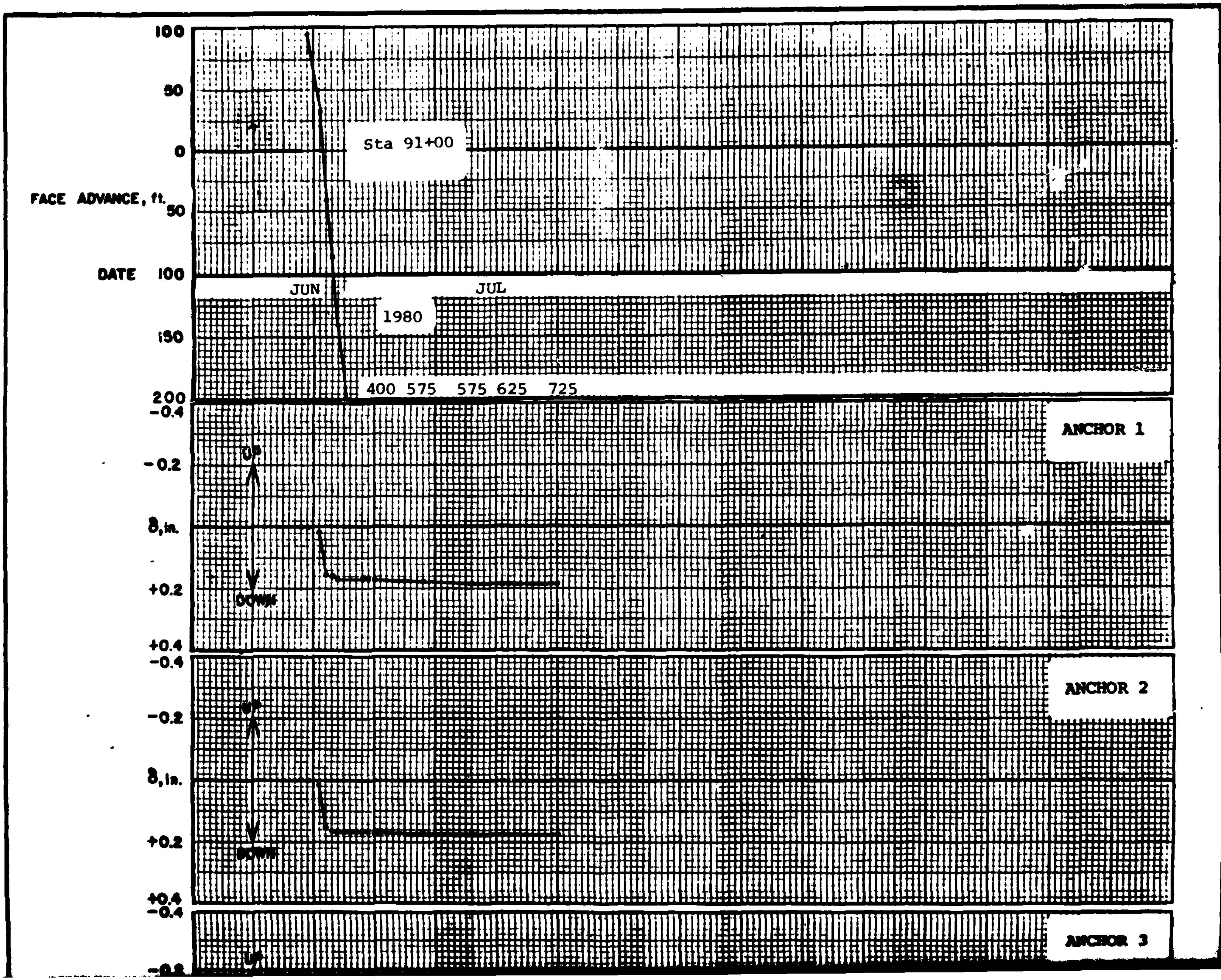
ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 7 ANCHOR MOVEMENT RELATIVE TO REFERENCE PL. TS 7 · STA 58 + 55
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-7

2

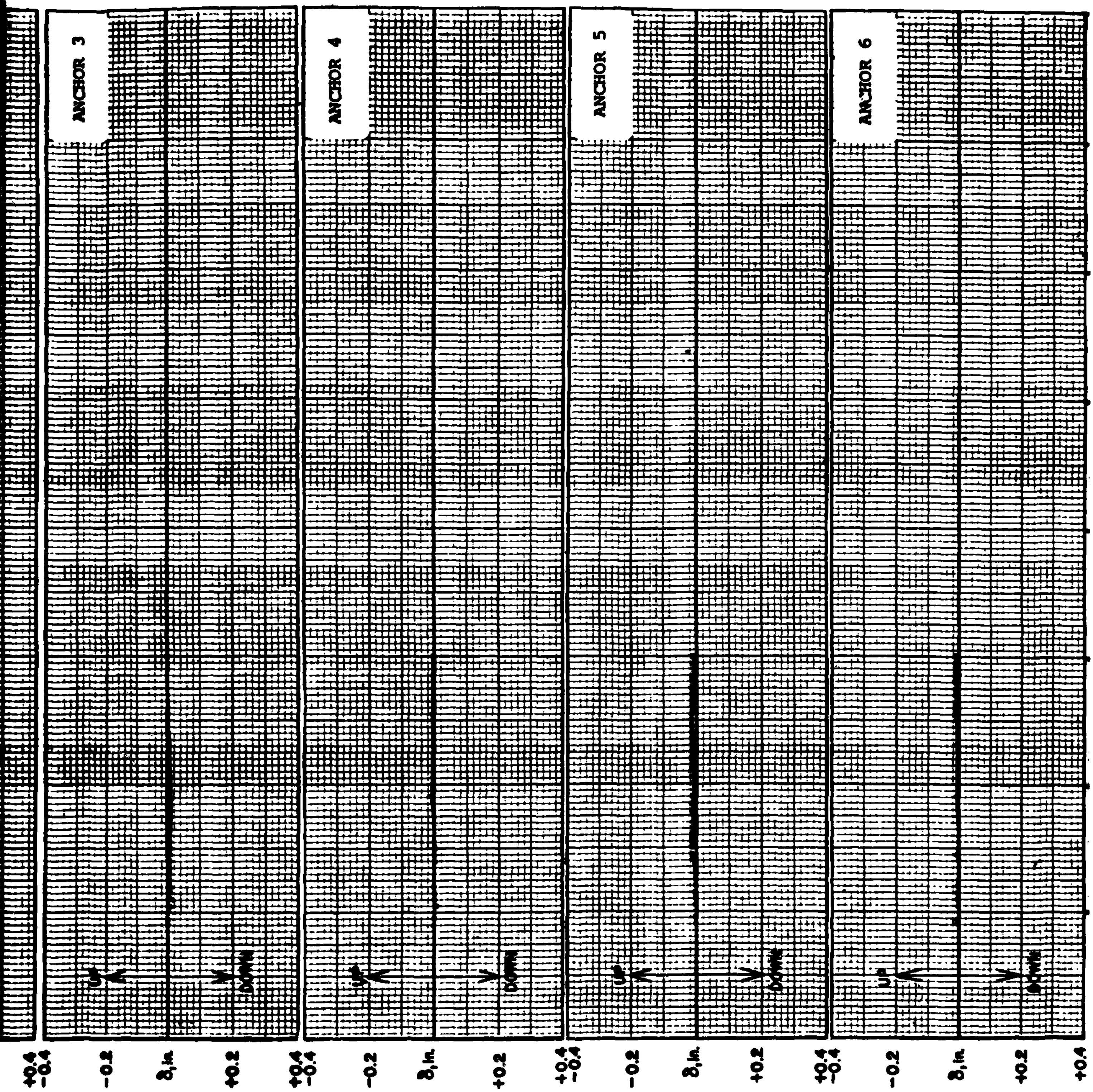


FACE ADVANCE, ft.
100
50
0
50
100
150
200
DATE
Sta 61+19
Installation
Complete
APR
MAY
JUN
JUL
375
1875
2975
3700
ANCHOR 1
-0.4
-0.2
δ, in.
+0.2
+0.4
UP
DOWN
ANCHOR 2
-0.4
-0.2
δ, in.
+0.2
+0.4
-0.4



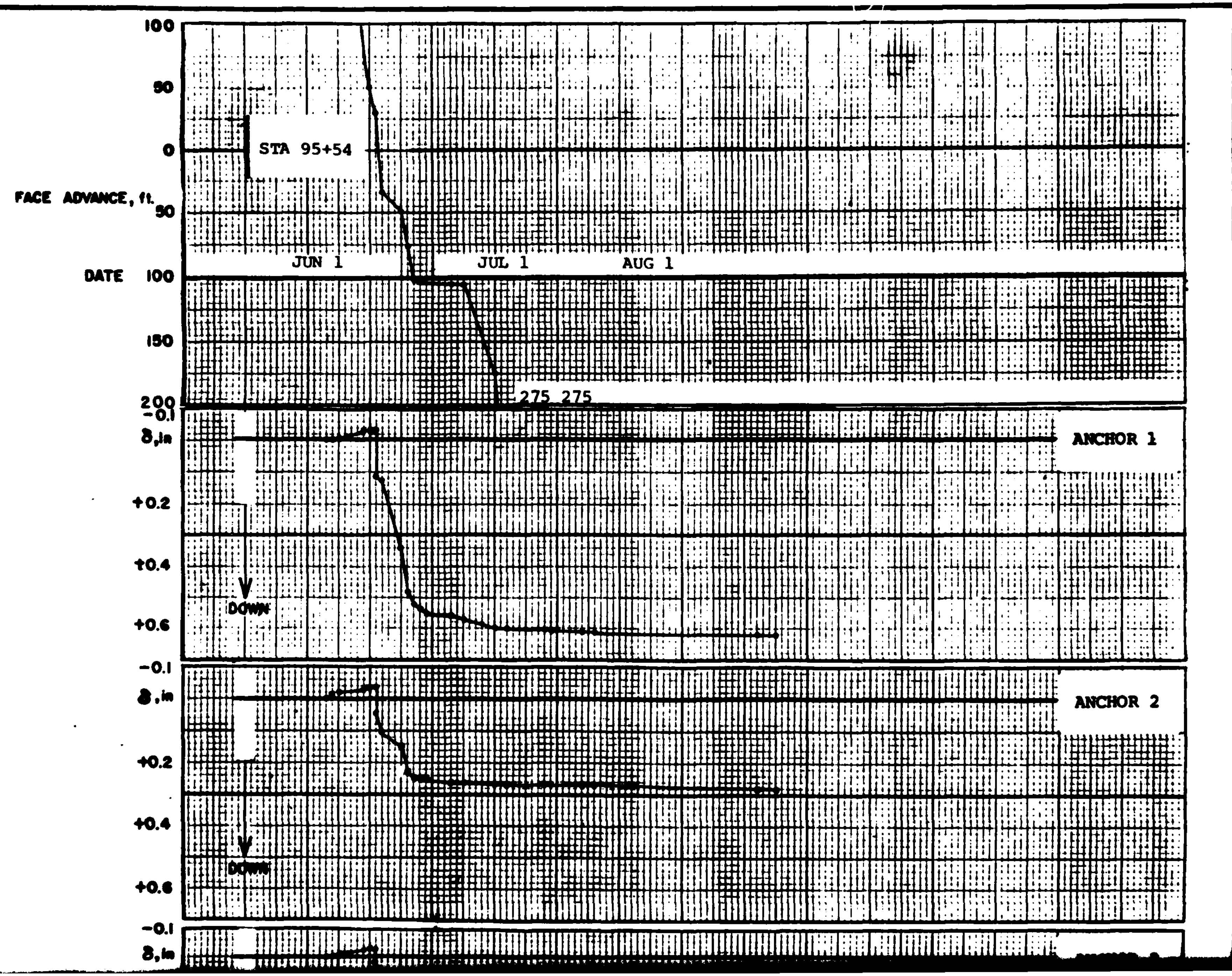


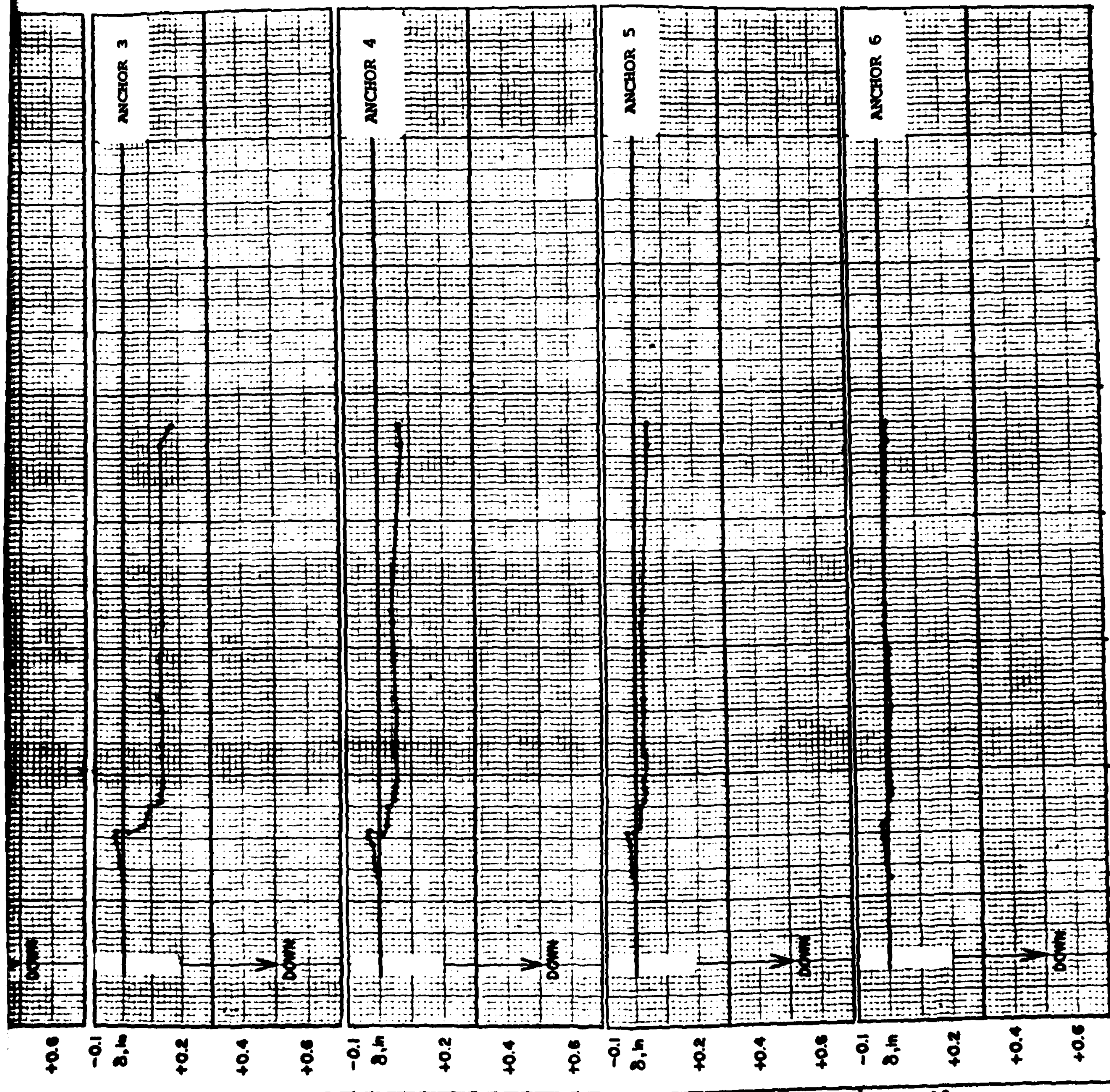
FACE ADVANCE, ft.
100
50
0
50
100
150
200
DATE
JUN
JUL
1980
Sta 91+00
400 575 575 625 725
ANCHOR 1
-0.4
-0.2
δ, in.
+0.2
+0.4
UP
DOWN
ANCHOR 2
-0.4
-0.2
δ, in.
+0.2
+0.4
UP
DOWN
ANCHOR 3
-0.4
UP



ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 9 ANCHOR MOVEMENT RELATIVE TO REFERENCE PL. TS 9 STA 91+00
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-9

2


FACE ADVANCE, ft
DATE
100
50
0
50
100
150
200
STA 95+54
JUN 1
JUL 1
AUG 1
275 275
-0.1
δ, in
+0.2
+0.4
+0.6
DOWN
ANCHOR 1
-0.1
δ, in
+0.2
+0.4
+0.6
DOWN
ANCHOR 2
-0.1
δ, in



ROGER J. AU & SON, INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	MPBX - 10 ANCHOR MOVEMENT RELATIVE TO REFERENCE PL TS10 ⊥ STA 95+54
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. A-10

2

APPENDIX B

ROCK BOLT LOAD CELL DATA
Roger J. Au & Son, Inc

LEGEND FOR FIGURES B-1 THROUGH B-6

Face Advance

1 - Beginning of bench removal
2 - Bench removed to Sta 9+20

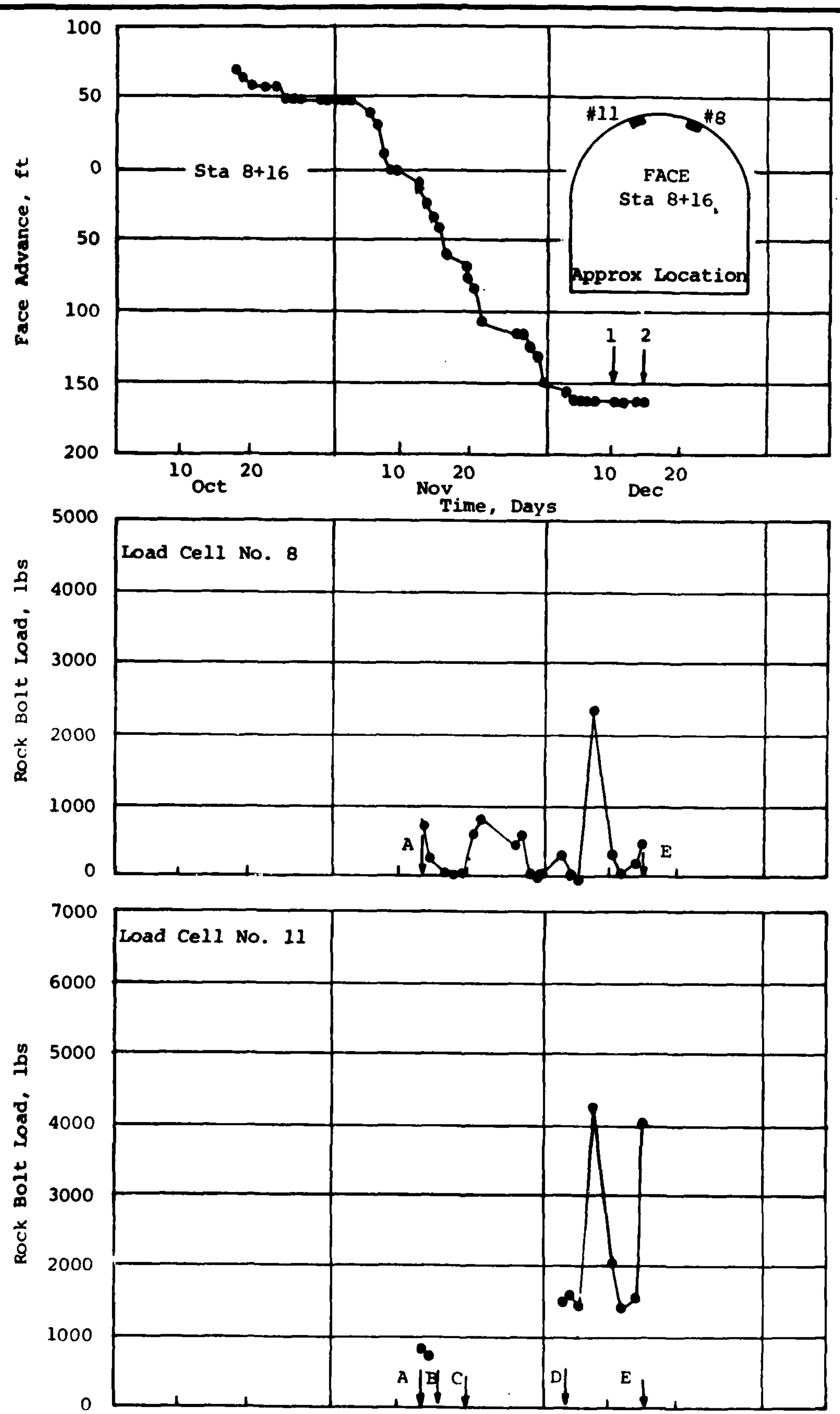
Load Cell Monitoring

A - Initial installation of cell with lockoff load
B - Cell damaged by blasting and flyrock
C - Cell temporarily removed for repair
D - Cell reinstalled with new lockoff load
E - Cell removed before shotcreteing
F - Rock against which cells bears were blown away during blast

Arrow indicates time of event.

Geotechnical Engineers Inc.

Project 77382
August 1980



ROCK BOLT LOAD DATA
CELL NOS. 8 AND 11

Fig. B-1

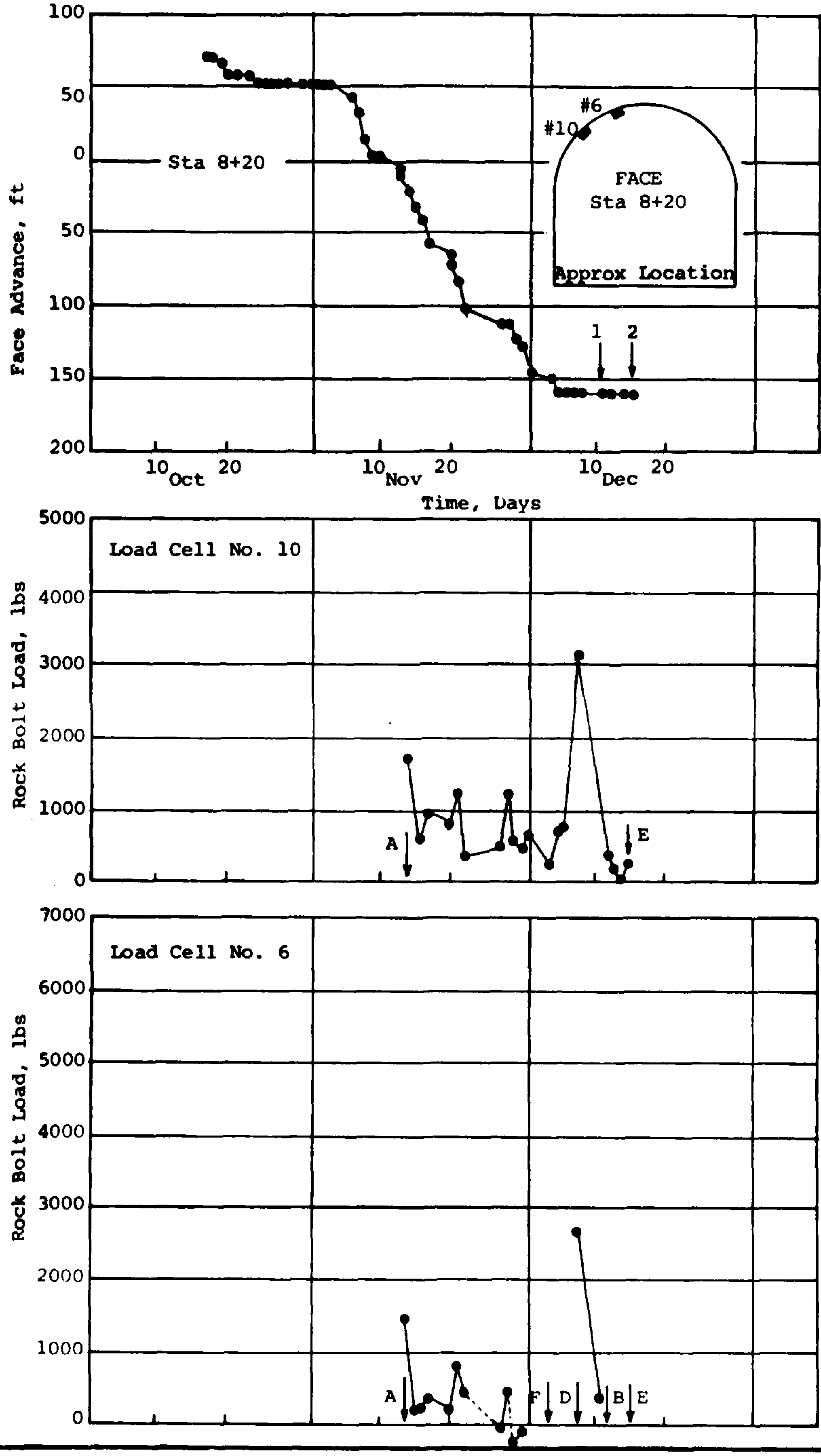
Park River Auxiliary
Tunnel
Hartford, CT

August 1980

Project 77382

Roger & Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS



ROCK BOLT LOAD DATA
CELL NOS. 10 AND 6

Fig. B-2

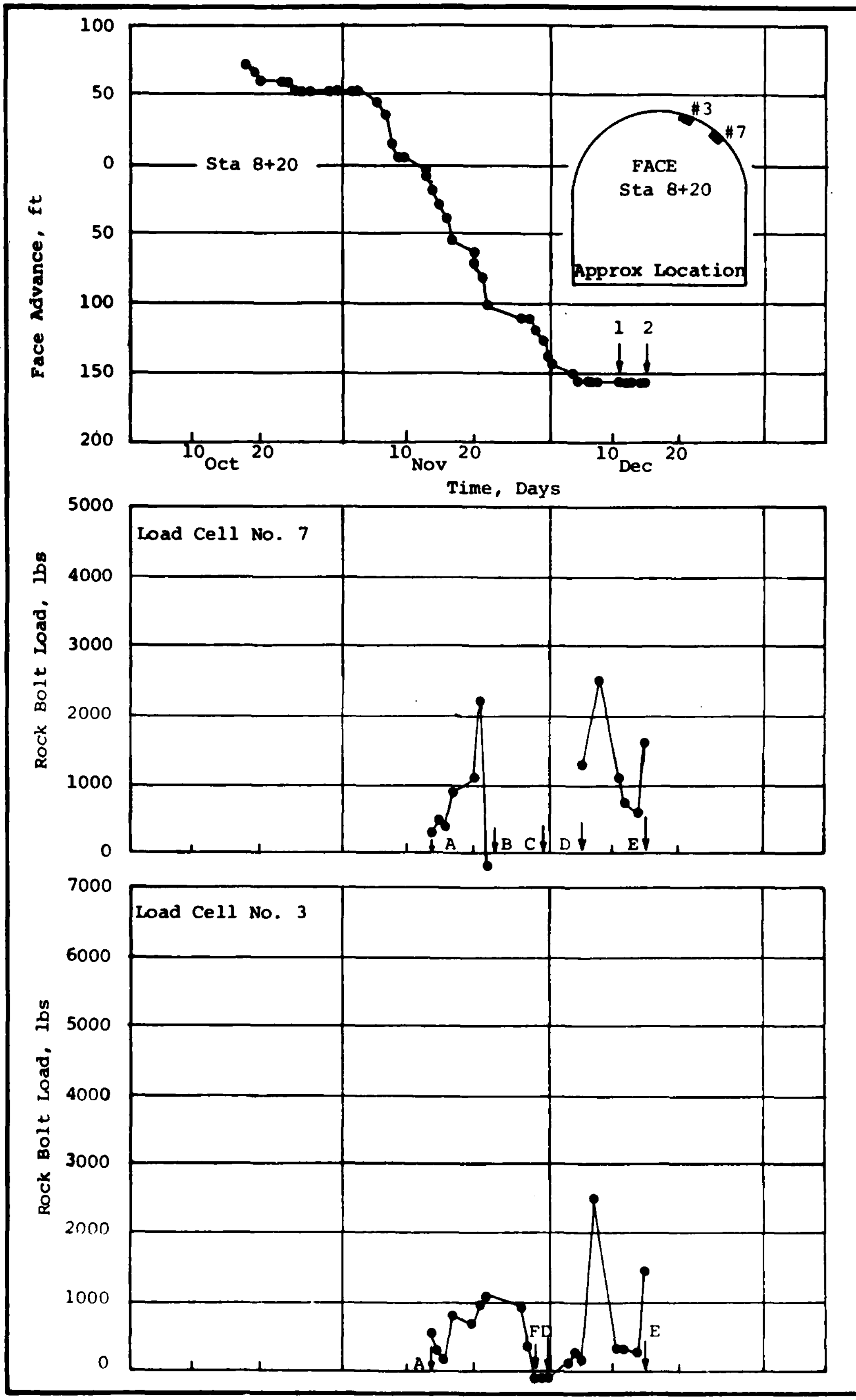
Park River Auxiliary
Tunnel
Hartford, CT

August 1980

Project 77382

Roger J. Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS



ROCK BOLT LOAD DATA
CELL NOS. 7 AND 3

Fig.B-3

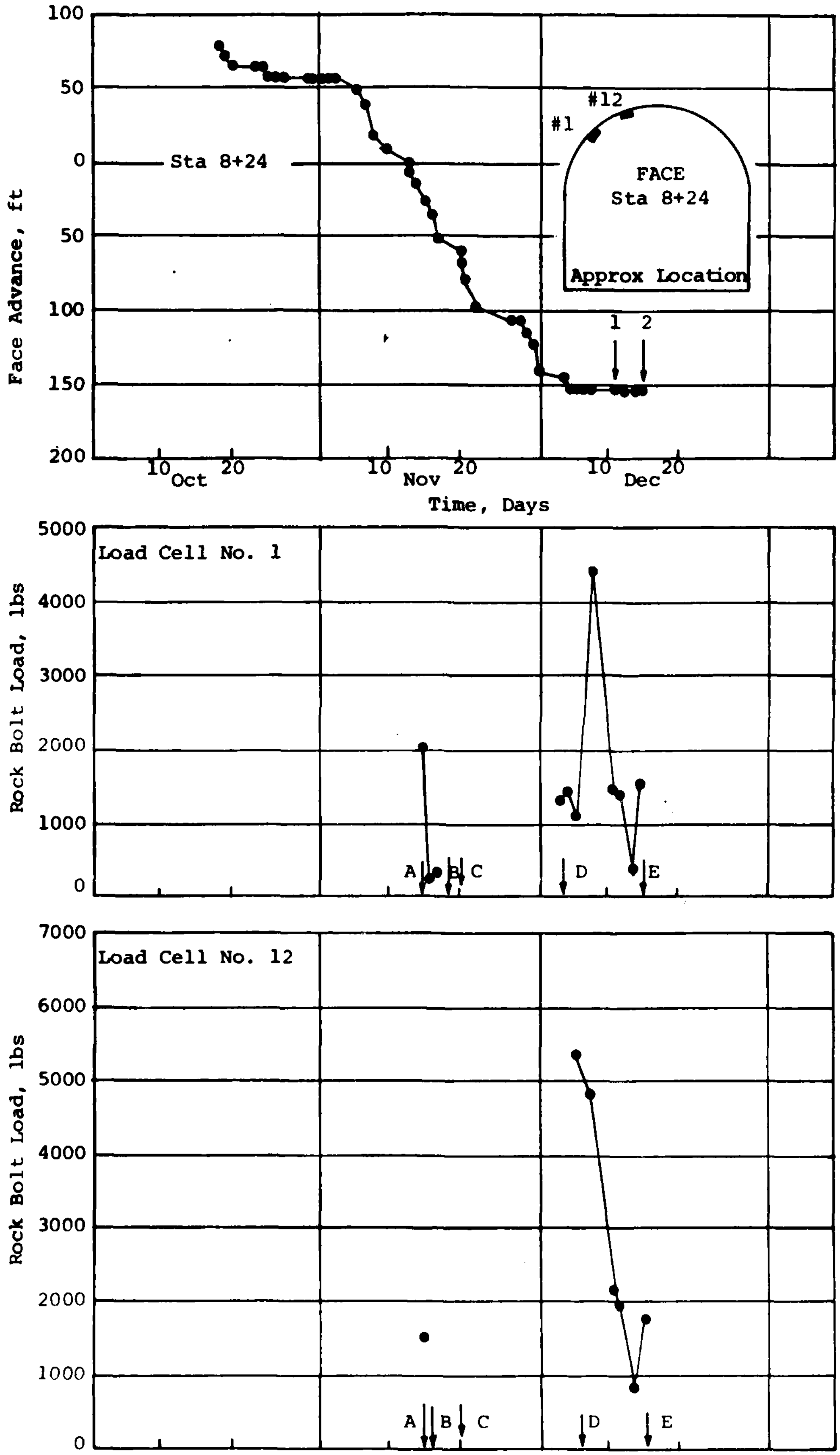
August 1980

Park River Auxiliary
Tunnel
Hartford, CT

Project 77382

Roger J. Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS



ROCK BOLT LOAD DATA
CELL NOS. 1 AND 12

Fig. B-4

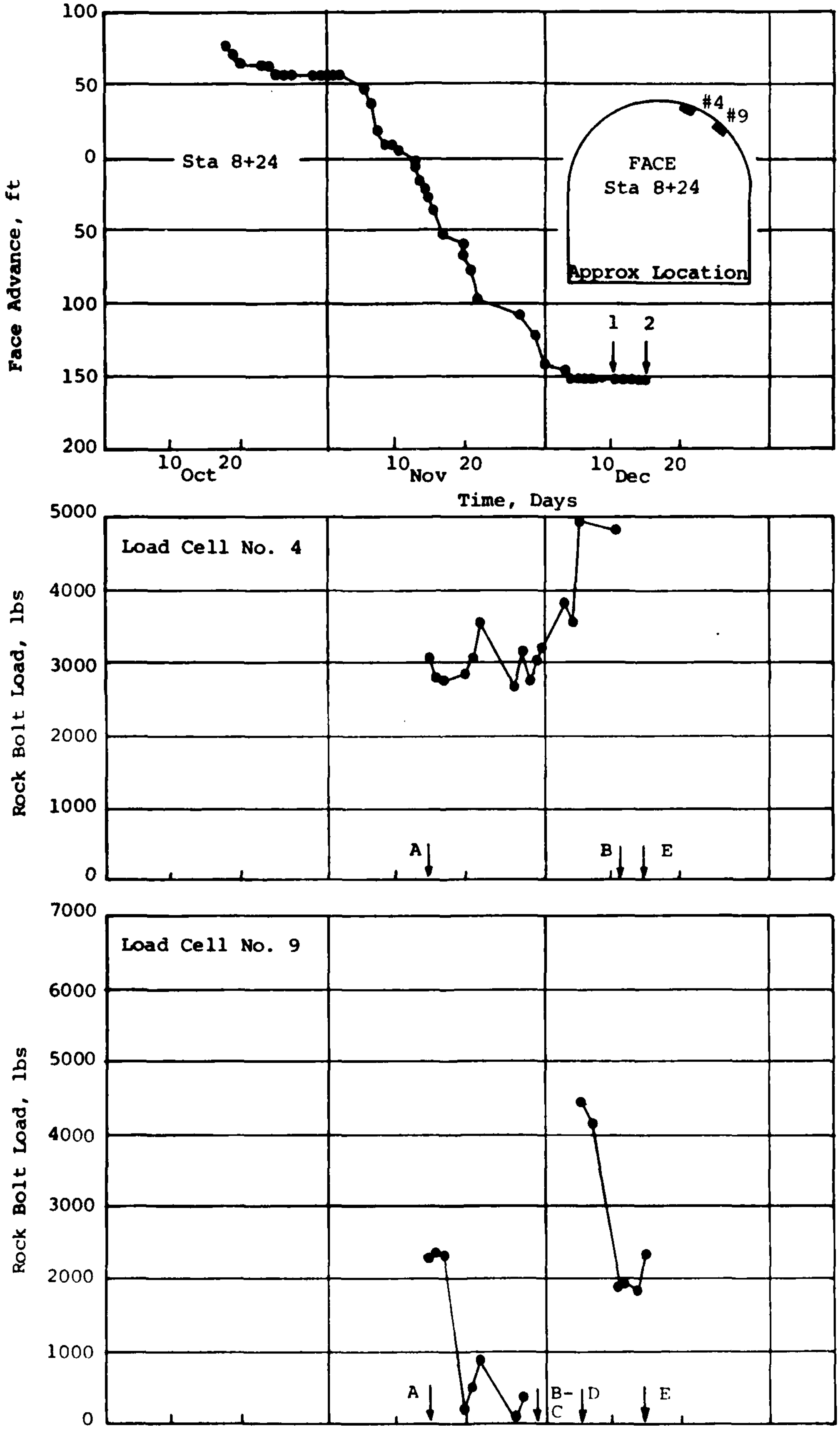
August 1980

Park River Auxiliary
Tunnel
Hartford, CT

Project 77382

Roger J. Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS



ROCK BOLT LOAD DATA
CELL NOS. 4 AND 9

Fig. B-5

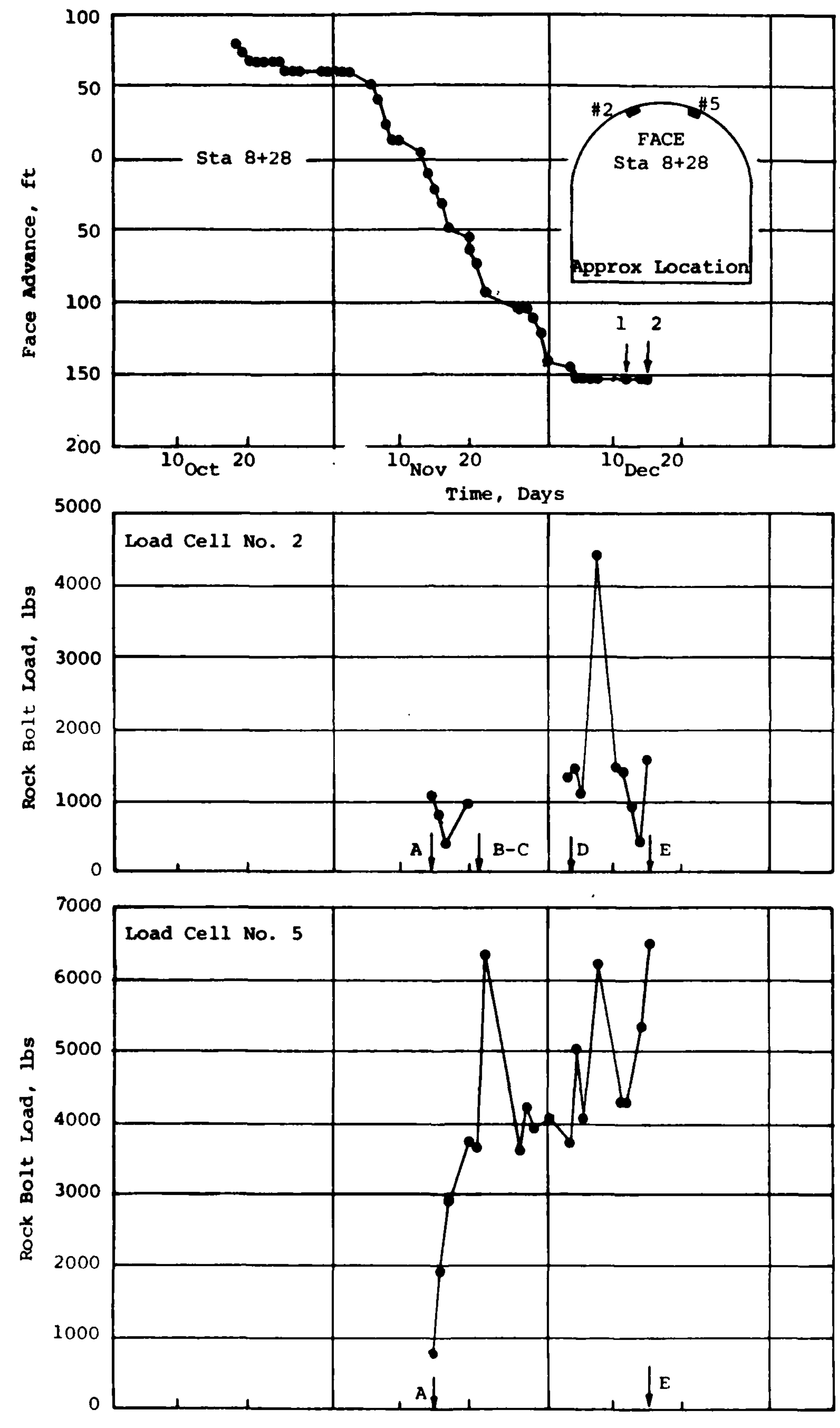
August 1980

Park River Auxiliary
Tunnel
Hartford, CT

Project 77382

Roger J. Au & Sons, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS



ROCK BOLT LOAD DATA
CELL NOS. 2 AND 5

Fig. B-6

August 1980

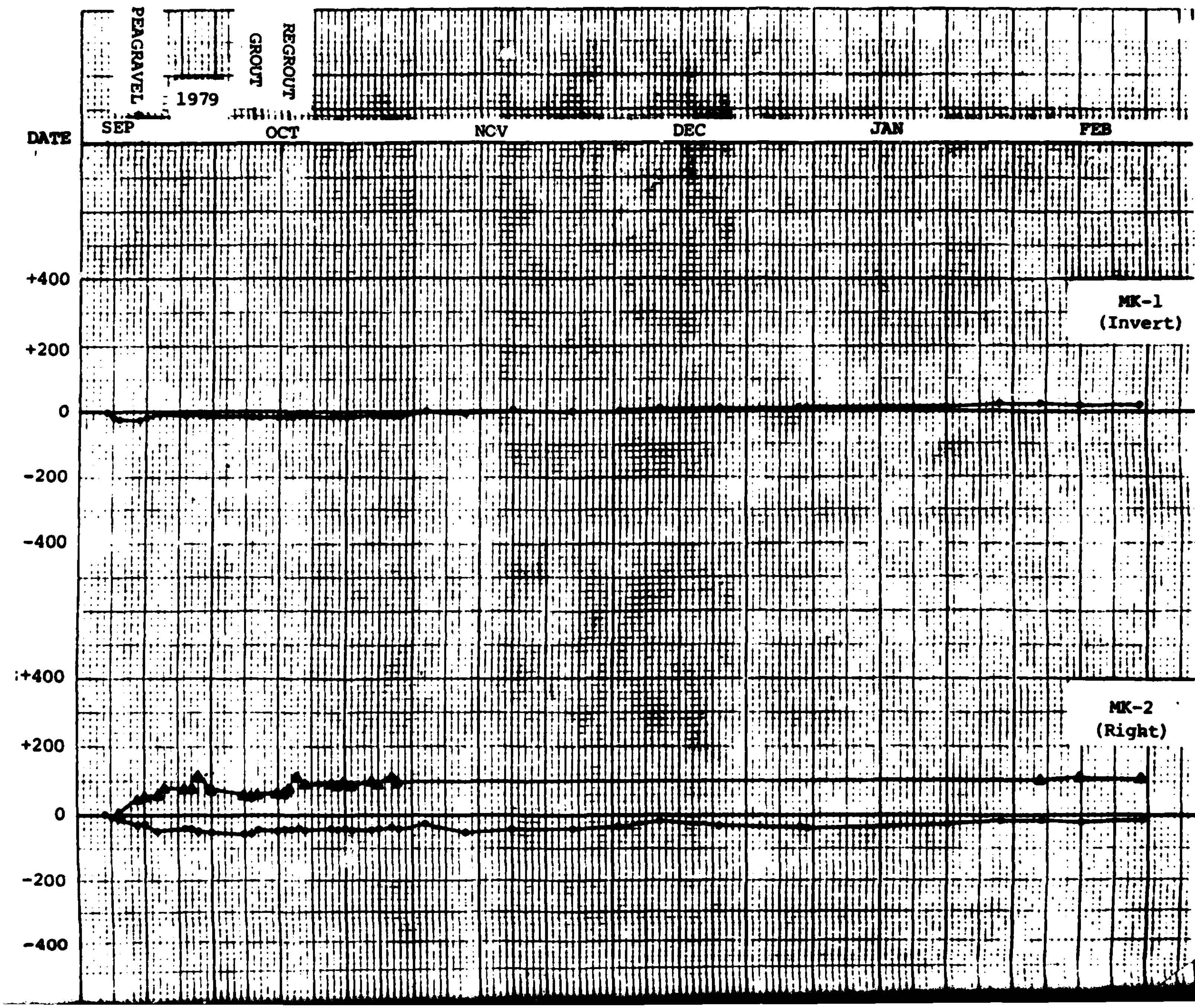
Park River Auxiliary
Tunnel
Hartford, CT

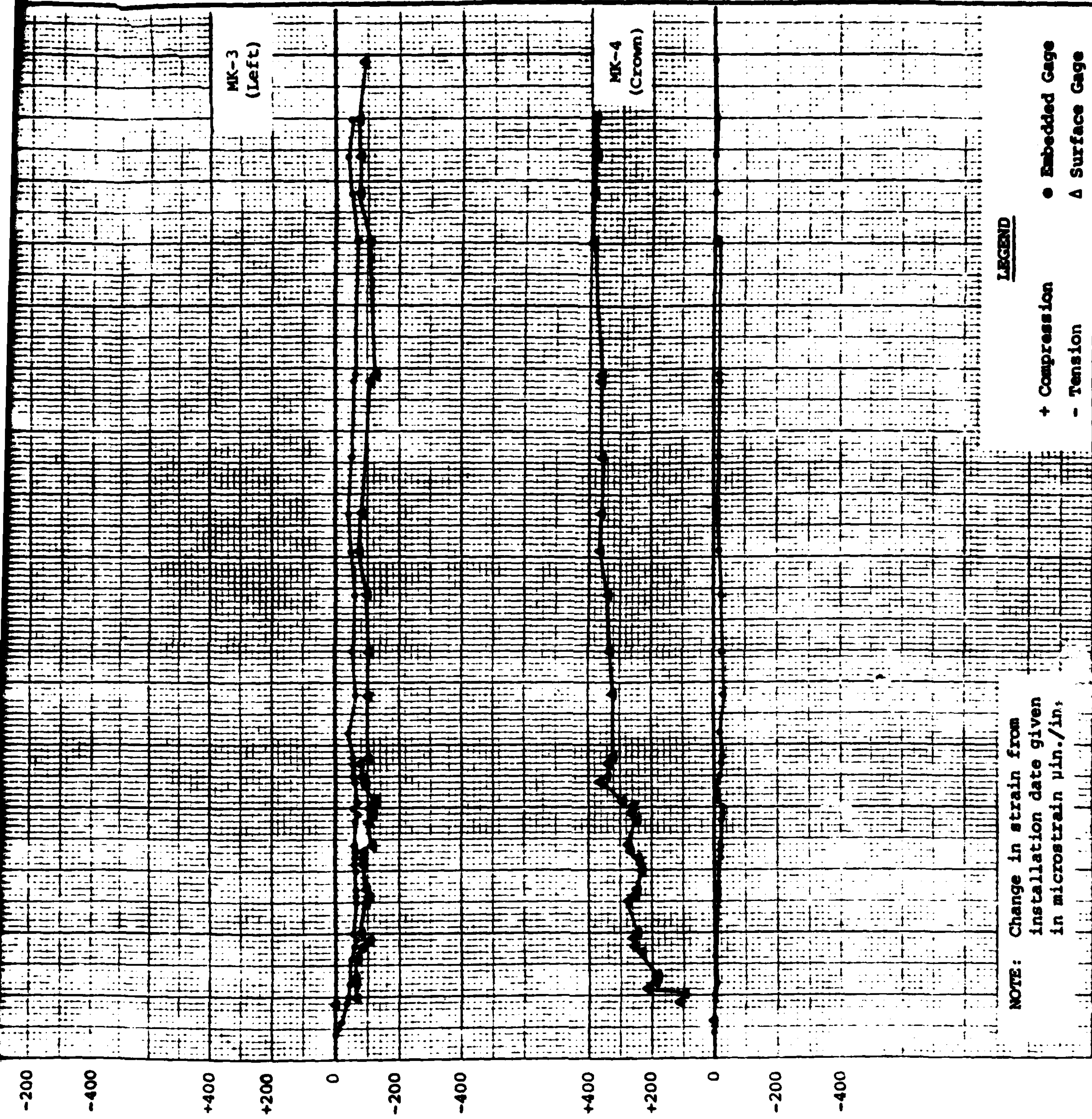
Project 77382

Roger J. Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS

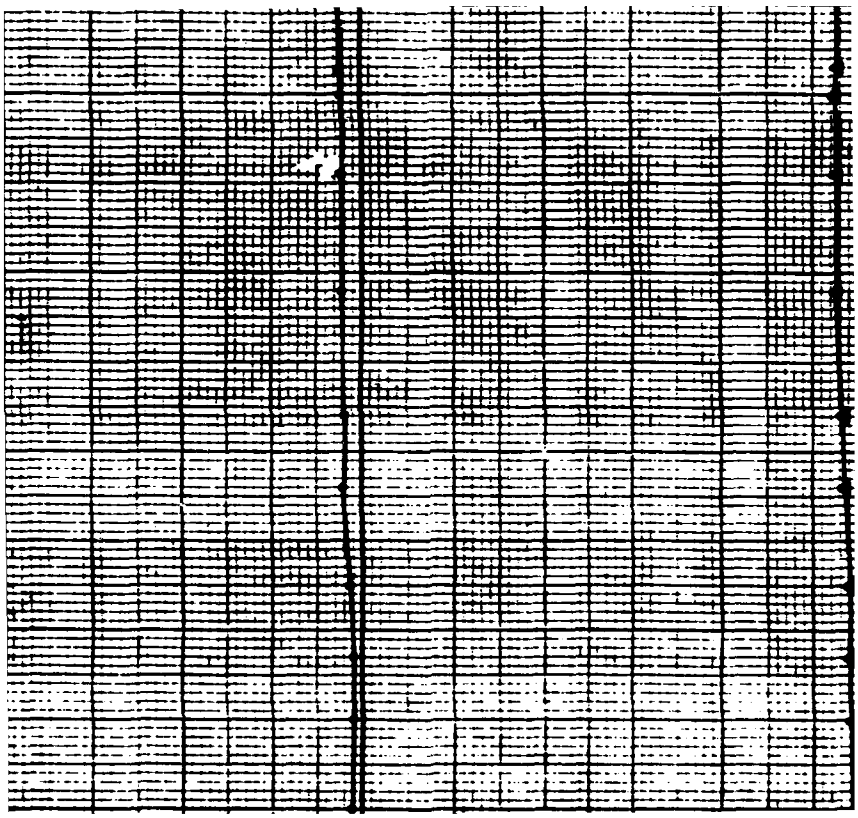
APPENDIX C

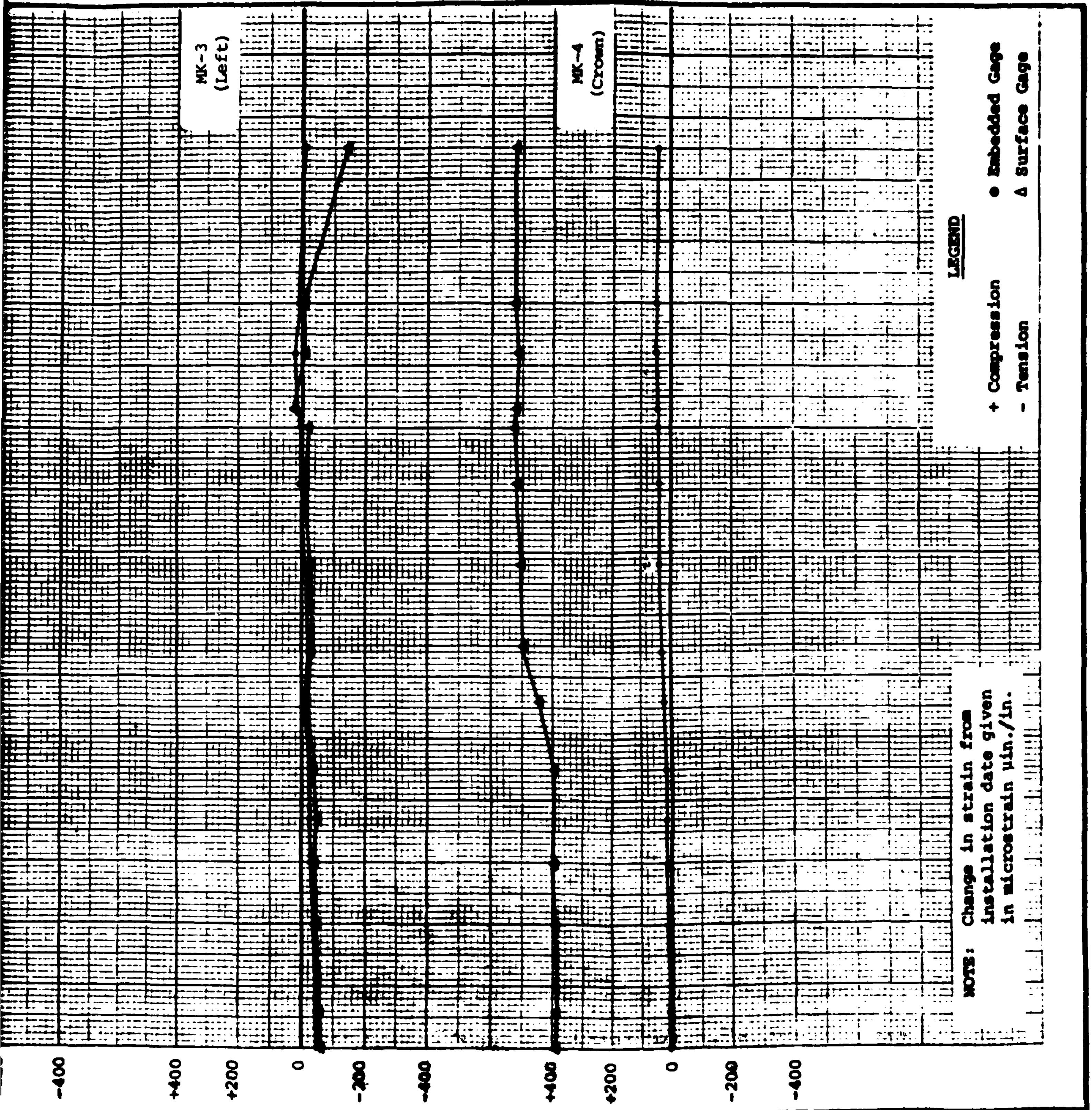

PEAGRAVEL
1979
GROUT
REGROUT
DATE
SEP
OCT
NCV
DEC
JAN
FEB
+400
+200
0
-200
-400
MK-1
(Invert)
+400
+200
0
-200
-400
MK-2
(Right)



ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 18 TS 2 STA. 11+32
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-

2

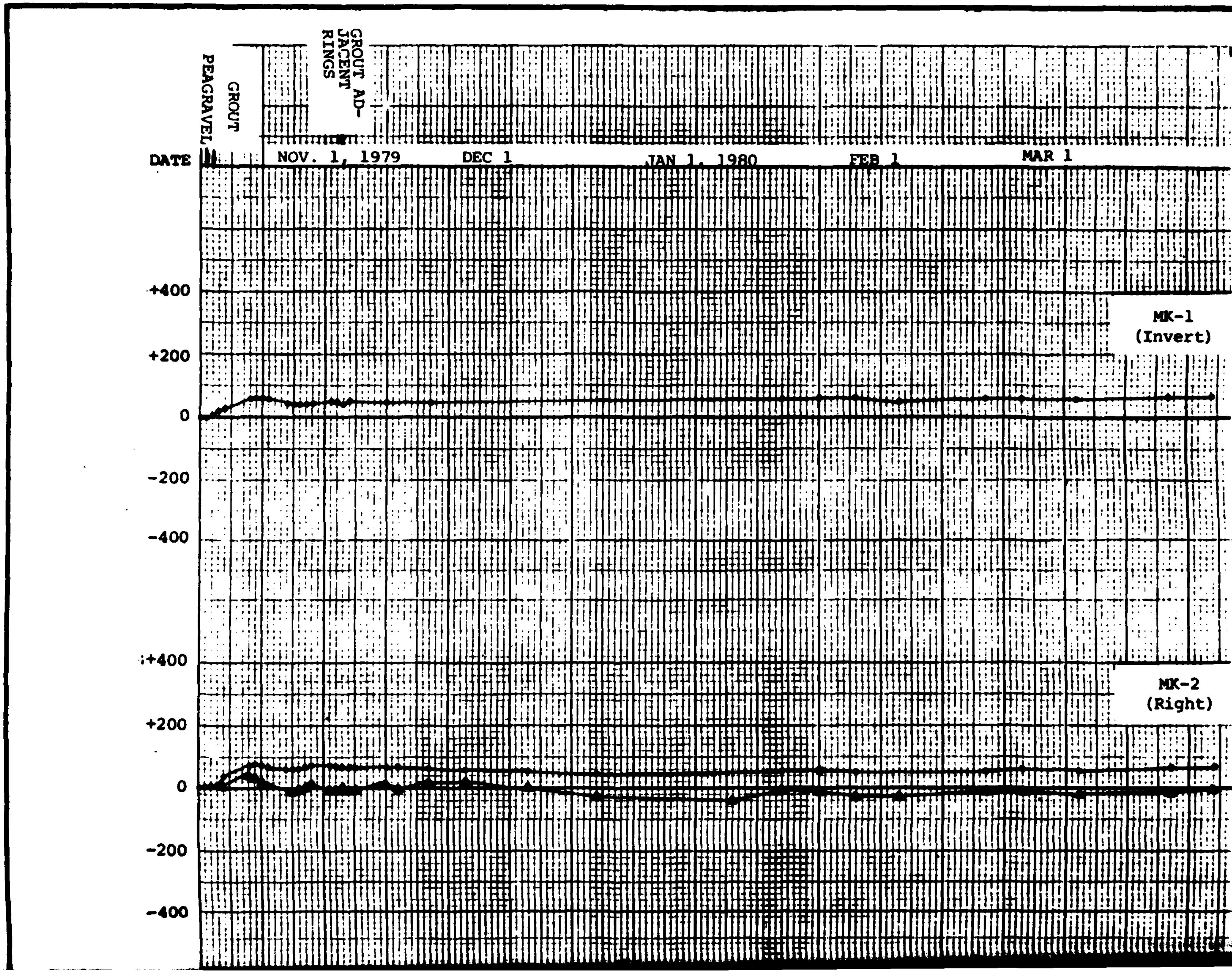




ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING #18 TS 2 STA. 11+32
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	PROJECT 77302	JULY 1980 FIG. C-1

Cont.

2



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JAN 1, 1980
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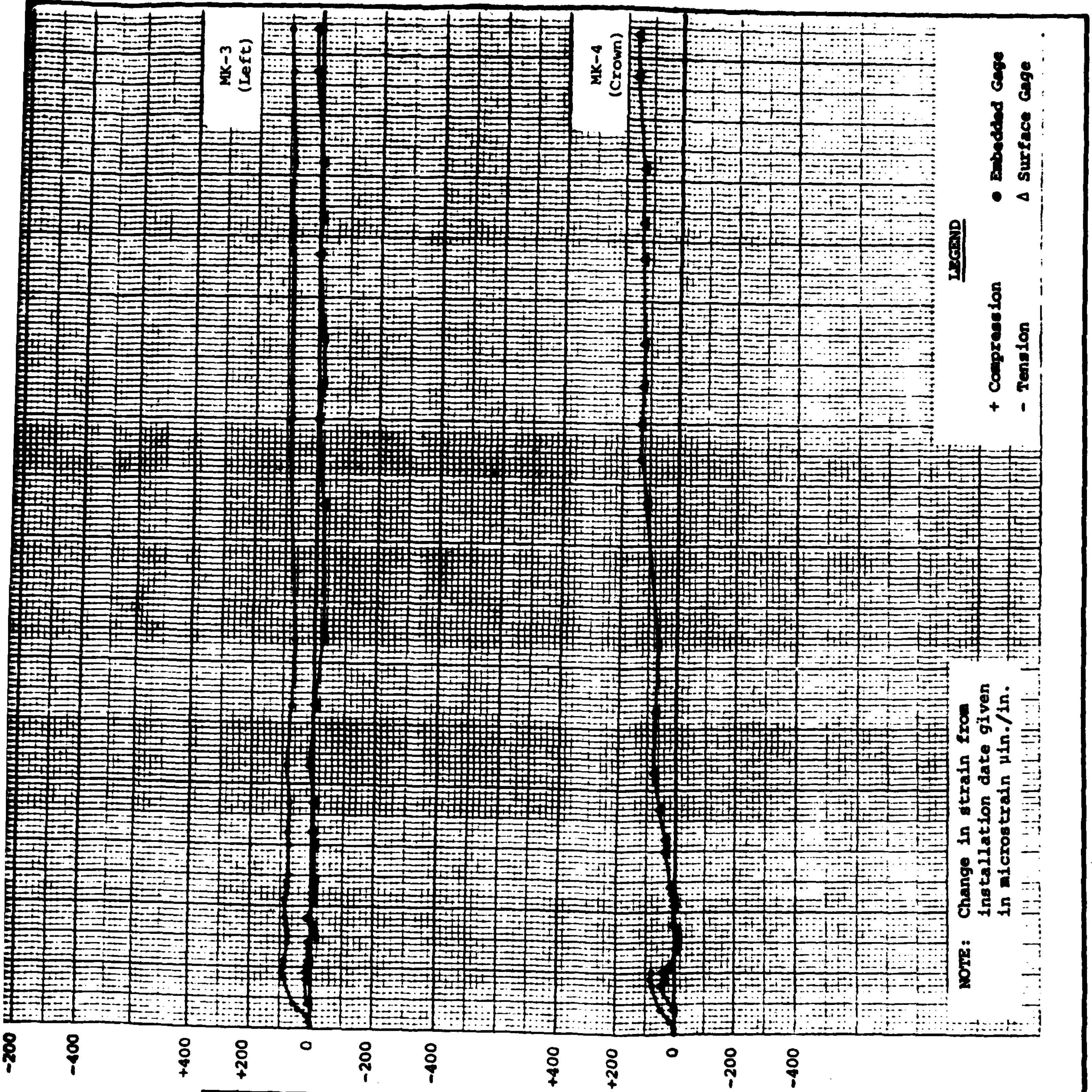
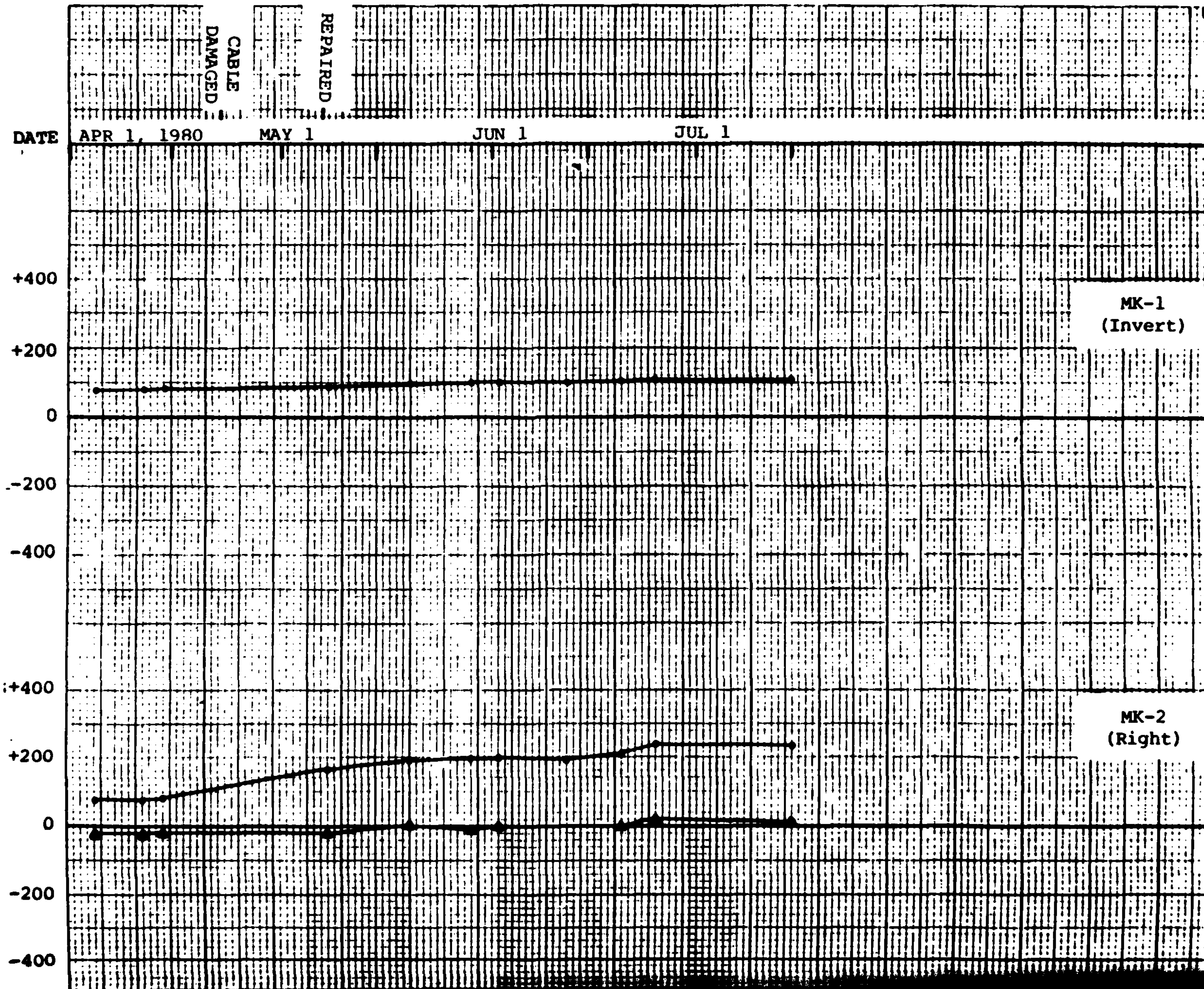
+200
0
-200
-400
MK-2
(Right)



ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 81 TS 3 STA 15+25
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-2

2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MAY 1
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JUL 1
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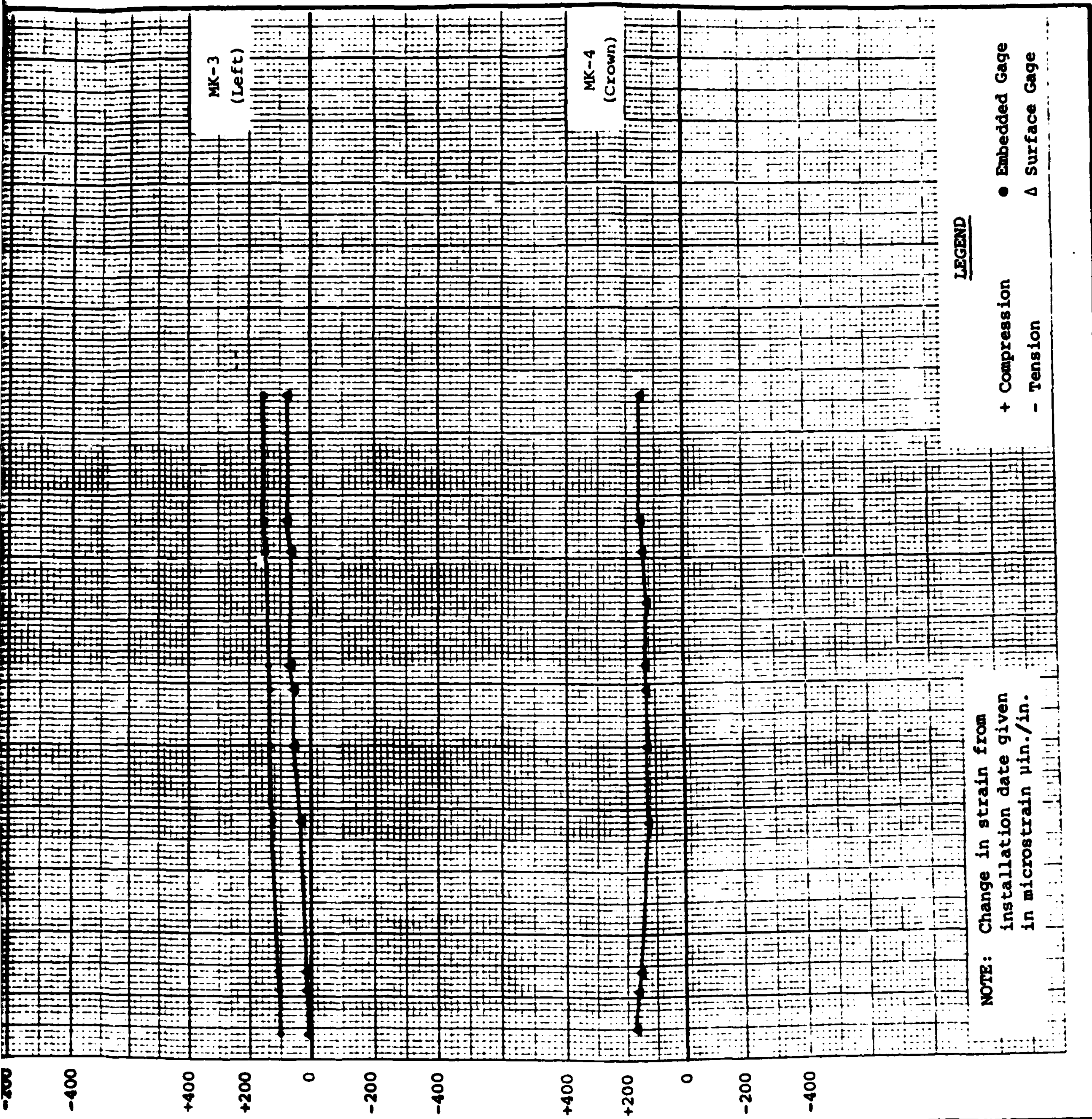
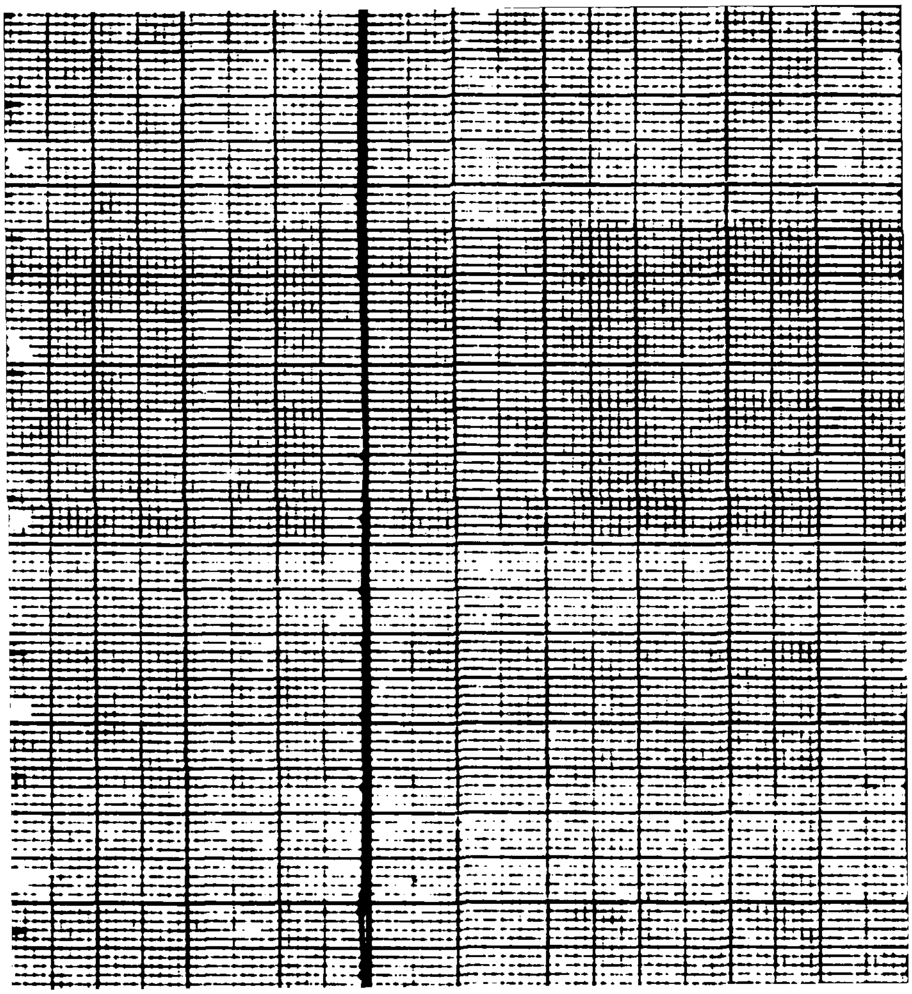
-400
MK-2 (Right)

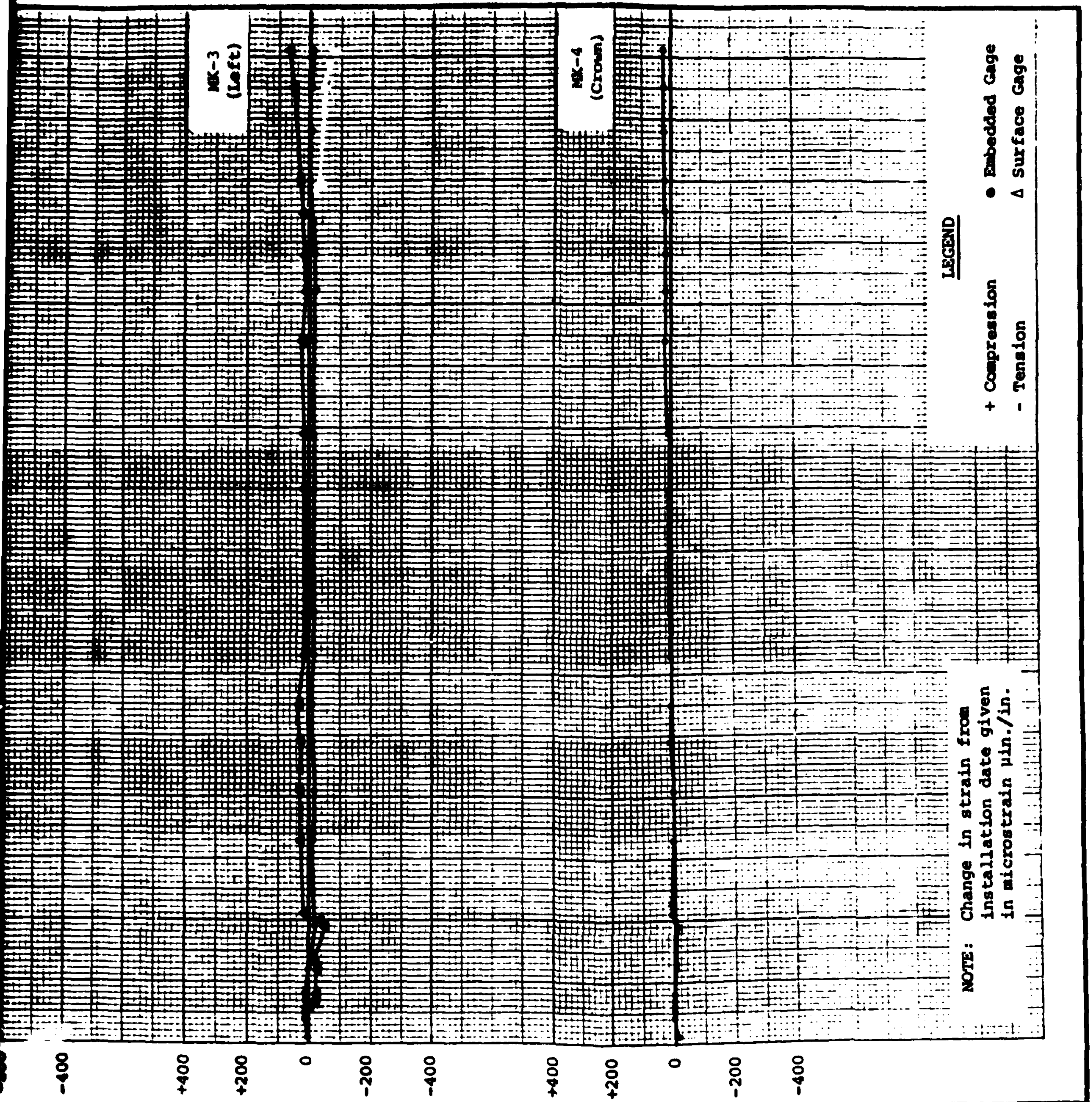


ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 81 TS 3 STA 15+25
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-2

Cont.

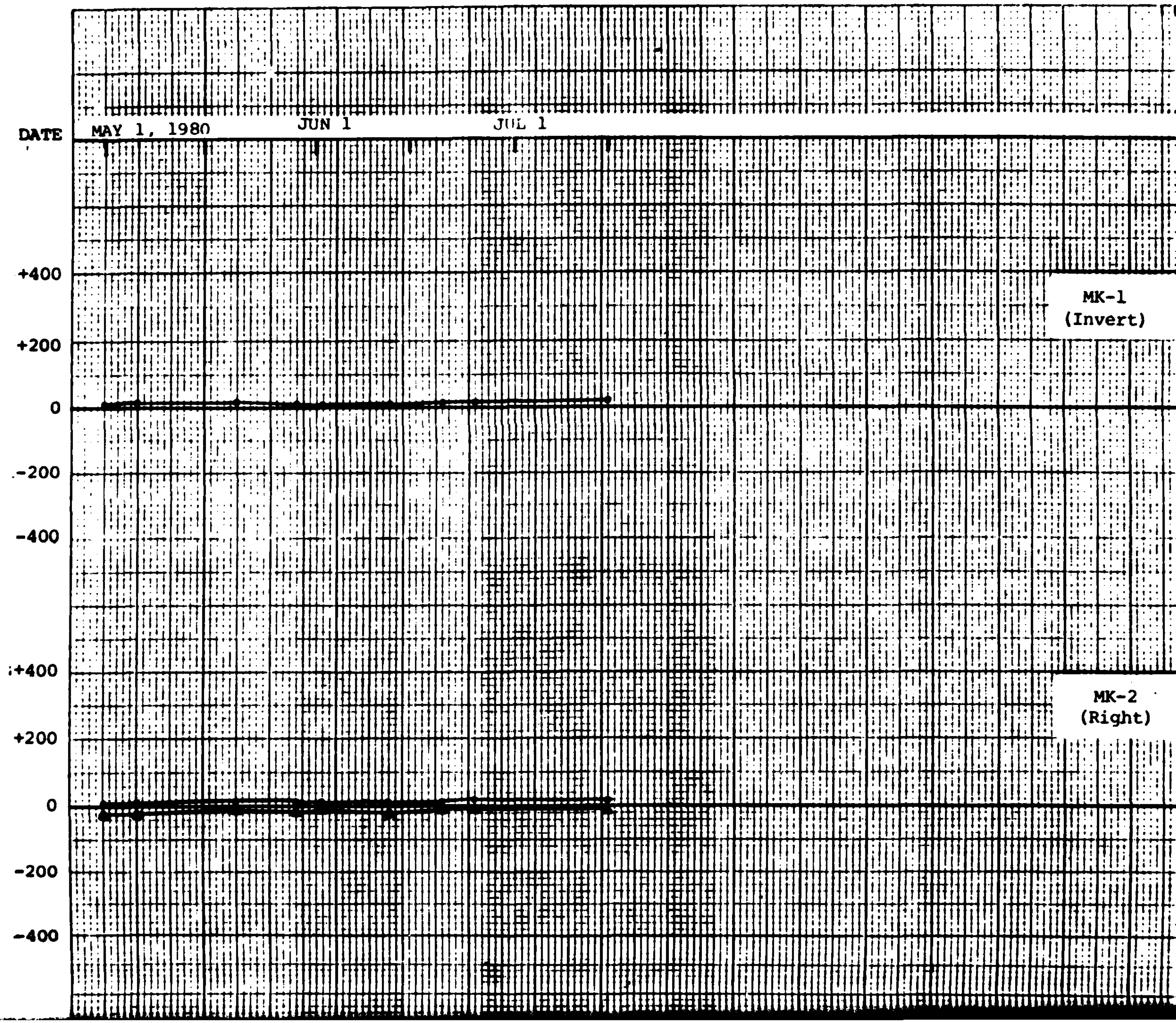
2





ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 225 TS 4 STA 24+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-3

2


DATE
MAY 1, 1980
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JUL 1
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+200
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MK-1
(Invert)
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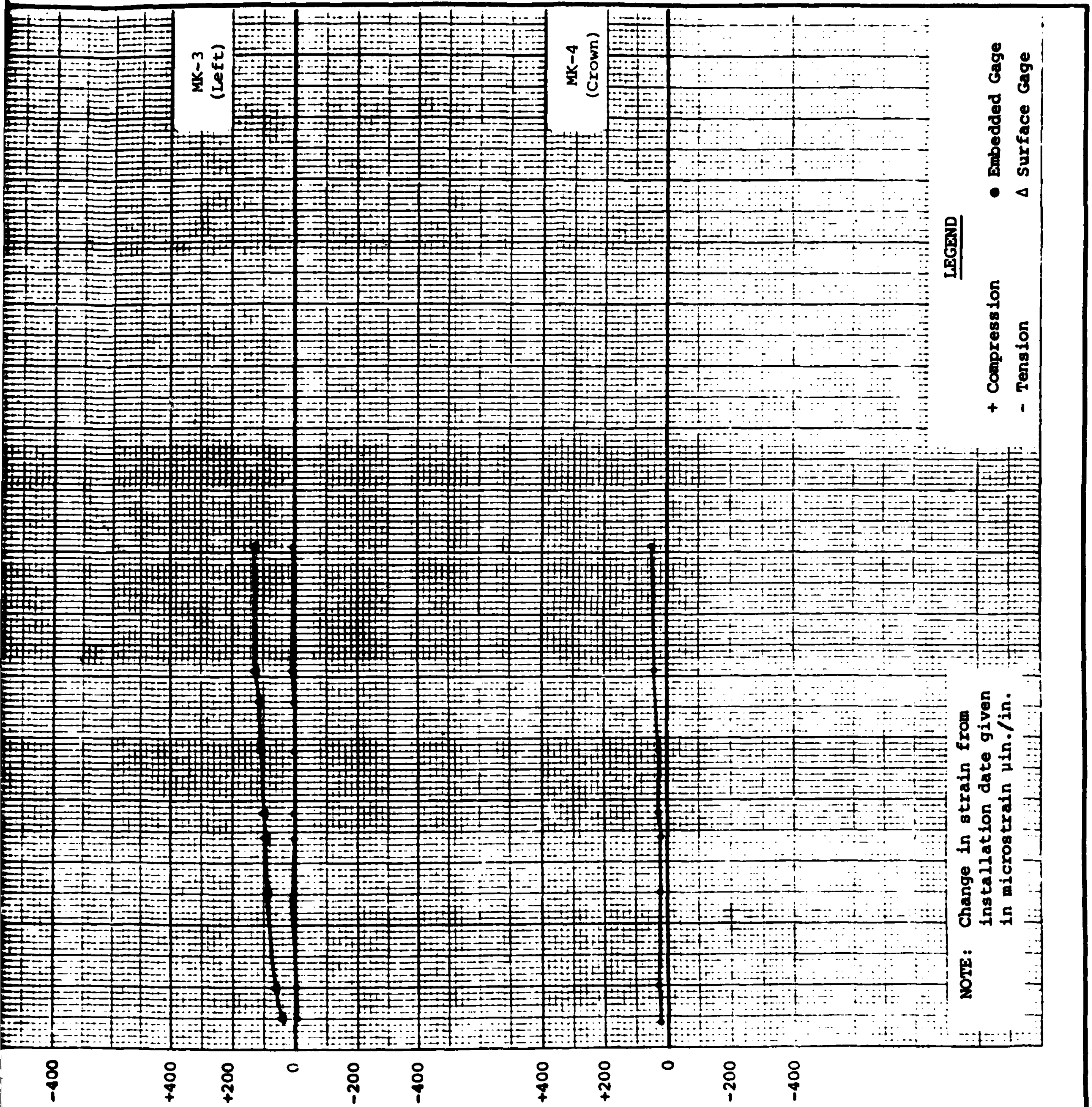
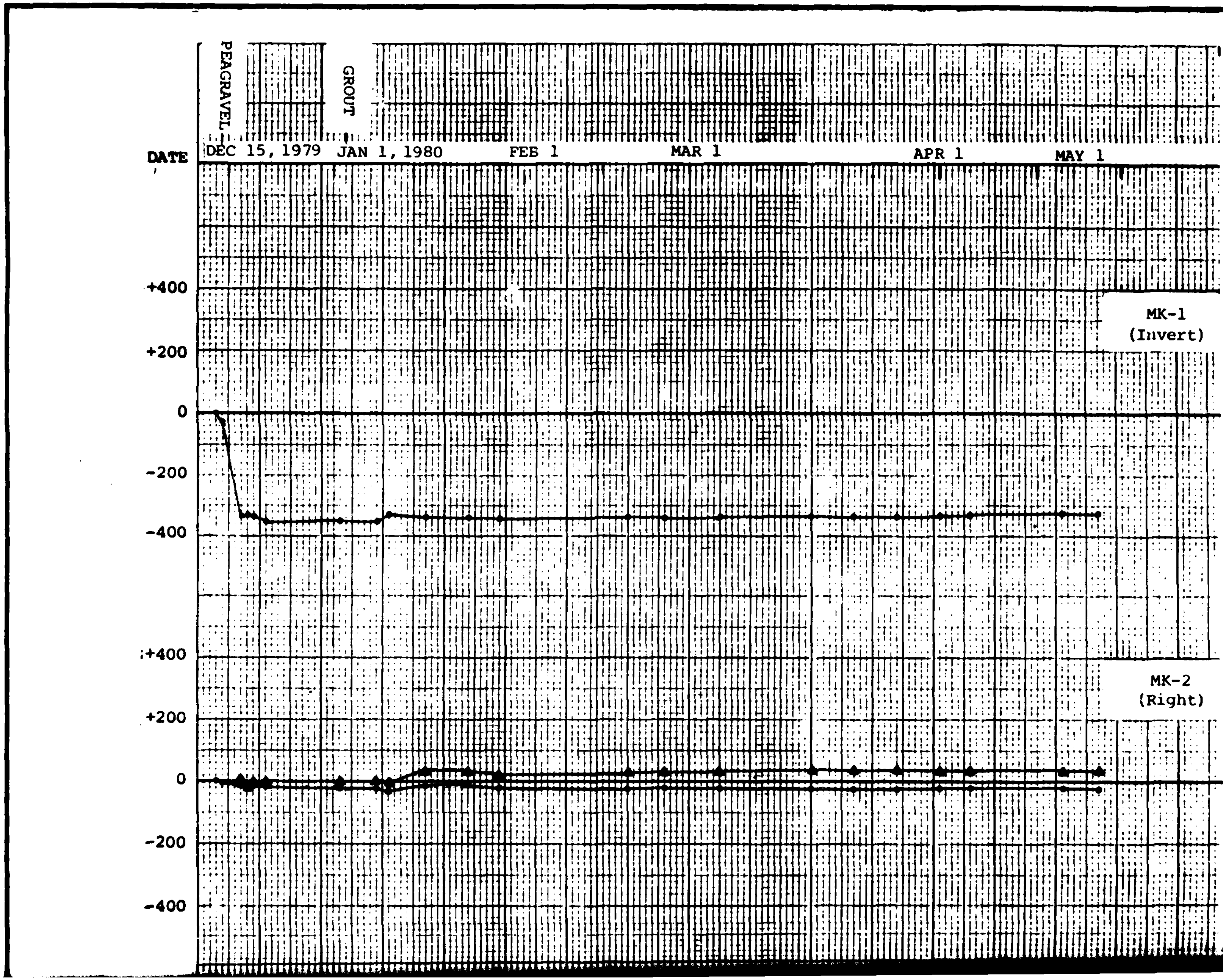
-200
-400
MK-2
(Right)



ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 225 TS 4 STA 24+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-3

Cont.

2



PEAGRAVEL
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JAN 1, 1980
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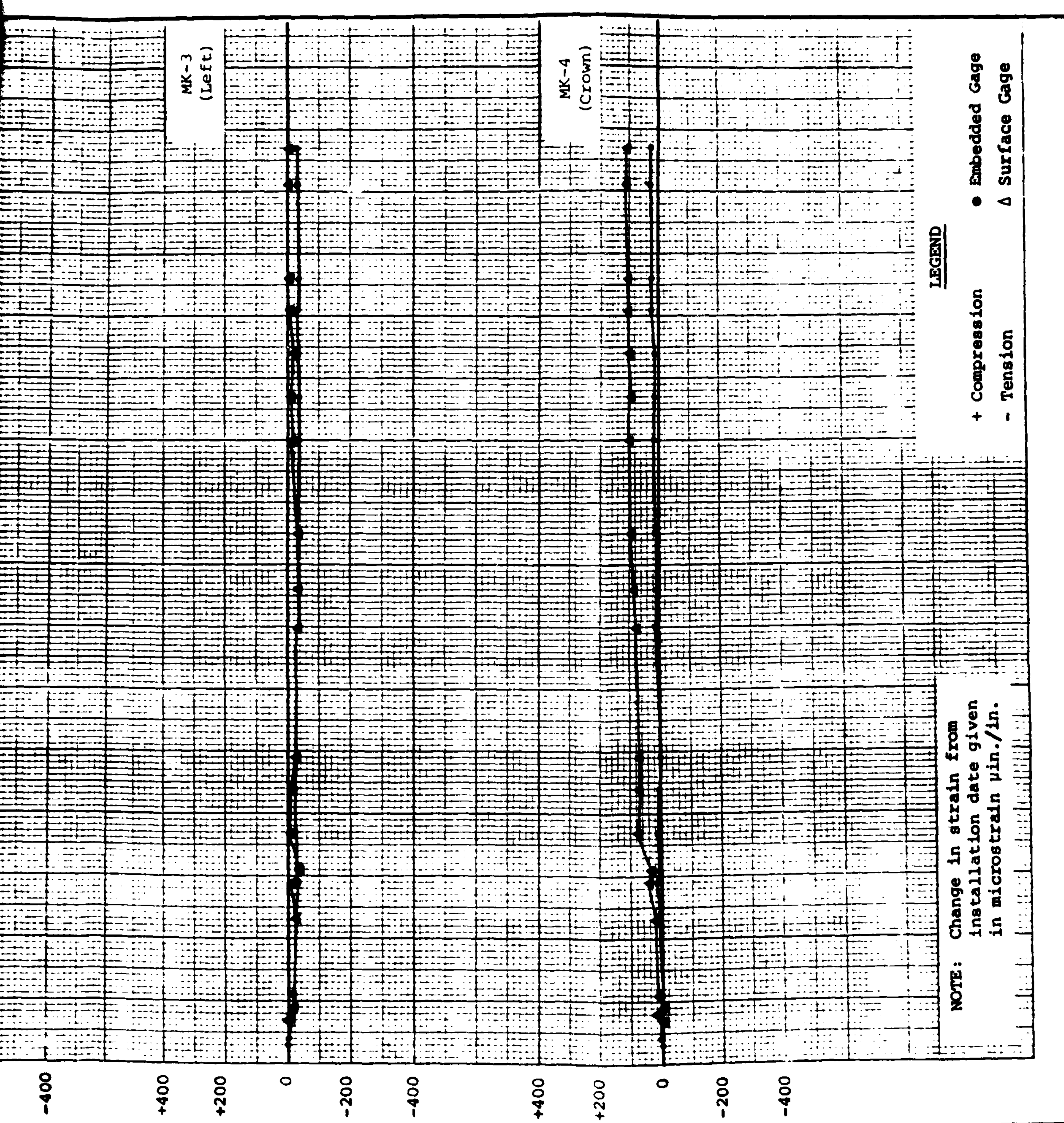
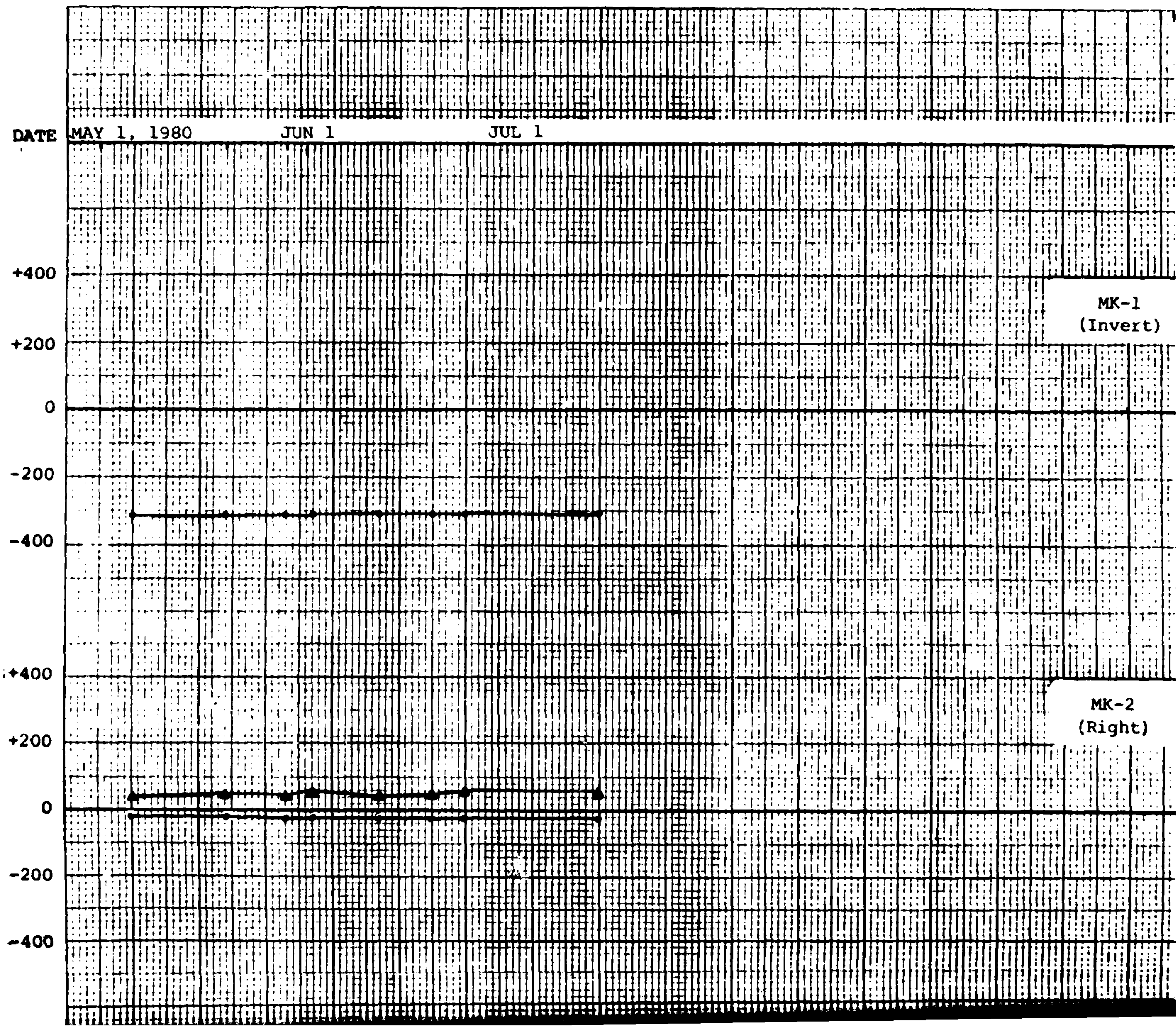
-200
-400
MK-2
(Right)



ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 275 TS 5 STA 27+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-4

2



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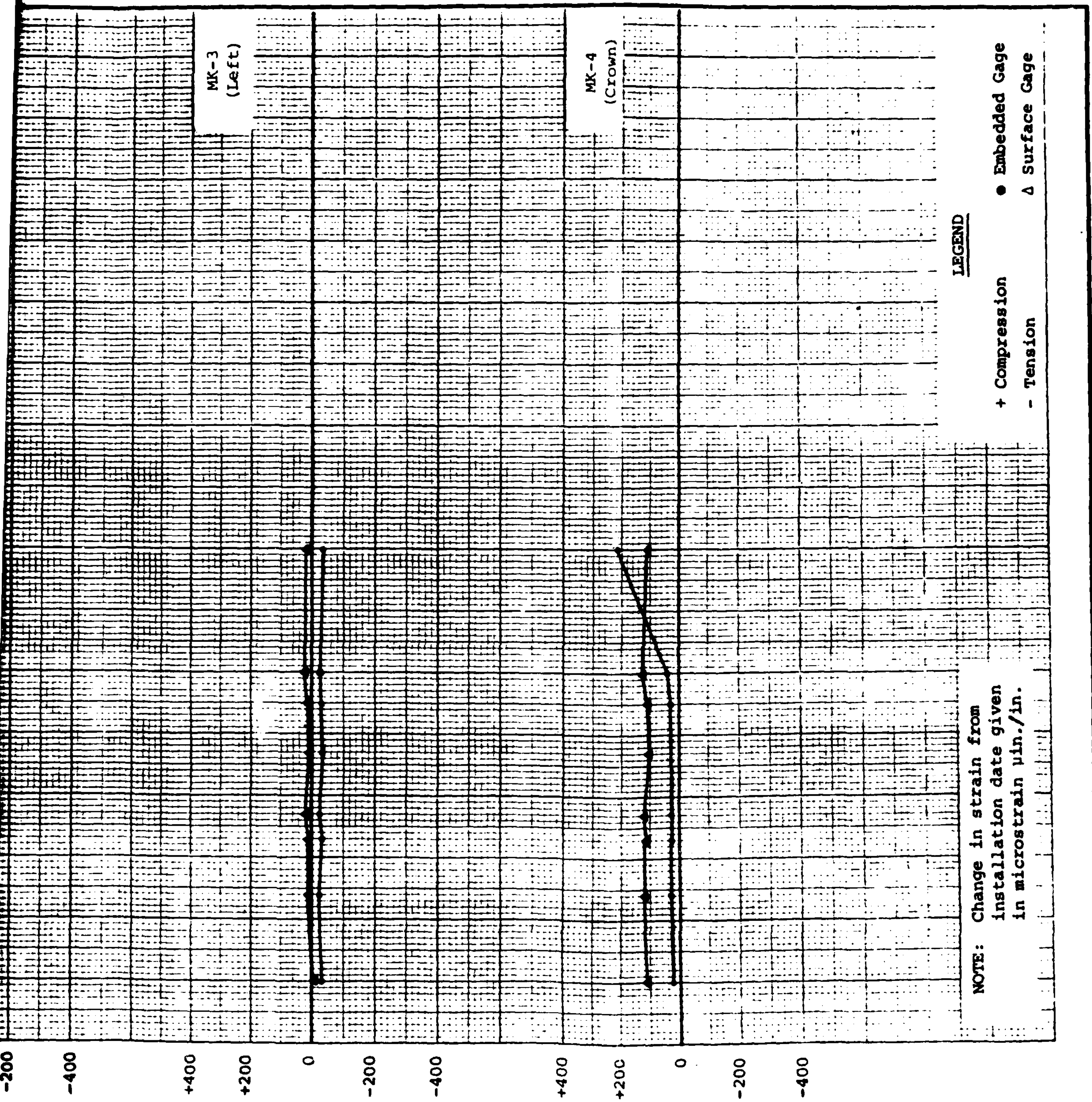
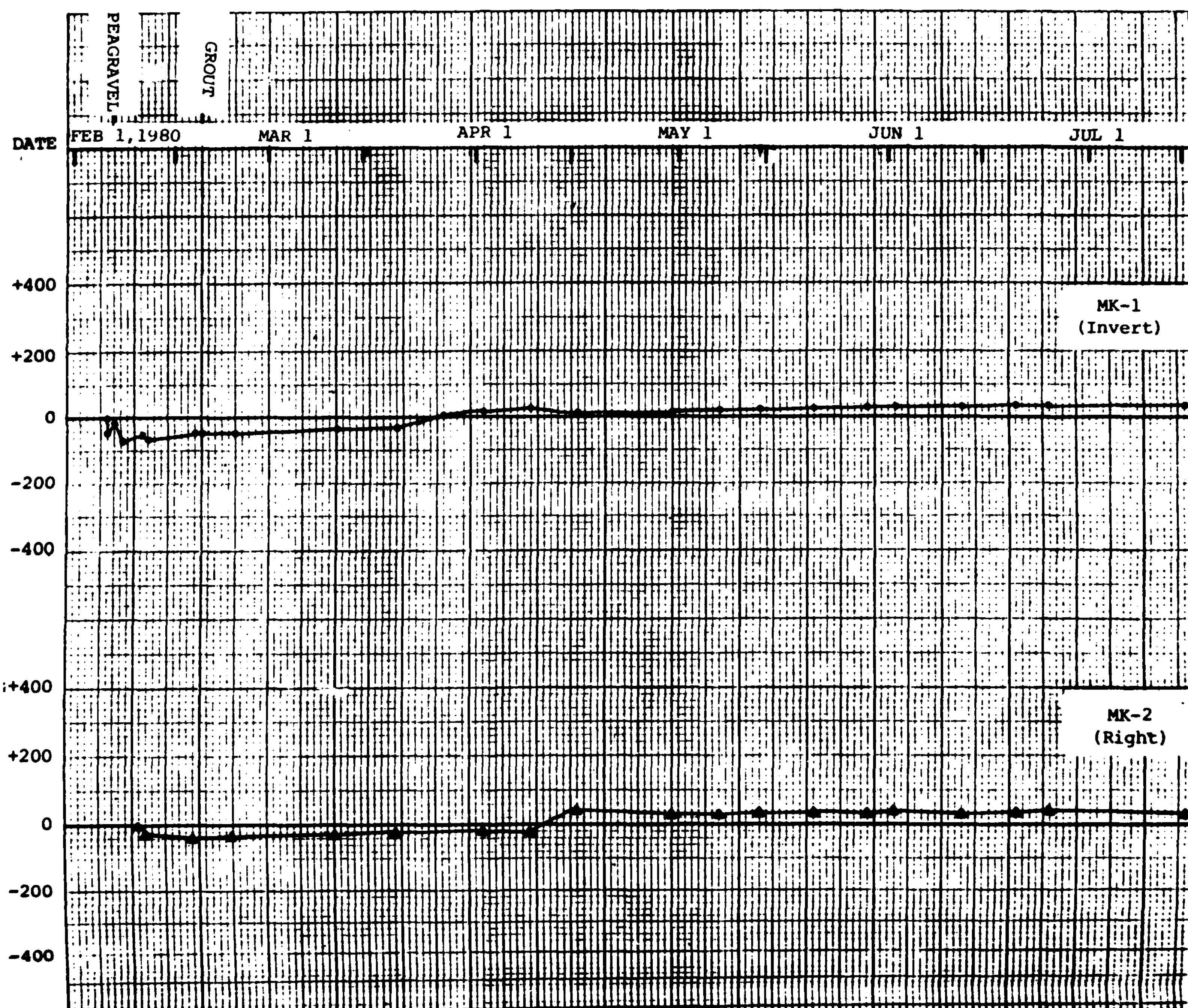
(Right)



ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 276 TS5 STA. 27+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-4

Cont.

2


PEAGRAVEL
GROUT
DATE
FEB 1,1980
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APR 1
MAY 1
JUN 1
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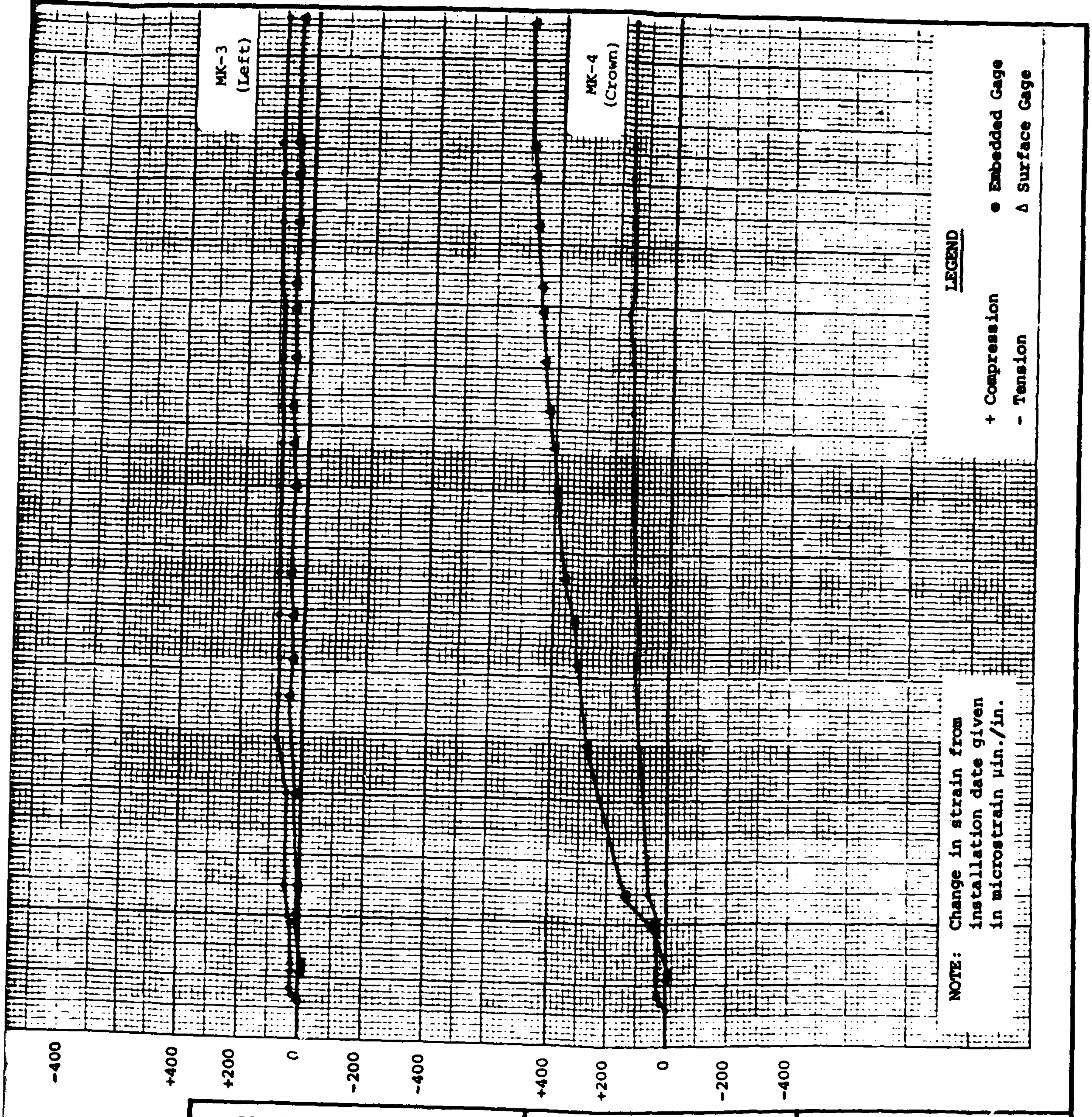
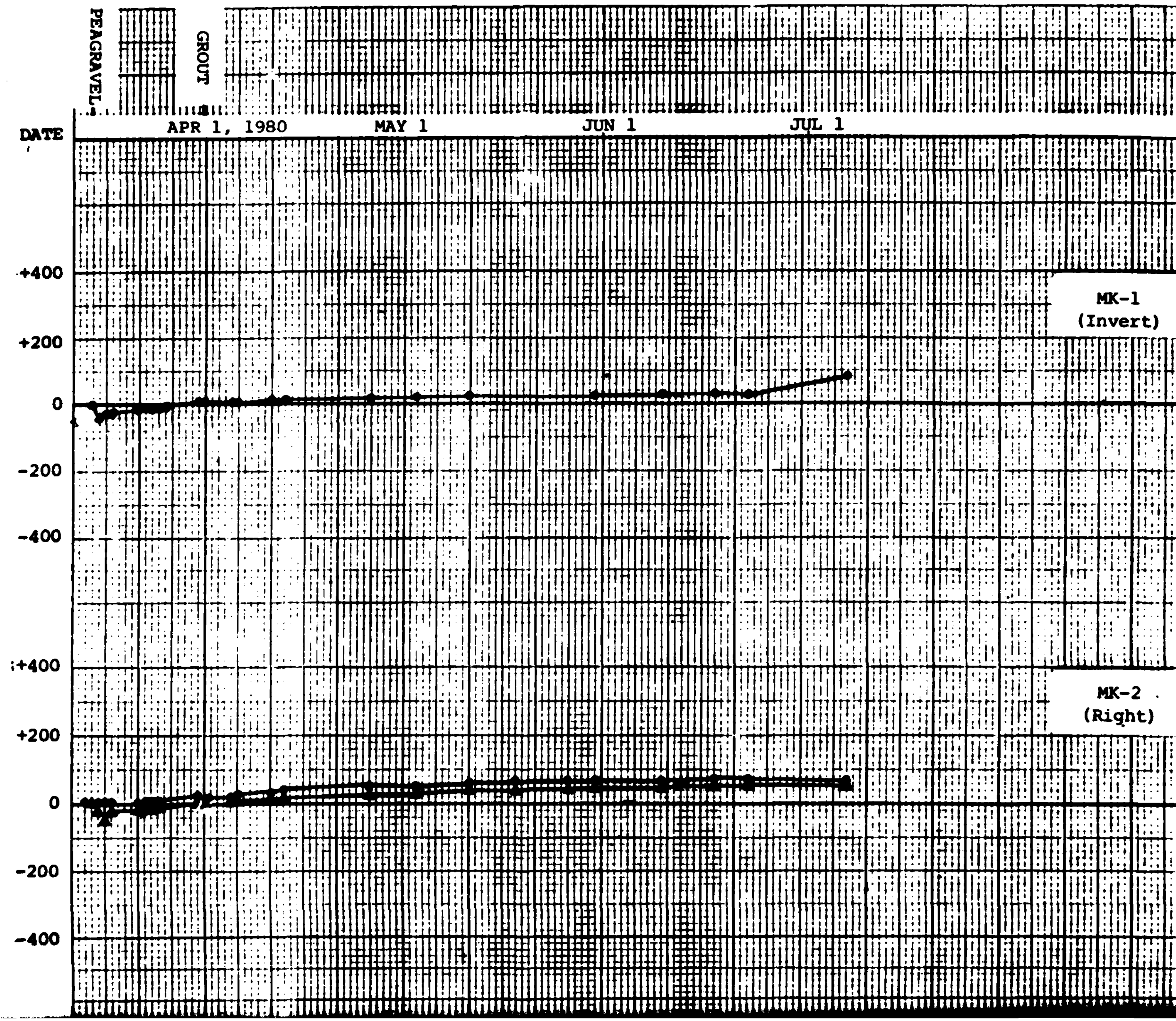
-200
-400
MK-2
(Right)



ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 548 TS 6 STA. 43+50
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-6



PEAGRAVEL
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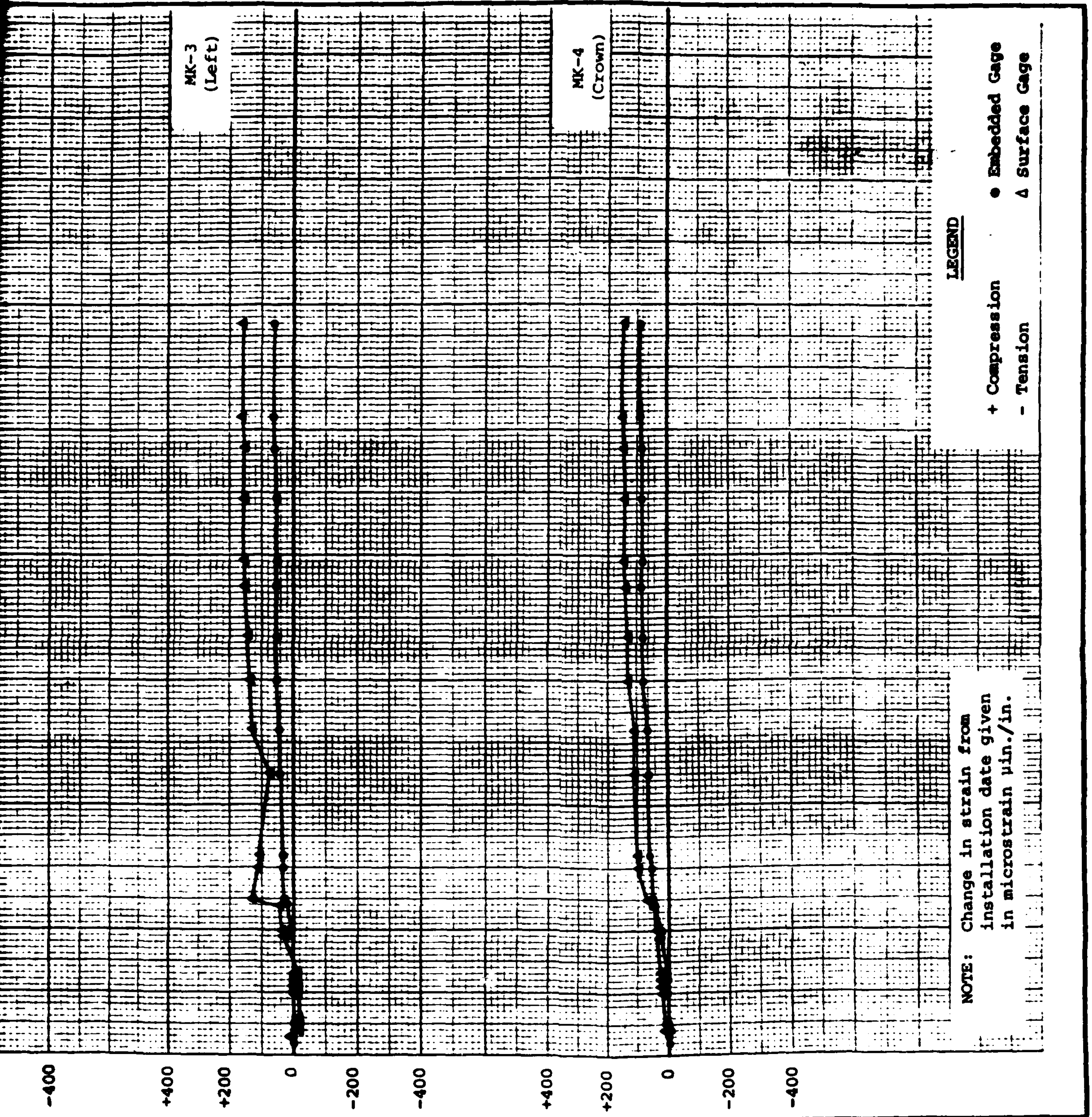


ROGER J. AU & SON INC.
MANSFIELD, OHIO

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS

PARK RIVER AUXILIARY TUNNEL

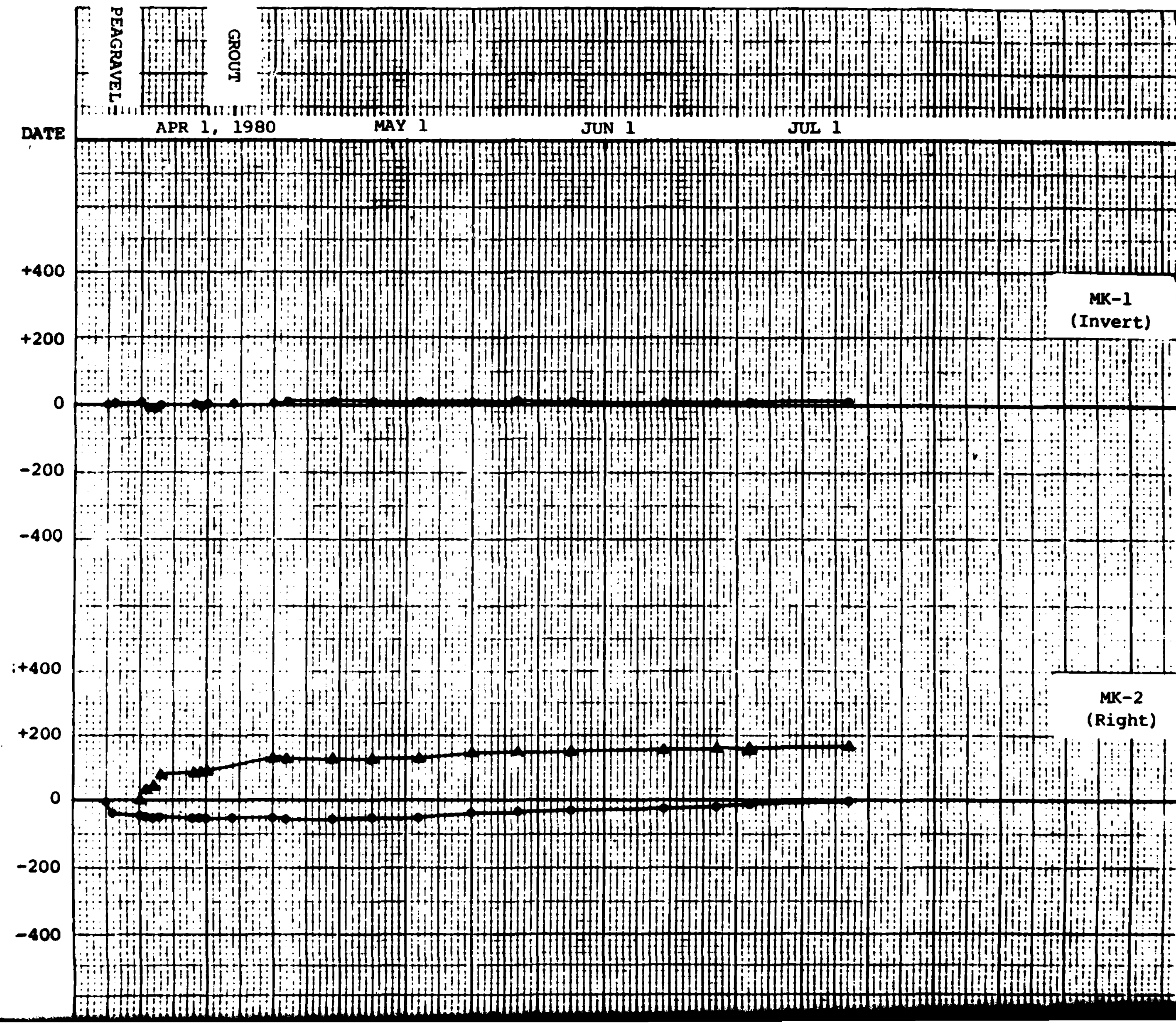
PROJECT 77382

STRAIN GAGE DATA
RING # 797
TS 7 STA. 58+55

JULY 1980

FIG. C-6

2



PEAGRAVEL
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MK-2
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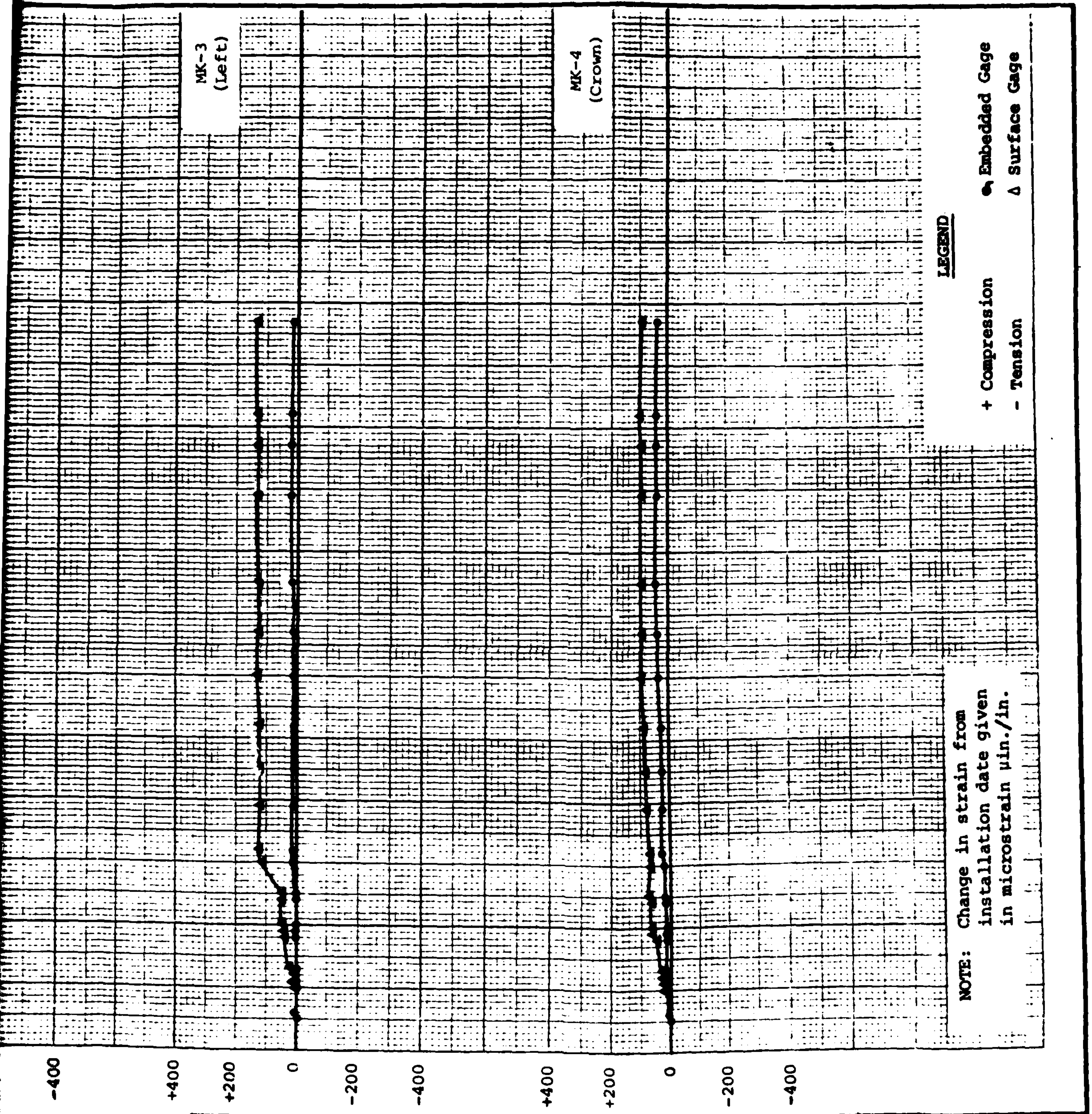
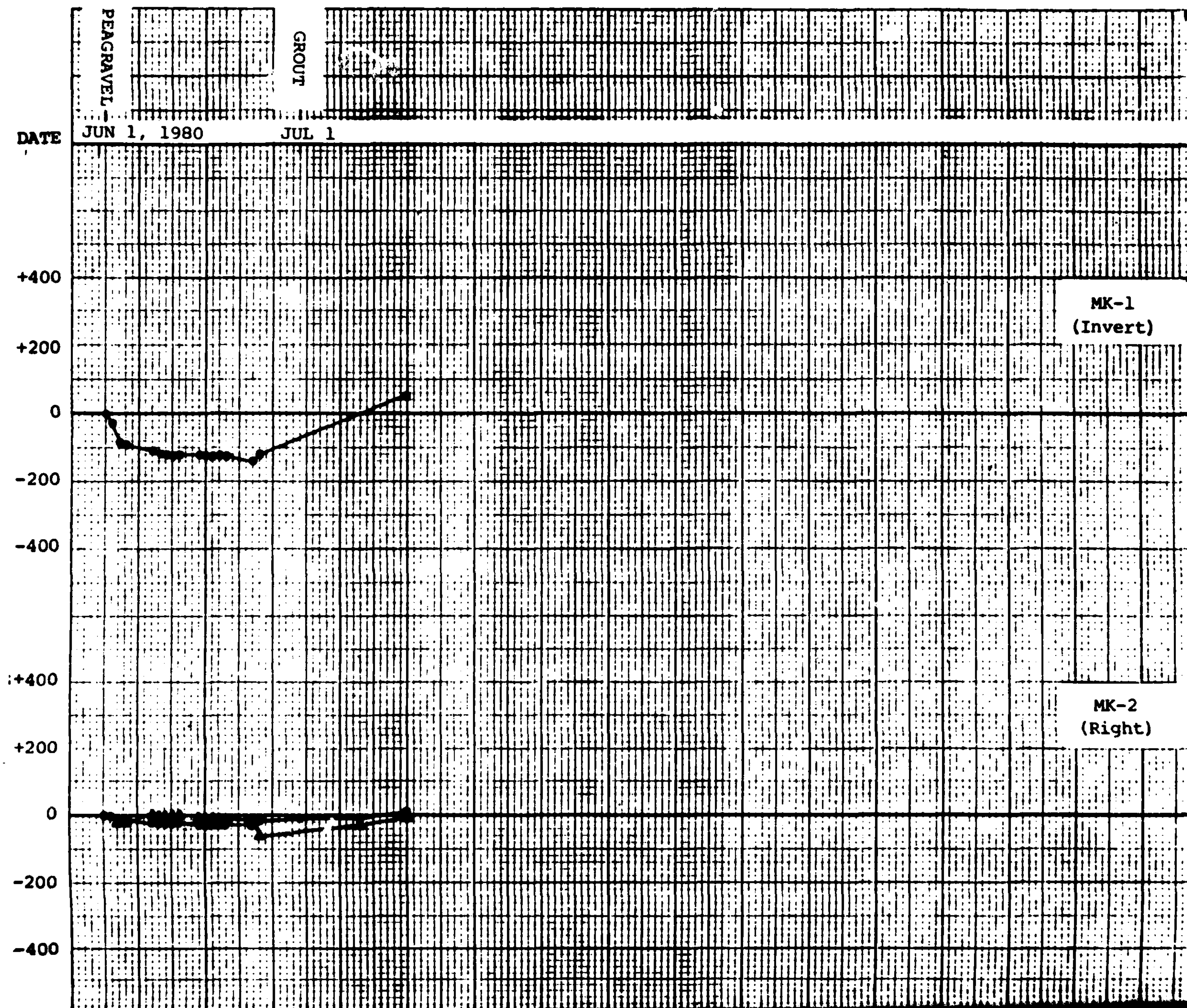


ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 840 TS 8 STA. 61+19
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-7

2



PEAGRAVEL
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-400
MK-1
(Invert)
+400
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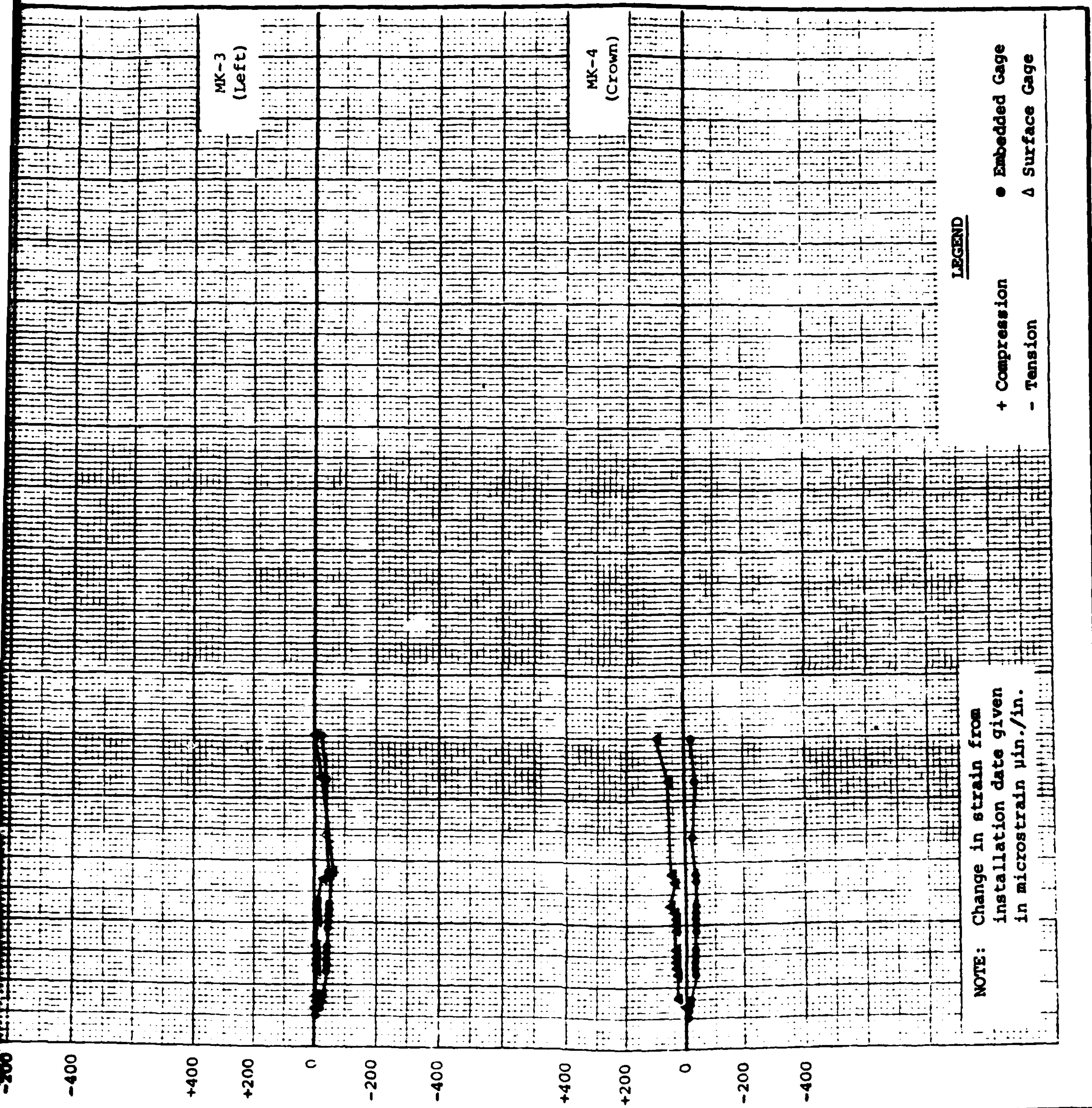
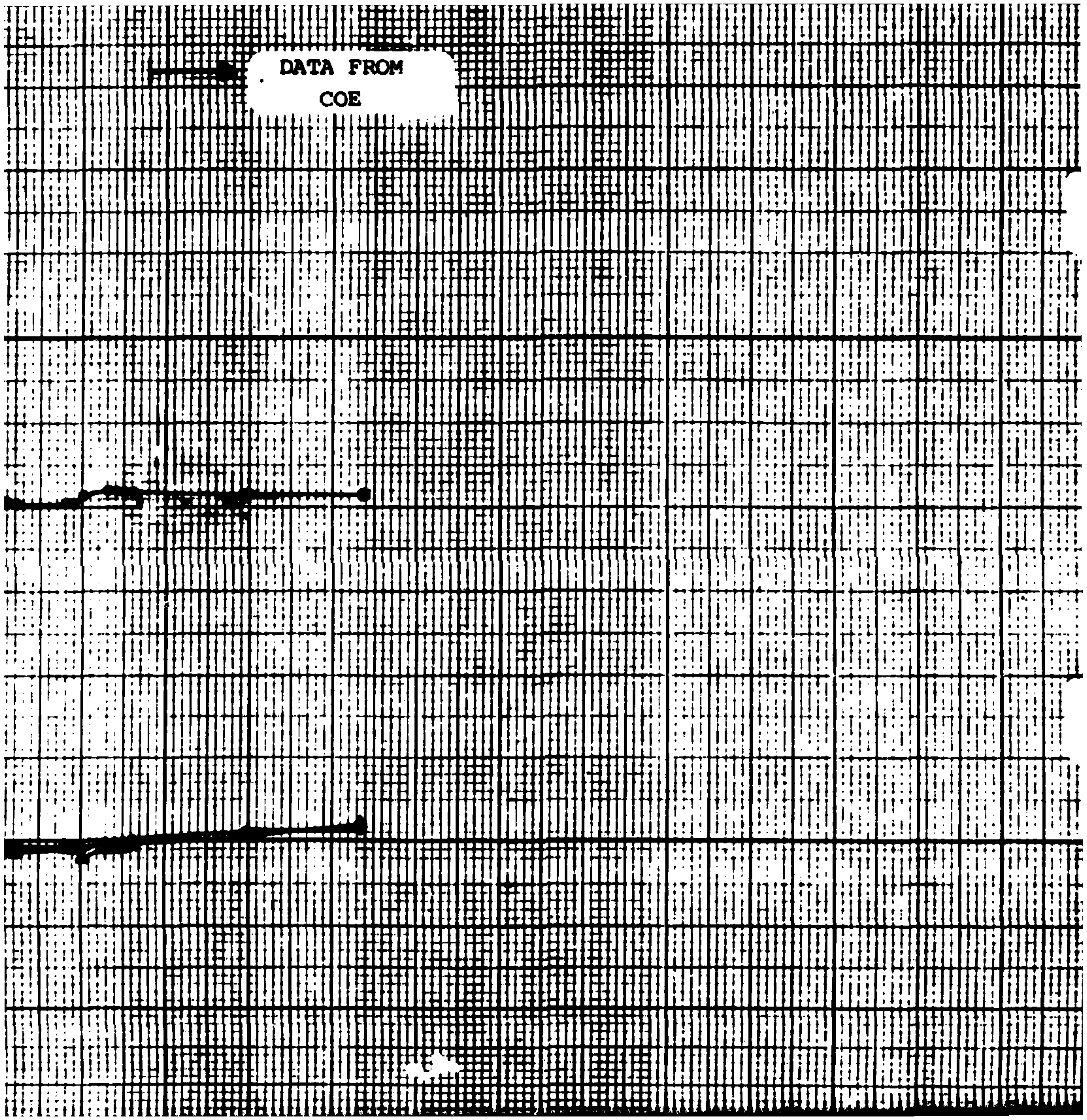
0
-200
-400
MK-2
(Right)



ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 1324 TS9 STA. 91+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-8



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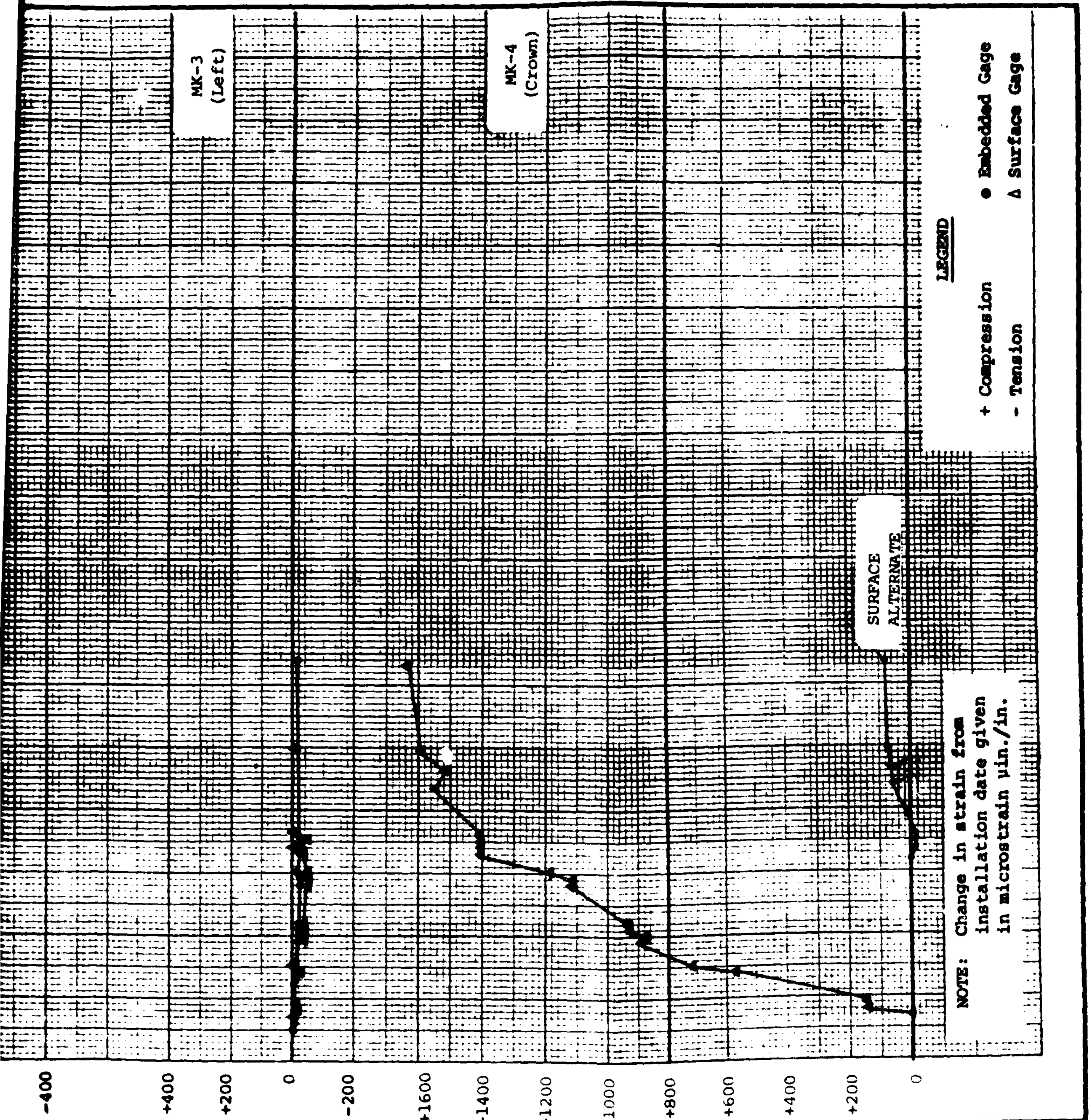
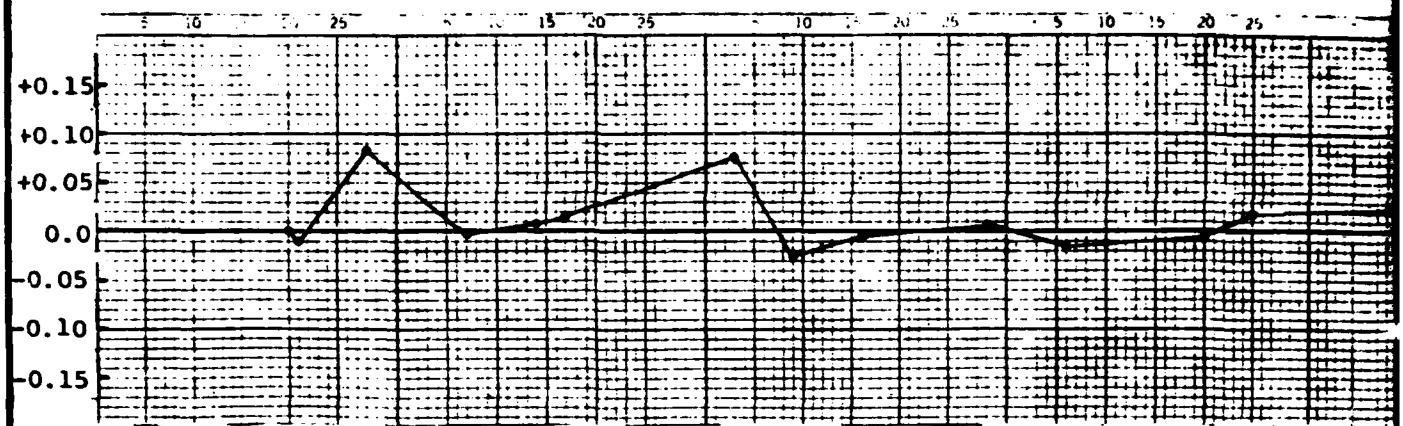


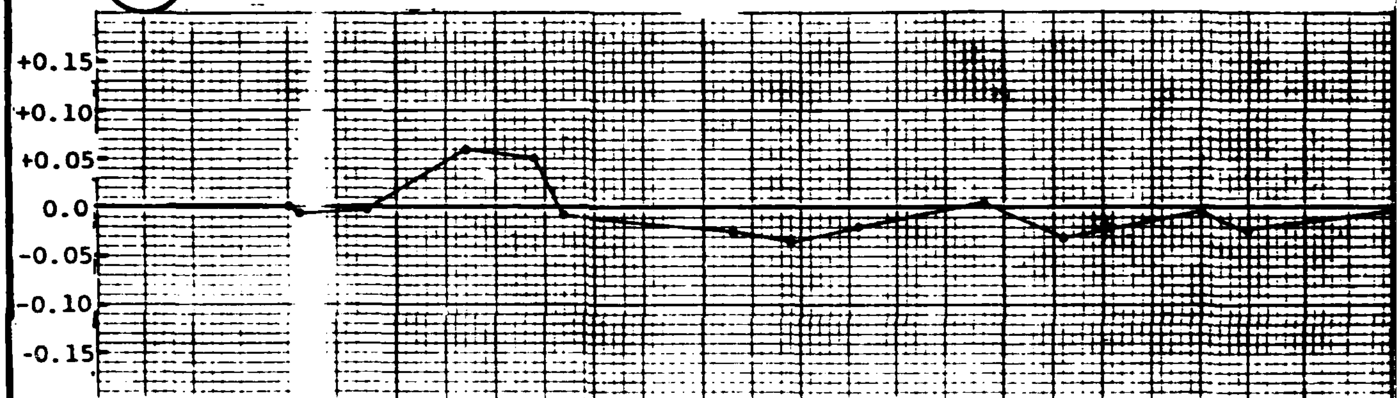
ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	STRAIN GAGE DATA RING # 1399 TS 10 STA. 95+54
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. C-9

2

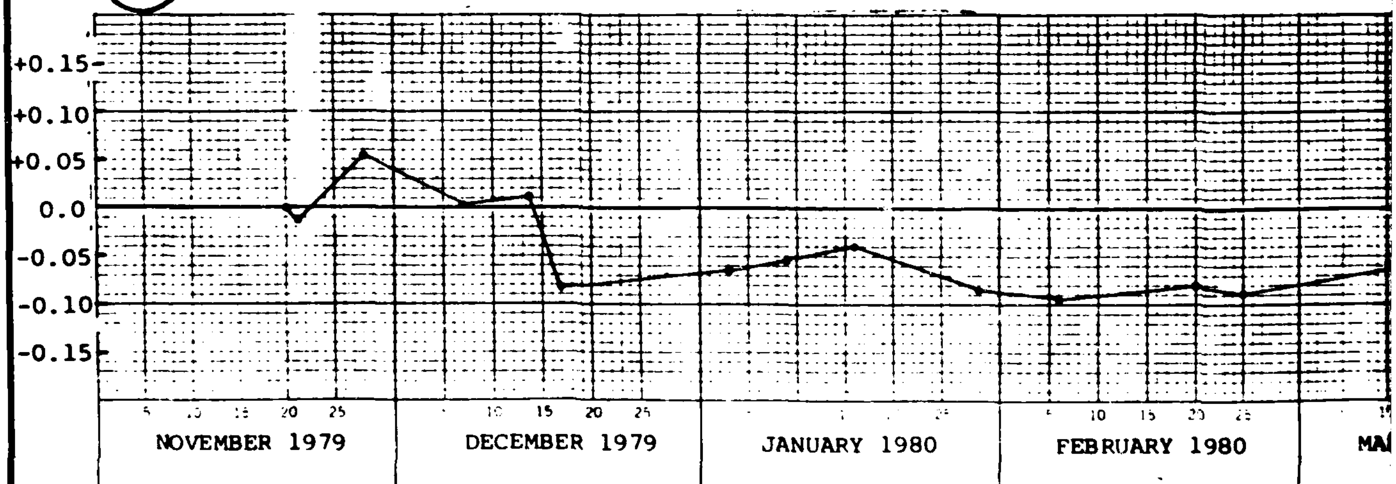
APPENDIX D



Set 8-3 = 10 o'clock - 4 o'clock position



Set 7-2 = 8 o'clock - 2 o'clock position



Set 7-3 = 8 o'clock - 4 o'clock position

NOTE: Change in distance given in inches from initial measurement

AD-A125 752 PARK RIVER LOCAL PROTECTION AUXILIARY CONDUIT TUNNEL AS-BUILT FOUNDATION..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV DEC 82

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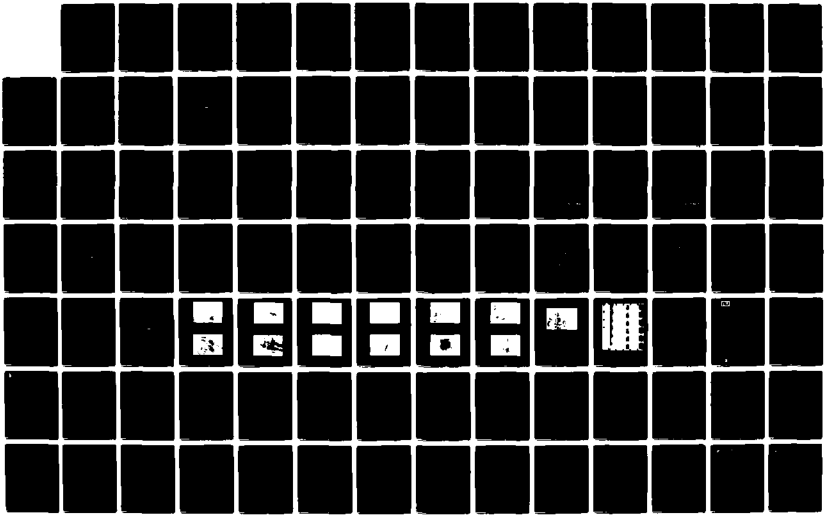
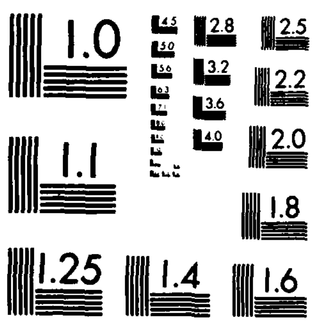
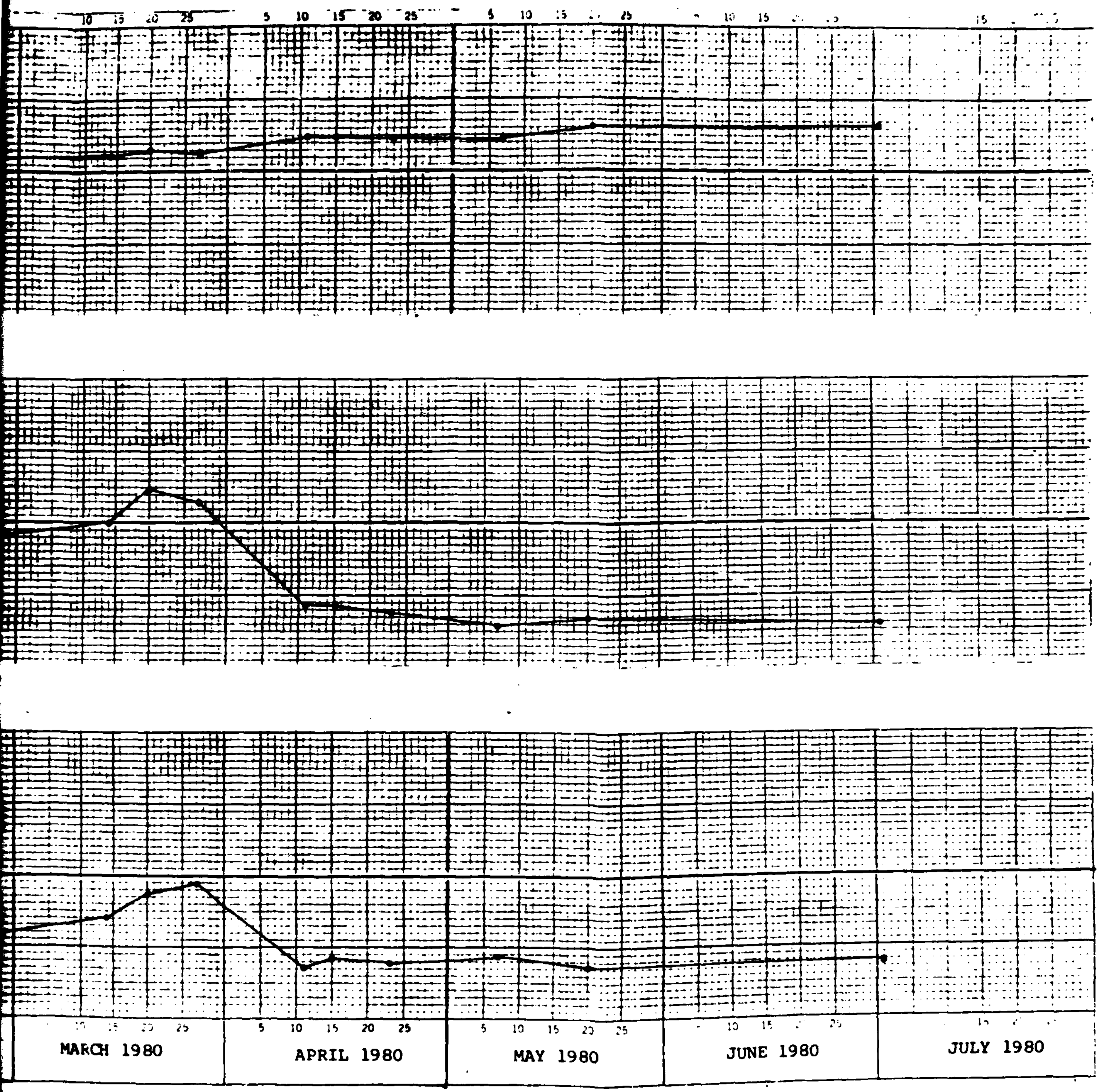


M-2

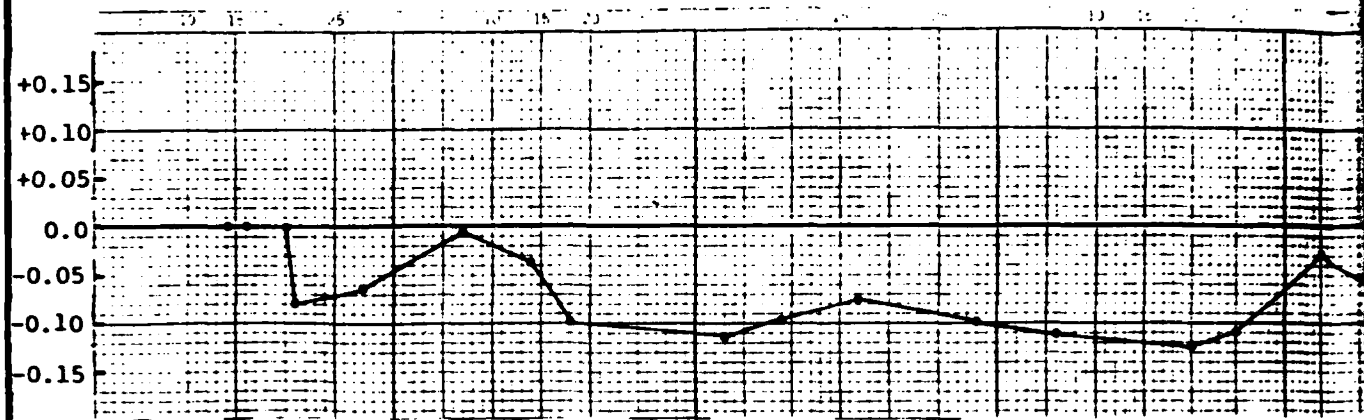


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

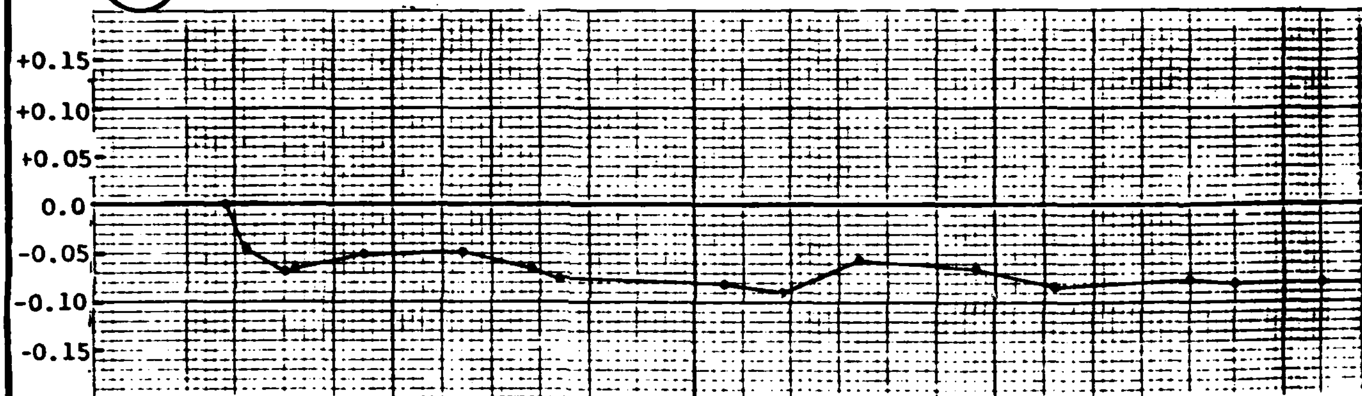


ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	TAPE EXTENSOMETER DATA RING NO. 15 TS 2 STA 11+32
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77362	JULY 1980 FIG. D-1

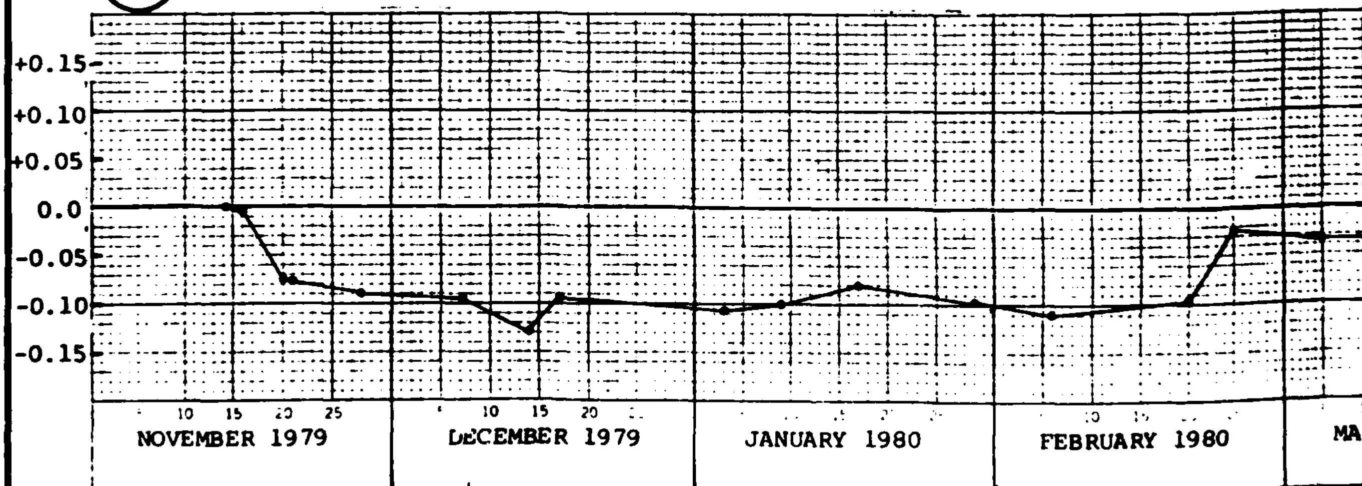
2



Set 8-3 = 10 o'clock - 4 o'clock position

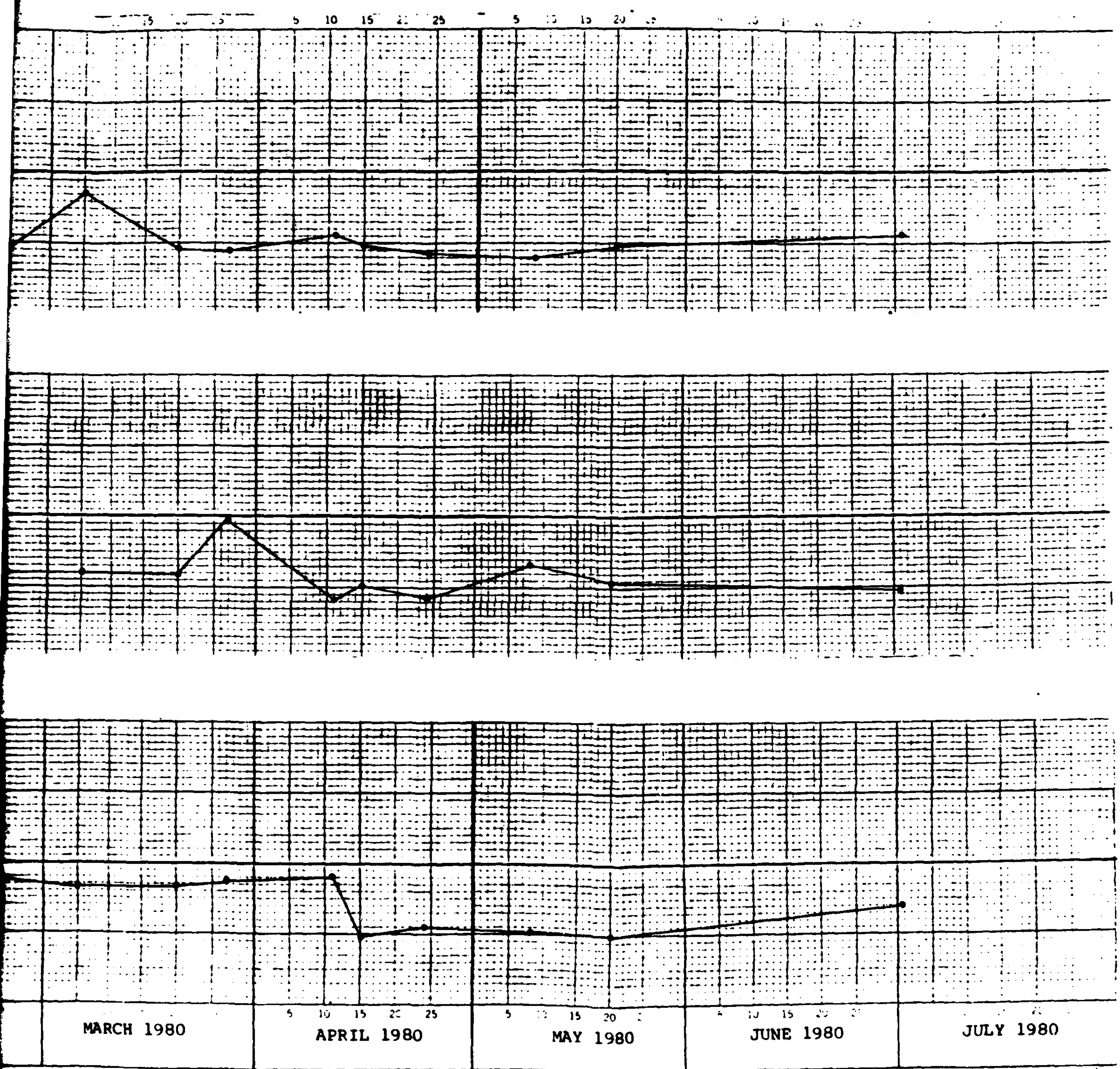


Set 7-2 = 8 o'clock - 2 o'clock position



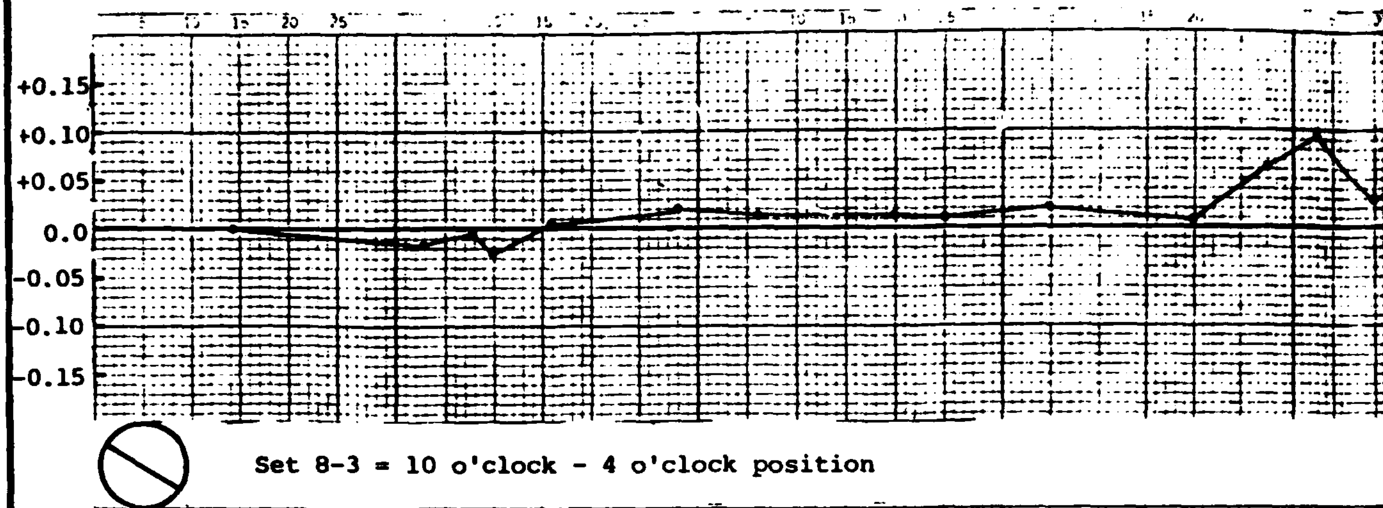
Set 7-3 = 8 o'clock - 4 o'clock position

NOTE: Change in distance given in inches from initial measurement

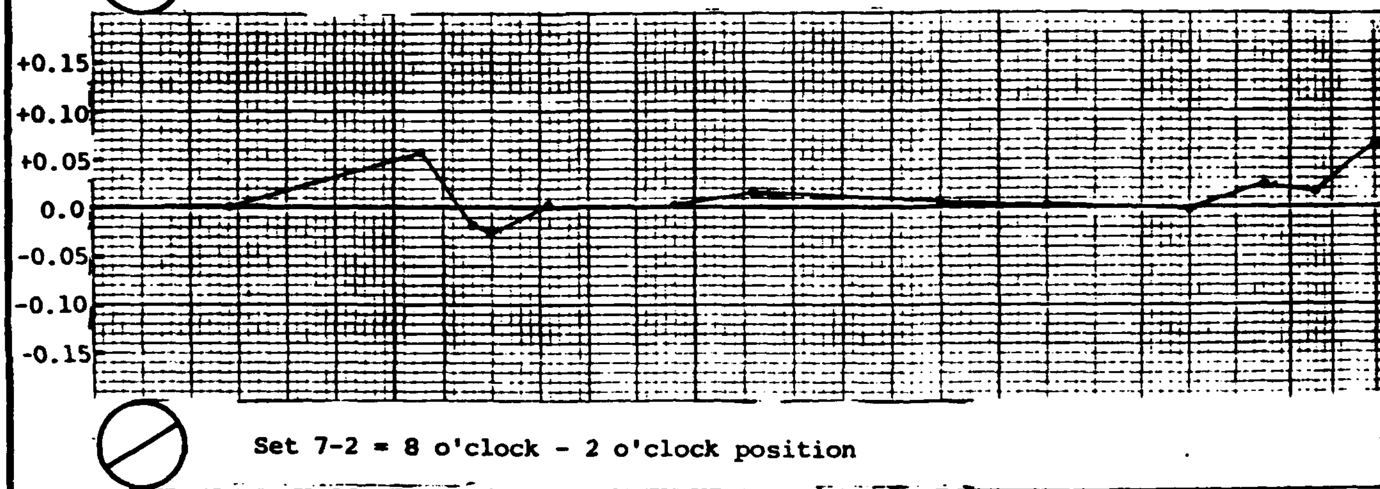


ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	TAPE EXTENSOMETER DATA RING NO. 81 TS 3 STA. 15+25
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. D-2

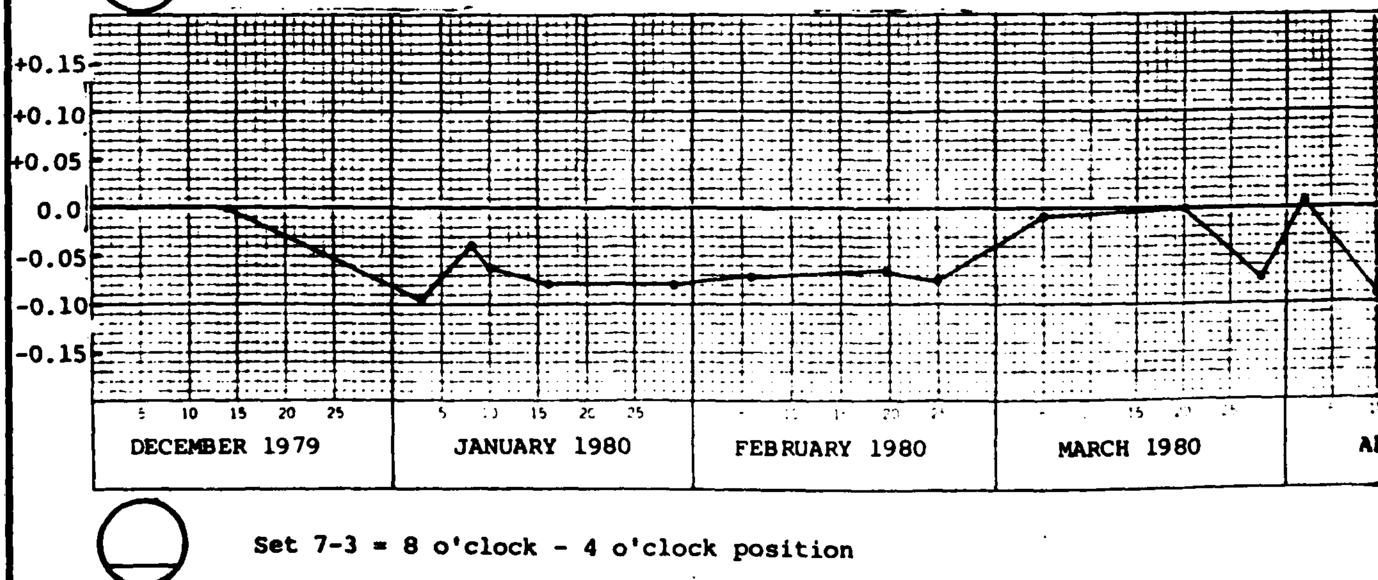
2



Set 8-3 = 10 o'clock - 4 o'clock position

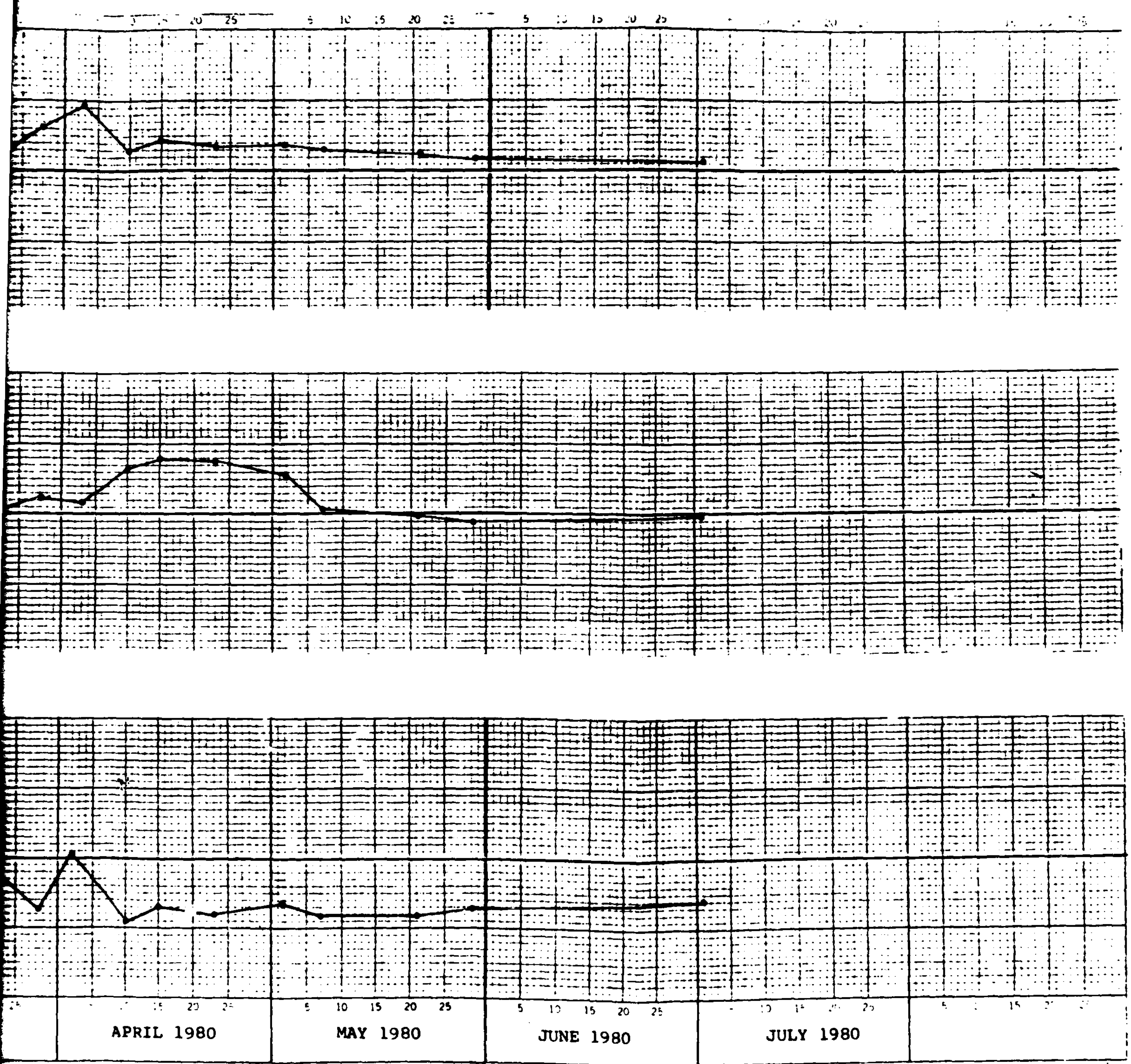


Set 7-2 = 8 o'clock - 2 o'clock position



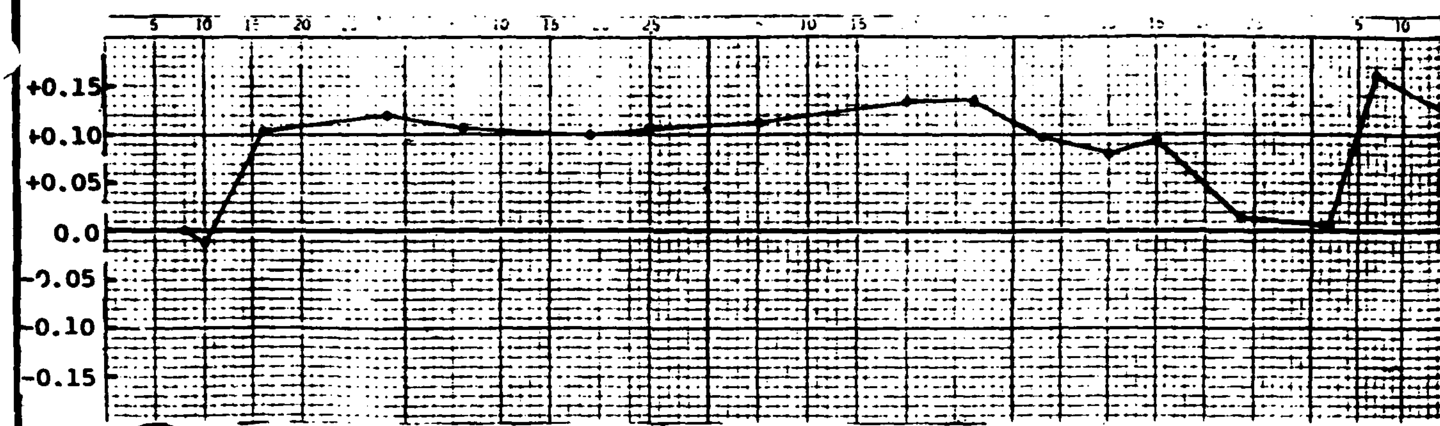
Set 7-3 = 8 o'clock - 4 o'clock position

NOTE: Change in distance given in inches from initial measurement

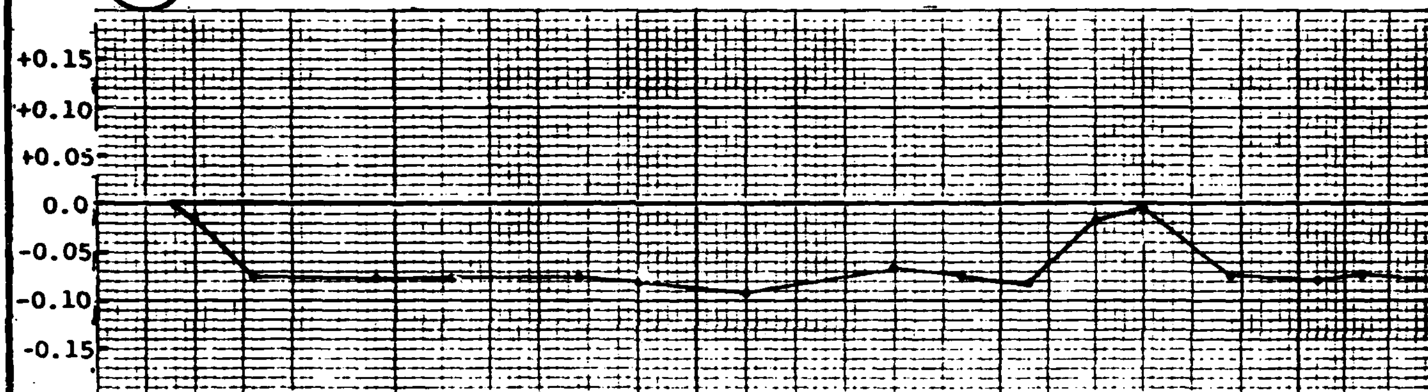


ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	TAPE EXTENSOMETER DATA RING NO. 225 TS4 STA. 24-00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. D-3

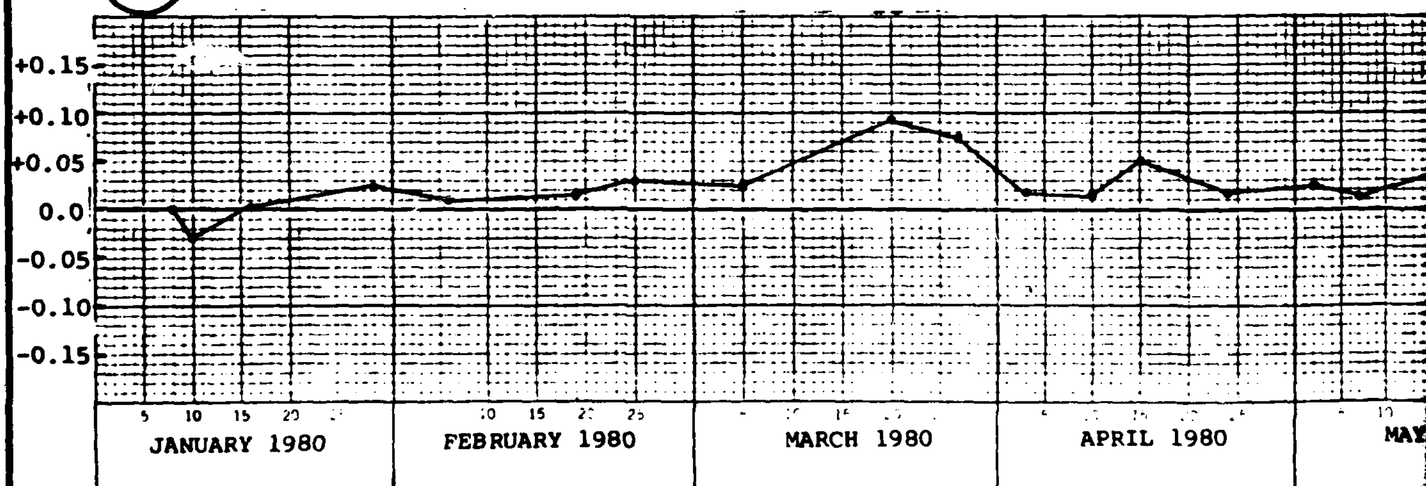
2



Set 8-3 = 10 o'clock - 4 o'clock position

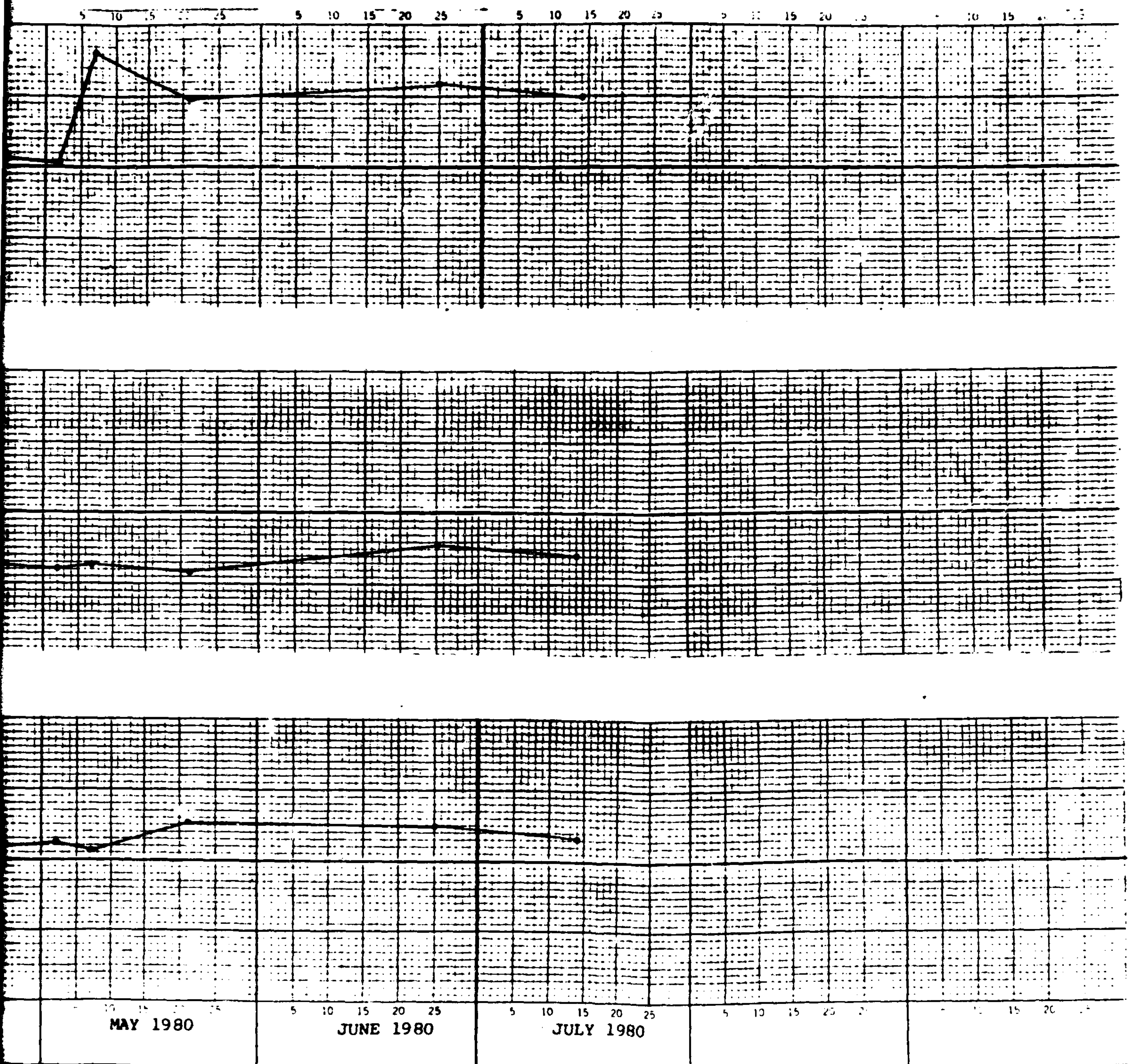


Set 7-2 = 8 o'clock - 2 o'clock position



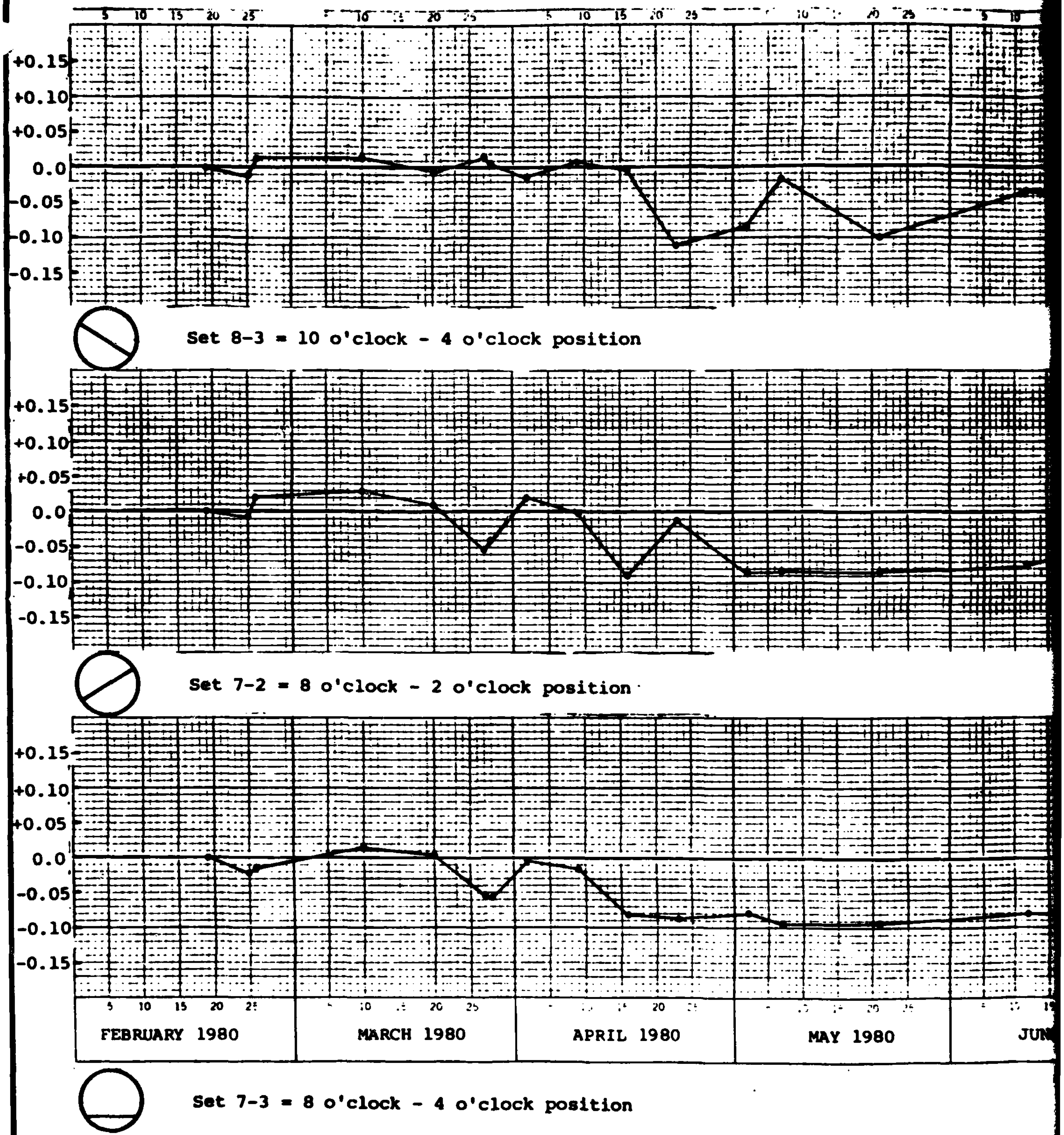
Set 7-3 = 8 o'clock - 4 o'clock position

NOTE: Change in distance given in inches from initial measurement

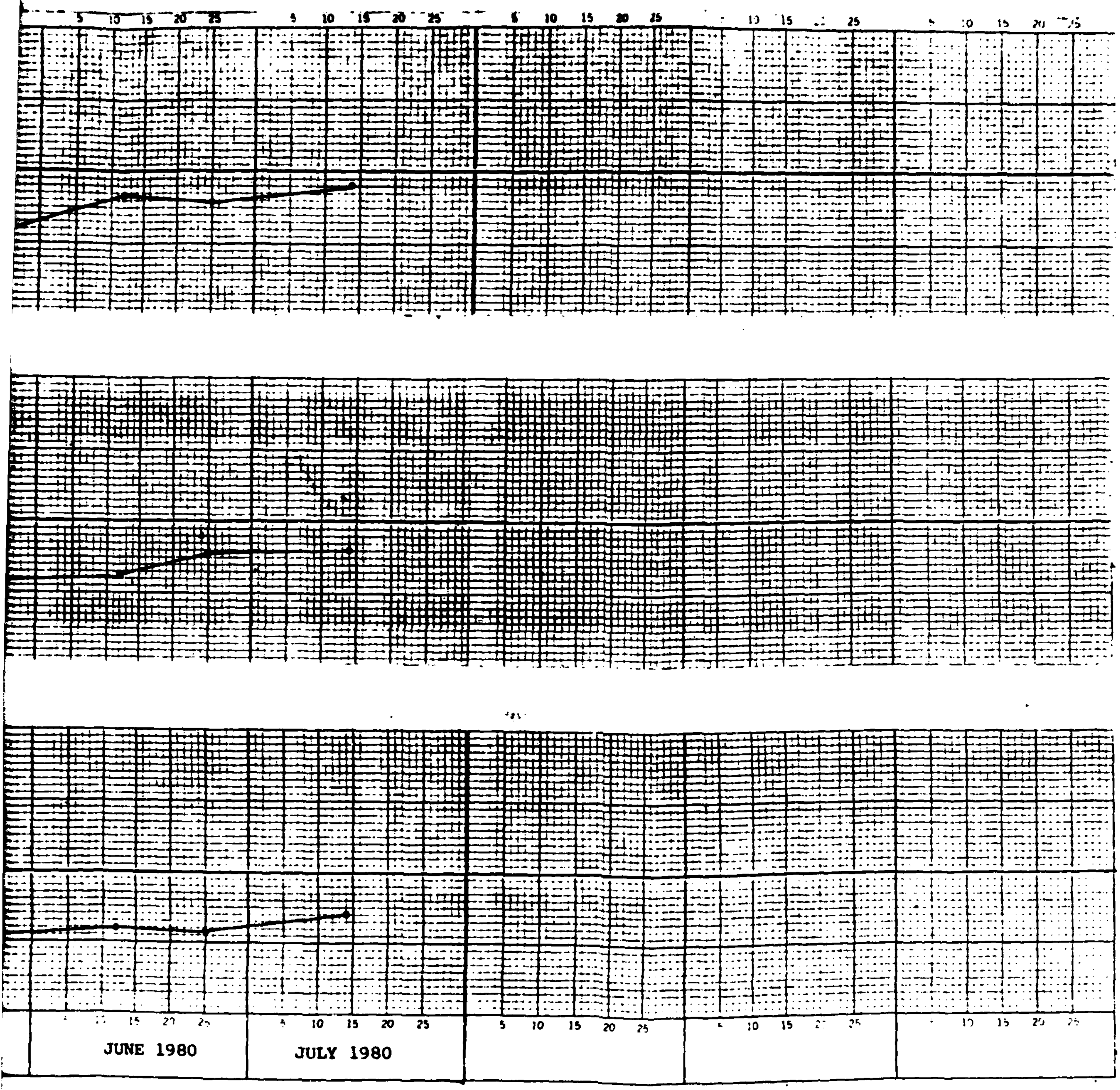


ROGER J. AU & SON INC. MANSFIELD, OHIO GEOTECHNICAL ENGINEERS INC WINCHESTER - MASSACHUSETTS	PARK RIVER AUXILIARY TUNNEL PROJECT 77382	TAPE EXTENSOMETER DATA RING NO. 275 TS 5 STA. 27+00 JULY 1980 FIG. D-4

7

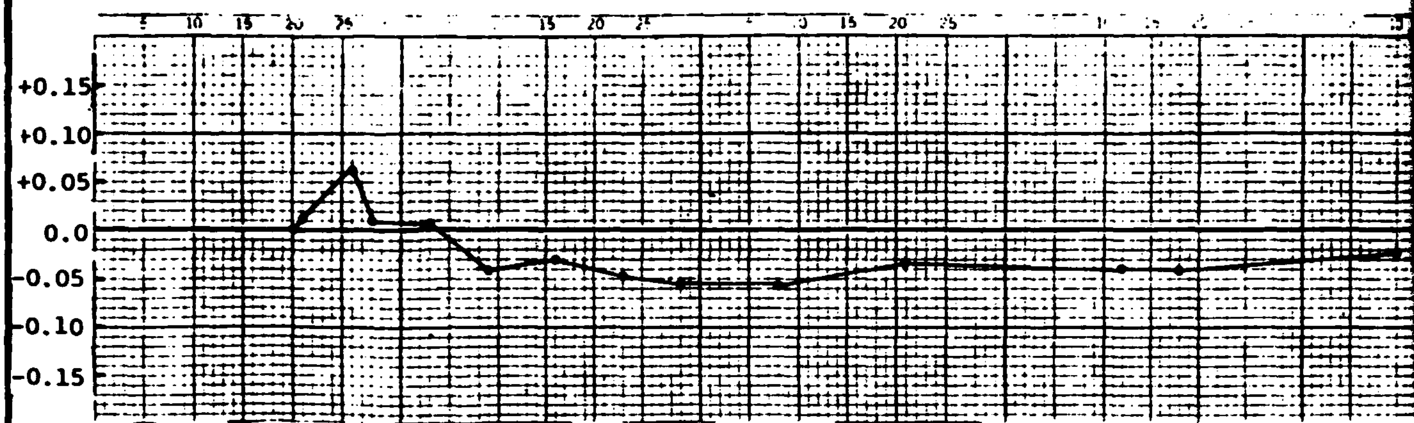


NOTE: Change in distance given in inches from initial measurement

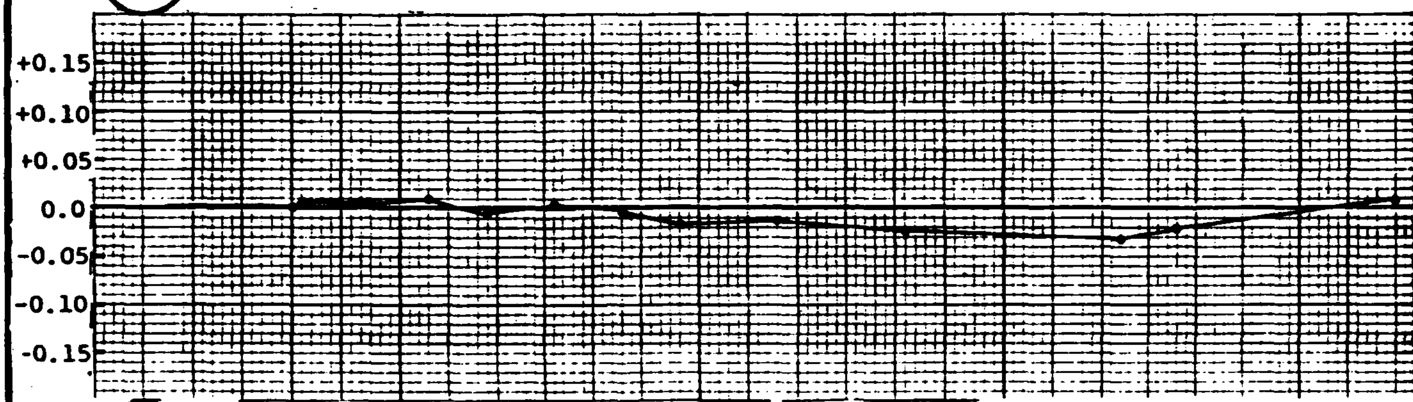


ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	TAPE EXTENSOMETER DATA RING NO. 548 TS 6 STA. 43+50
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. D-5

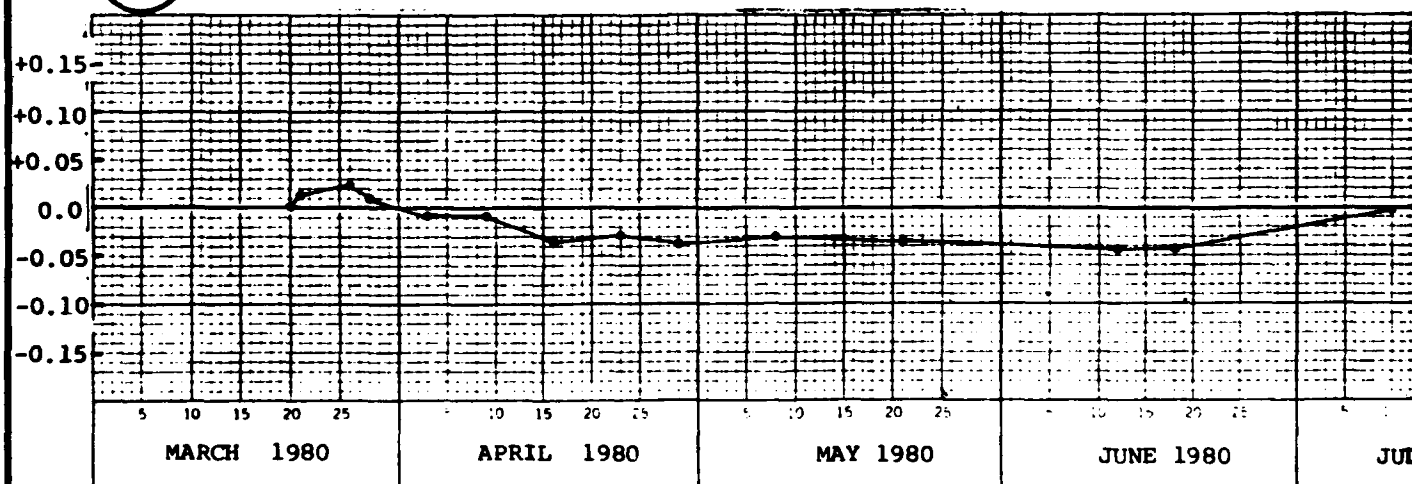
2



Set 8-3 = 10 o'clock - 4 o'clock position

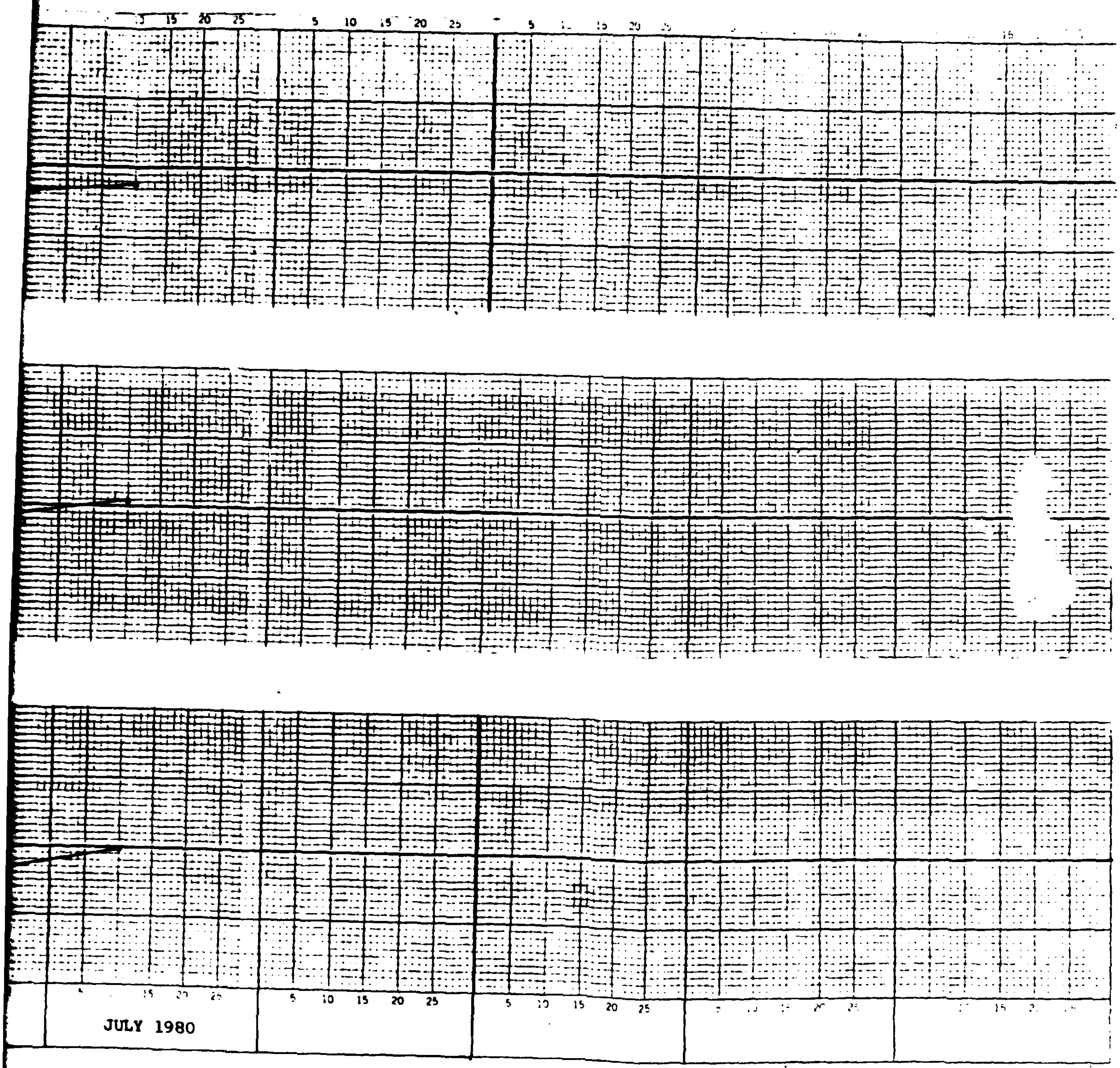


Set 7-2 = 8 o'clock - 2 o'clock position



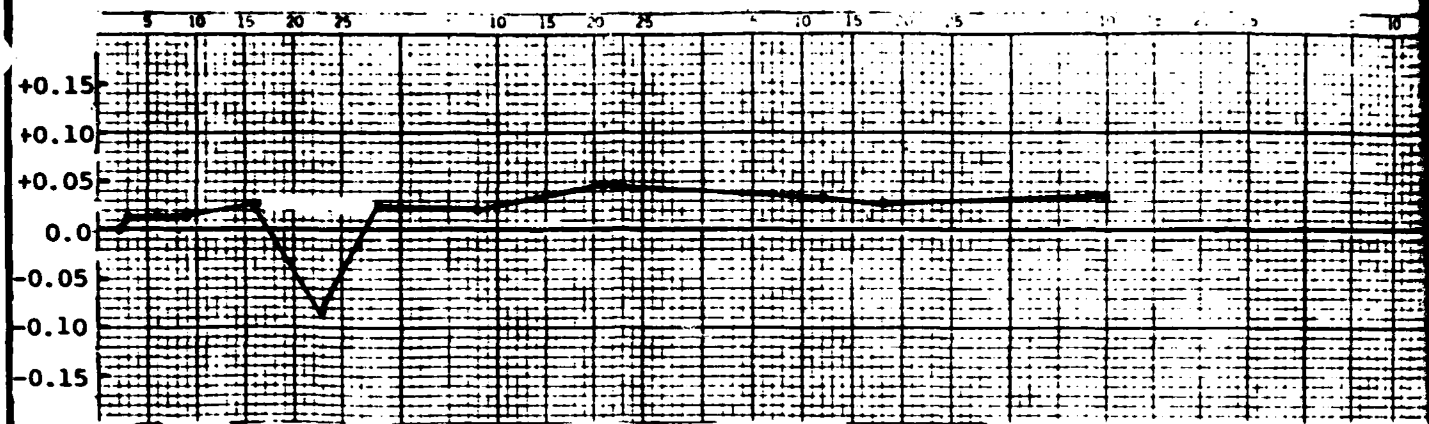
Set 7-3 = 8 o'clock - 4 o'clock position

NOTE: Change in distance given in inches from initial measurement

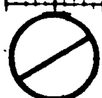


ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	TAPE EXTENSOMETER DATA RING NO. 797 TS 7 STA. 58 + 55
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. D-6

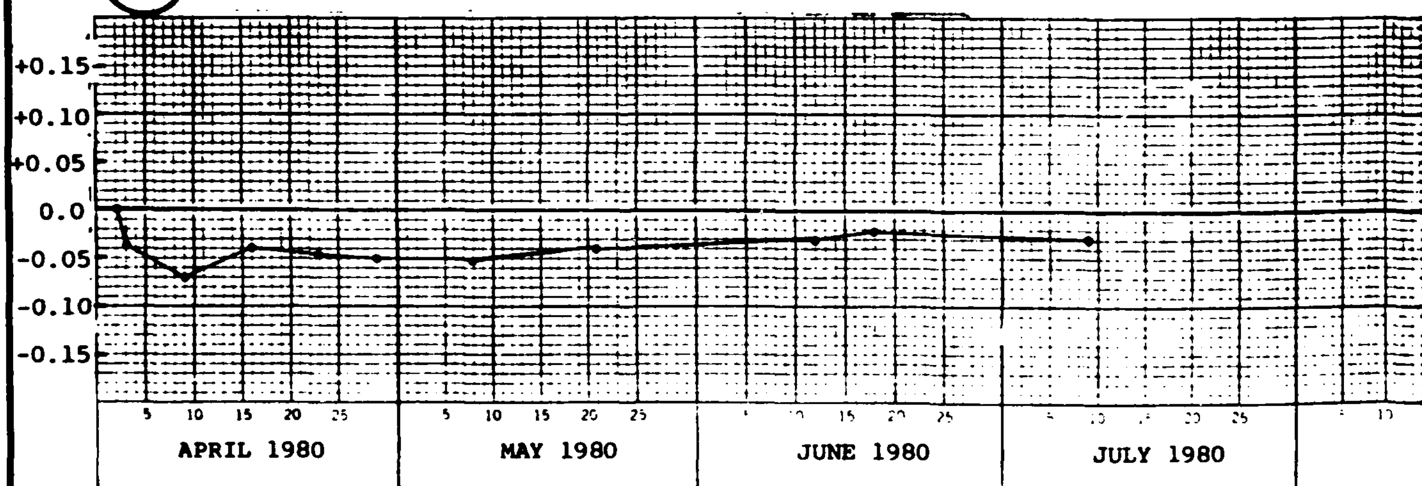
2

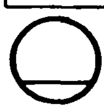


Set 8-3 = 10 o'clock - 4 o'clock position



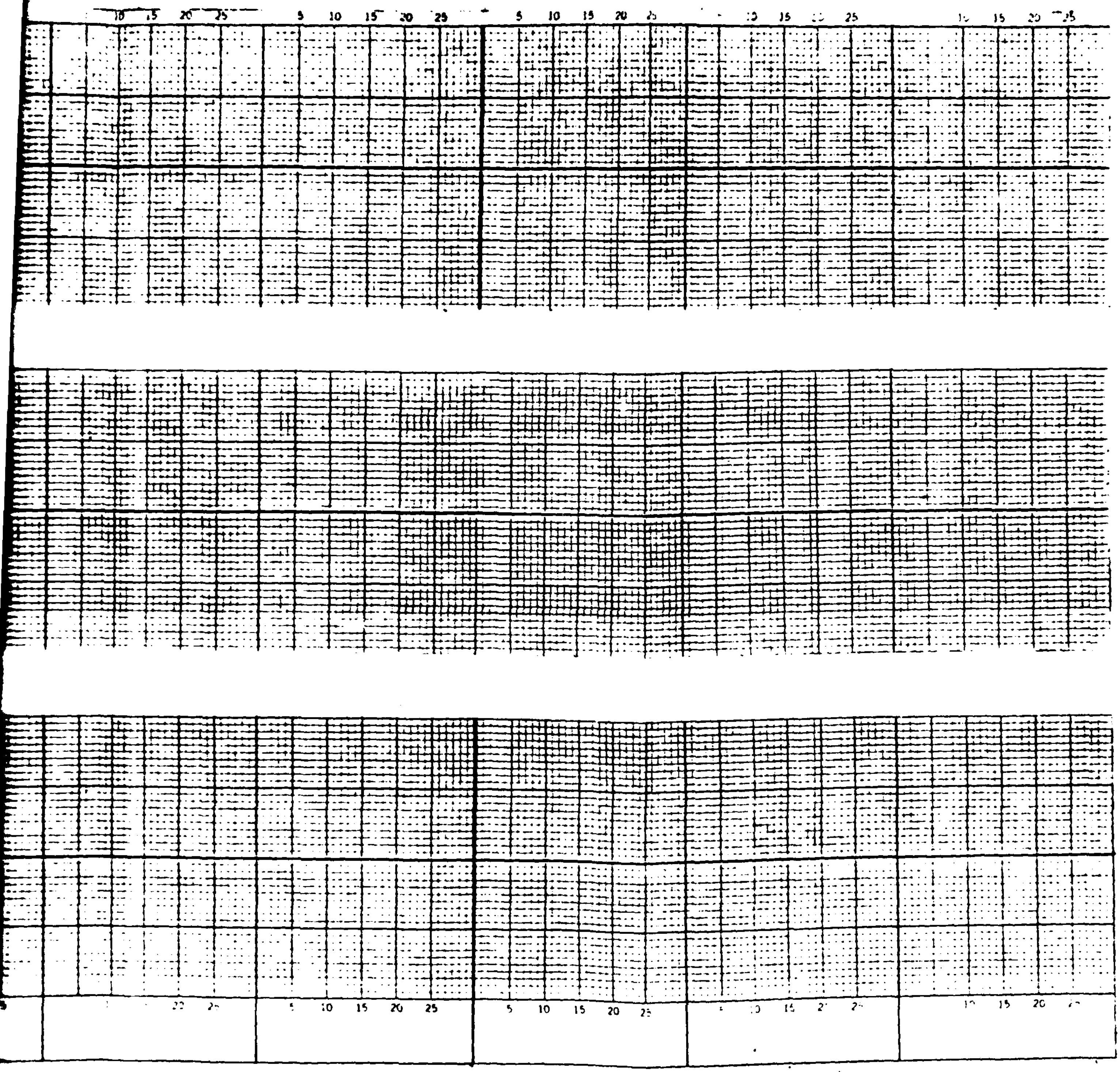
Set 7-2 = 8 o'clock - 2 o'clock position





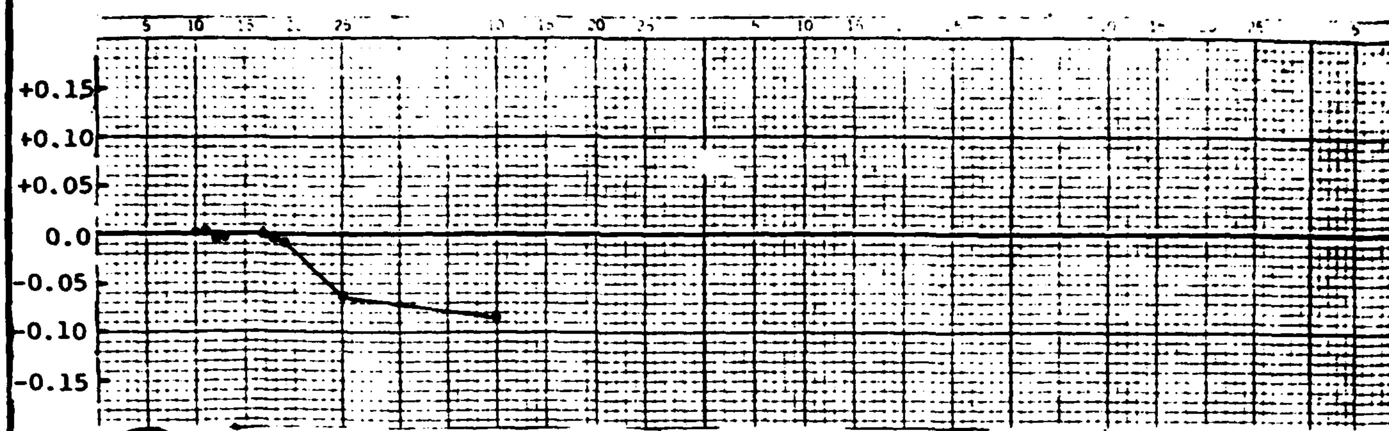
Set 7-3 = 8 o'clock - 4 o'clock position

NOTE: Change in distance given in inches from initial measurement

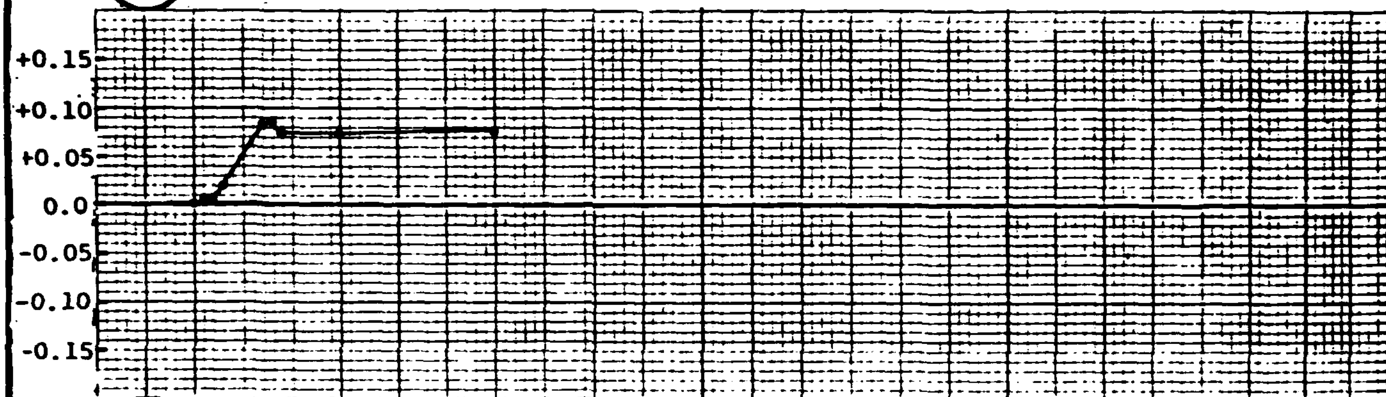


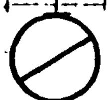
ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	TAPE EXTENSOMETER DATA RING NO. 840 TS 8 STA, 61+19
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. D-7

2

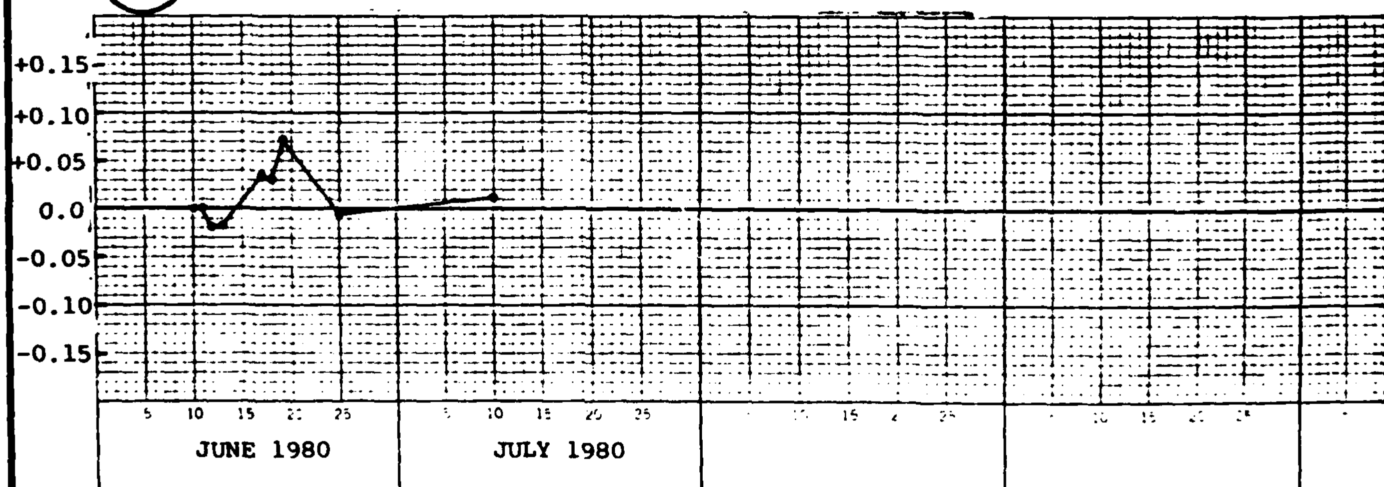


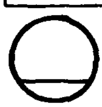
Set 8-3 = 10 o'clock - 4 o'clock position





Set 7-2 = 8 o'clock - 2 o'clock position





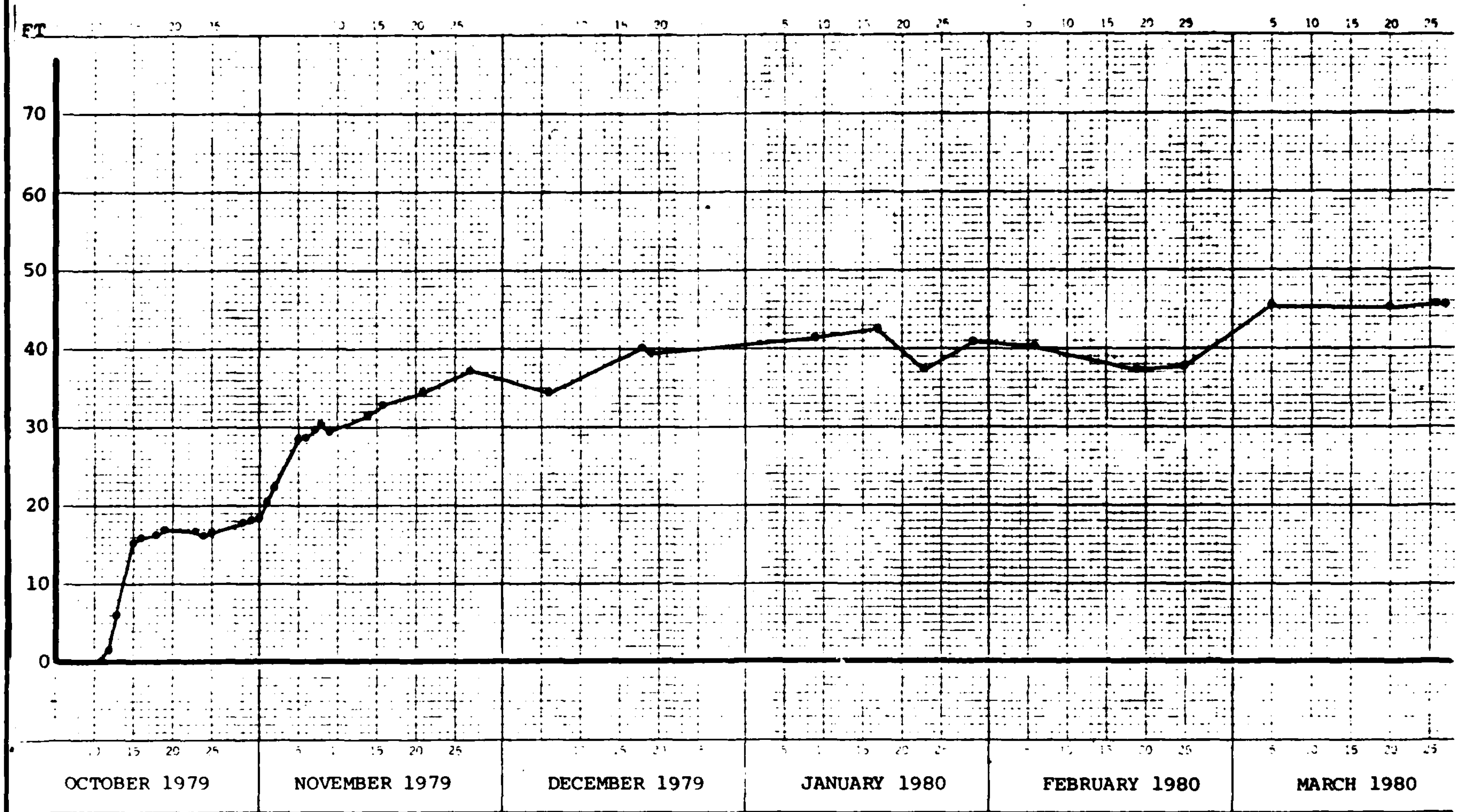
Set 7-3 = 8 o'clock - 4 o'clock position

NOTE: Change in distance given in inches from initial measurement

ROGER J. AU & SON INC. MANSFIELD, OHIO	PARK RIVER AUXILIARY TUNNEL	TAPE EXTENSOMETER DATA RING NO. 1324 TS 9 STA. 91+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	PROJECT 77382	JULY 1980 FIG. D-8

2

APPENDIX E



NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≈ +139

Roger J. Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS

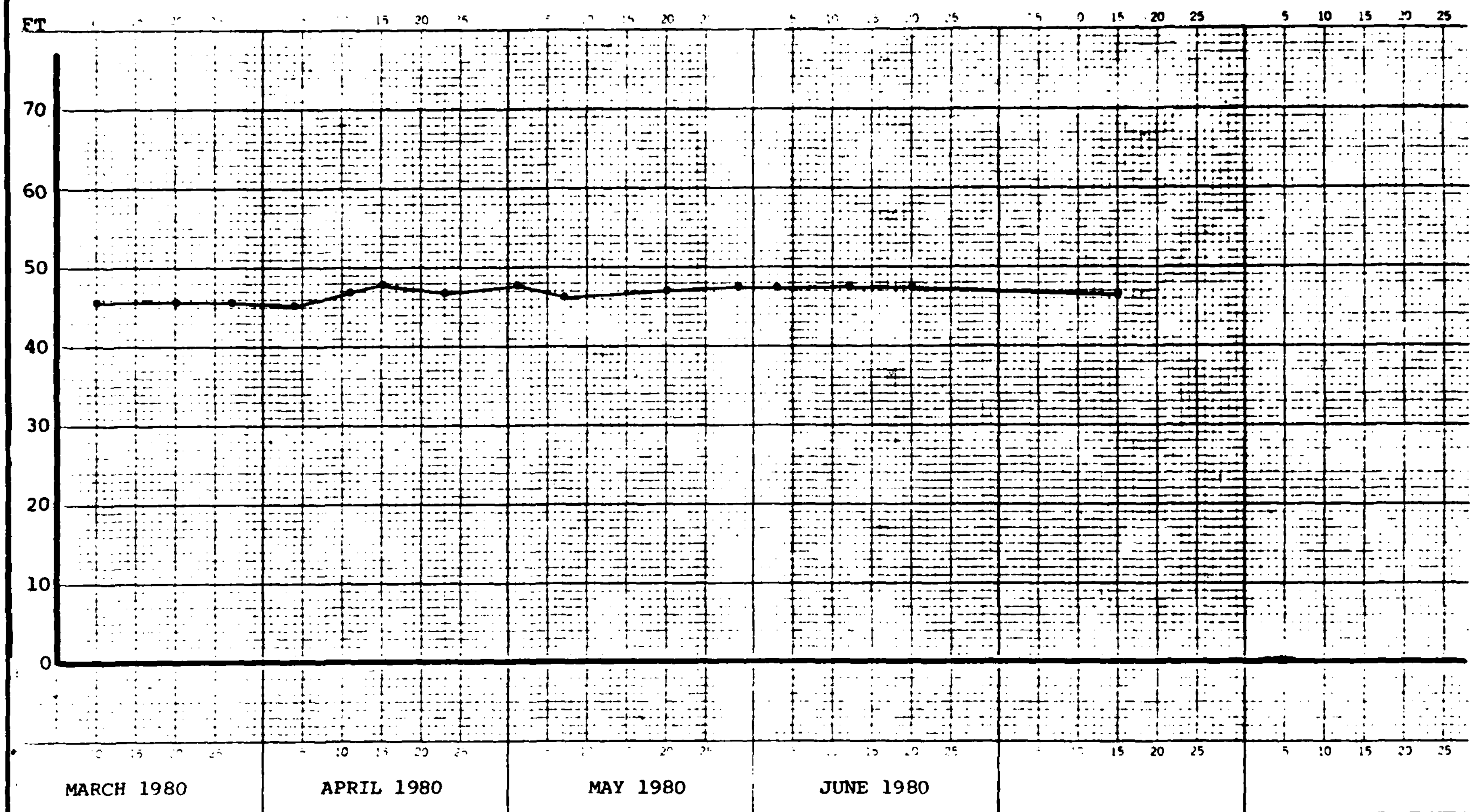
Park River Auxiliary Tunnel
Hartford, CT

Project 77382

TUNNEL PIEZOMETER DATA
Ring # 15
TS2 STA 11+32

July 1980

Fig. E-1



NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≈ -139

Roger J. Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS

Park River Auxiliary Tunnel
Hartford, CT

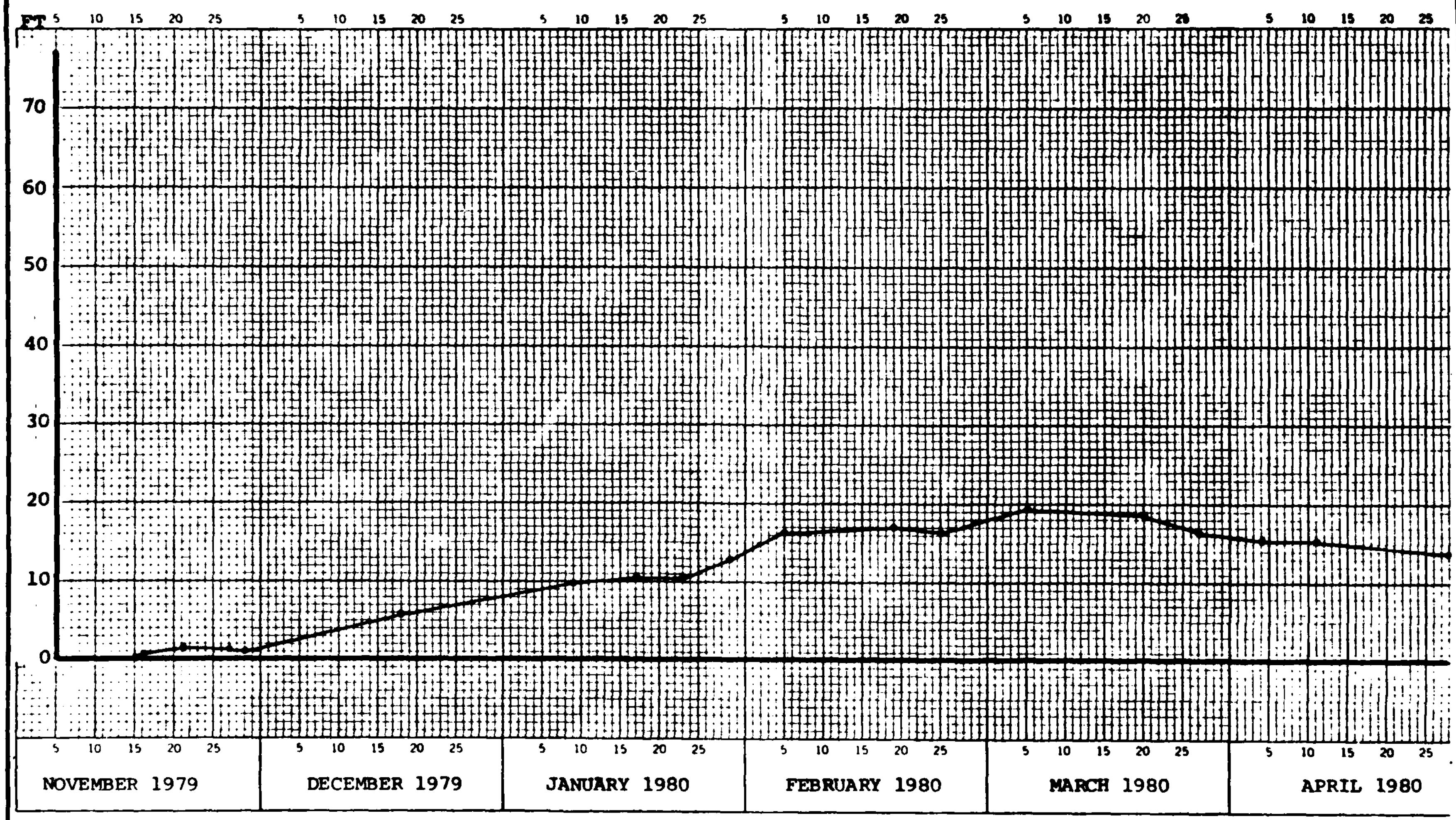
Project 77382

TUNNEL PIEZOMETER DATA
Ring # 15
TS2 STA 11+32

July 1980

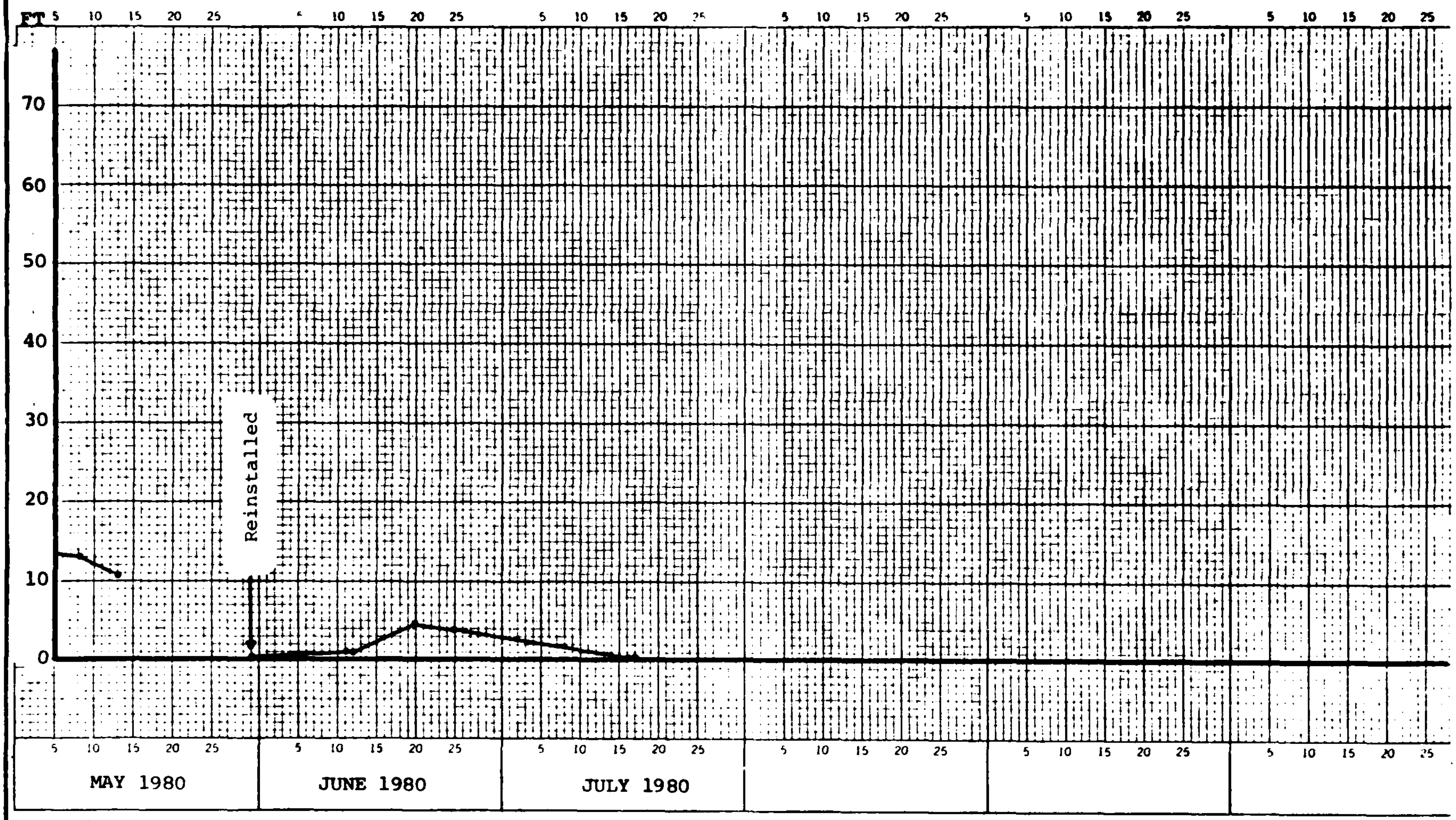
Fig. E-1

(Continued)



NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≈ -137

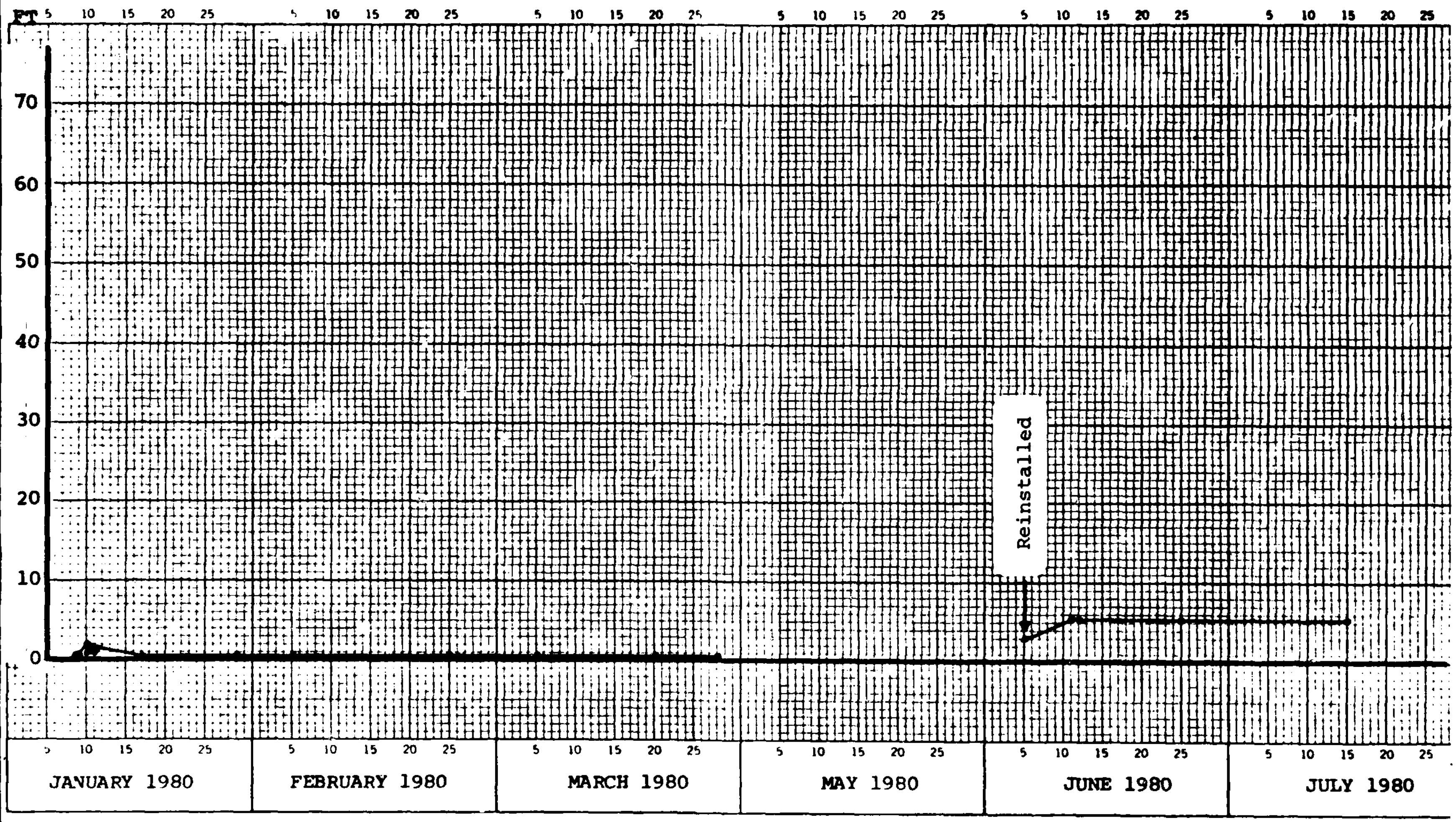
Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	TUNNEL PIEZOMETER DATA Ring # 81 TS3 STA 15+25
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	July 1980 Fig. E-2



NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≈ -137

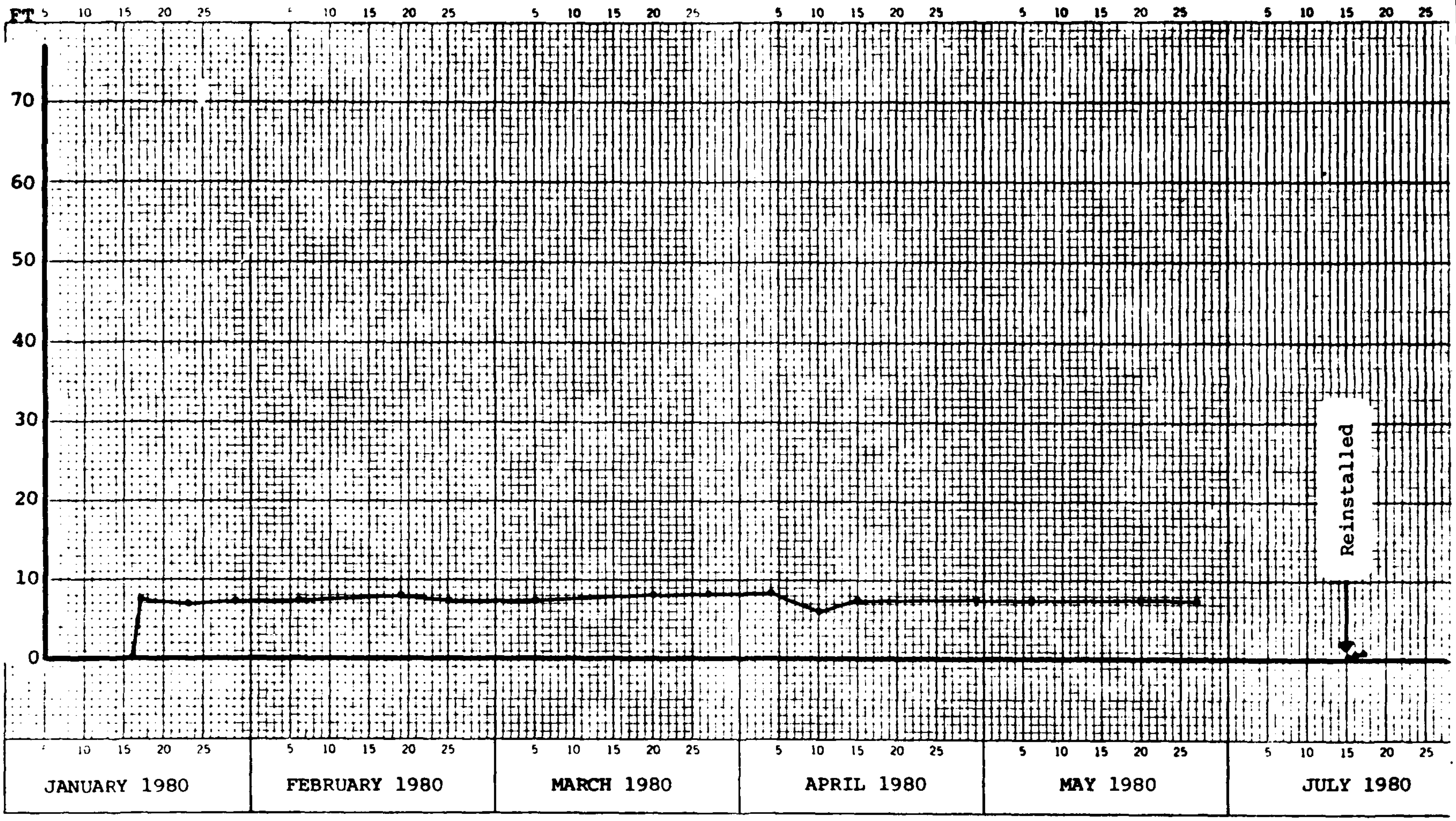
Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	TUNNEL PIEZOMETER DATA Ring # 81 TS3 STA 15+25
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	July 1980 Fig. E-2

(Continued)



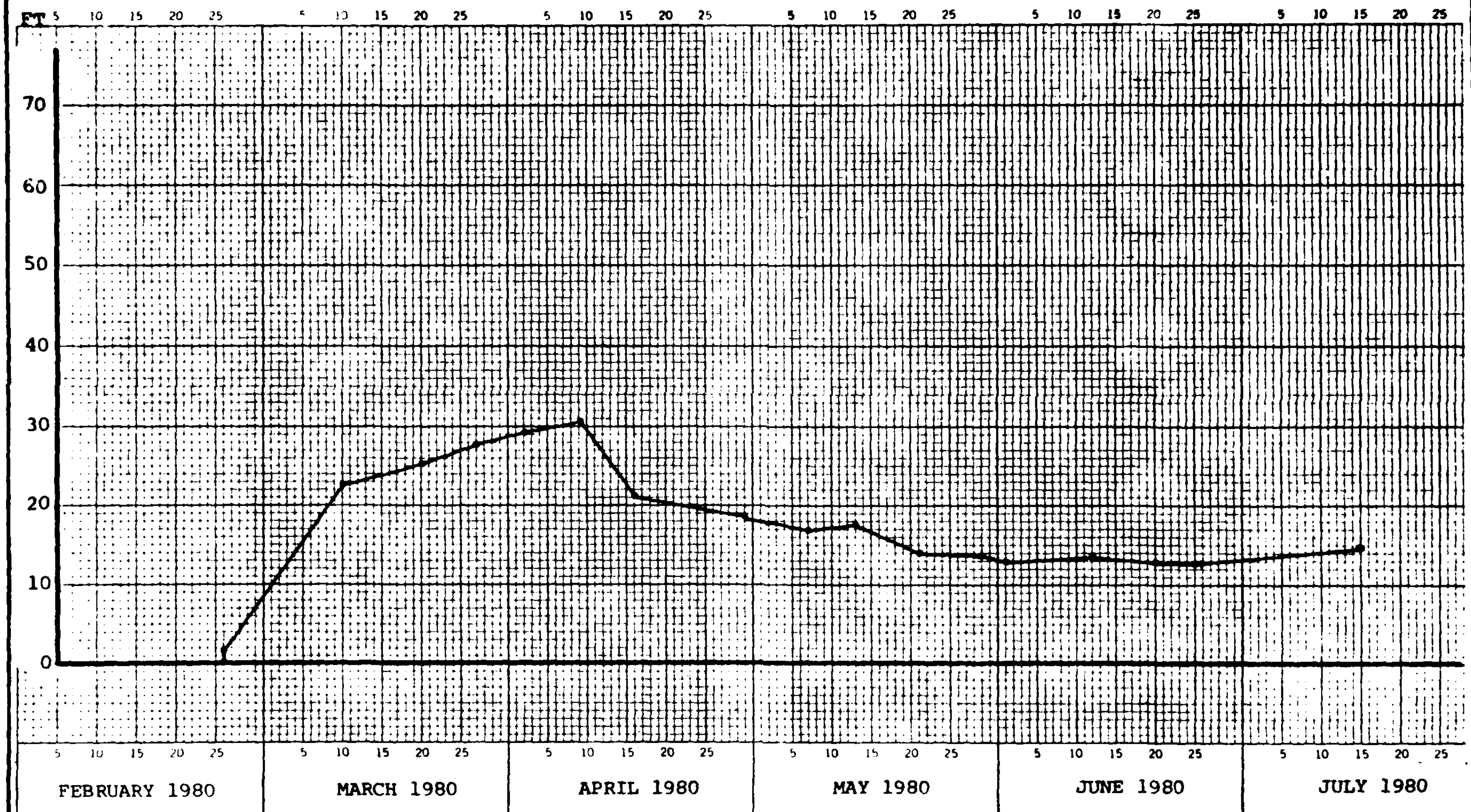
NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ℄ -133

Roger J. Au & Son, Inc. Mansfield, Ohio GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Park River Auxiliary Tunnel Hartford, CT Project 77382	TUNNEL PIEZOMETER DATA Ring # 225 TS4 STA 24+00 July 1980 Fig. E-3



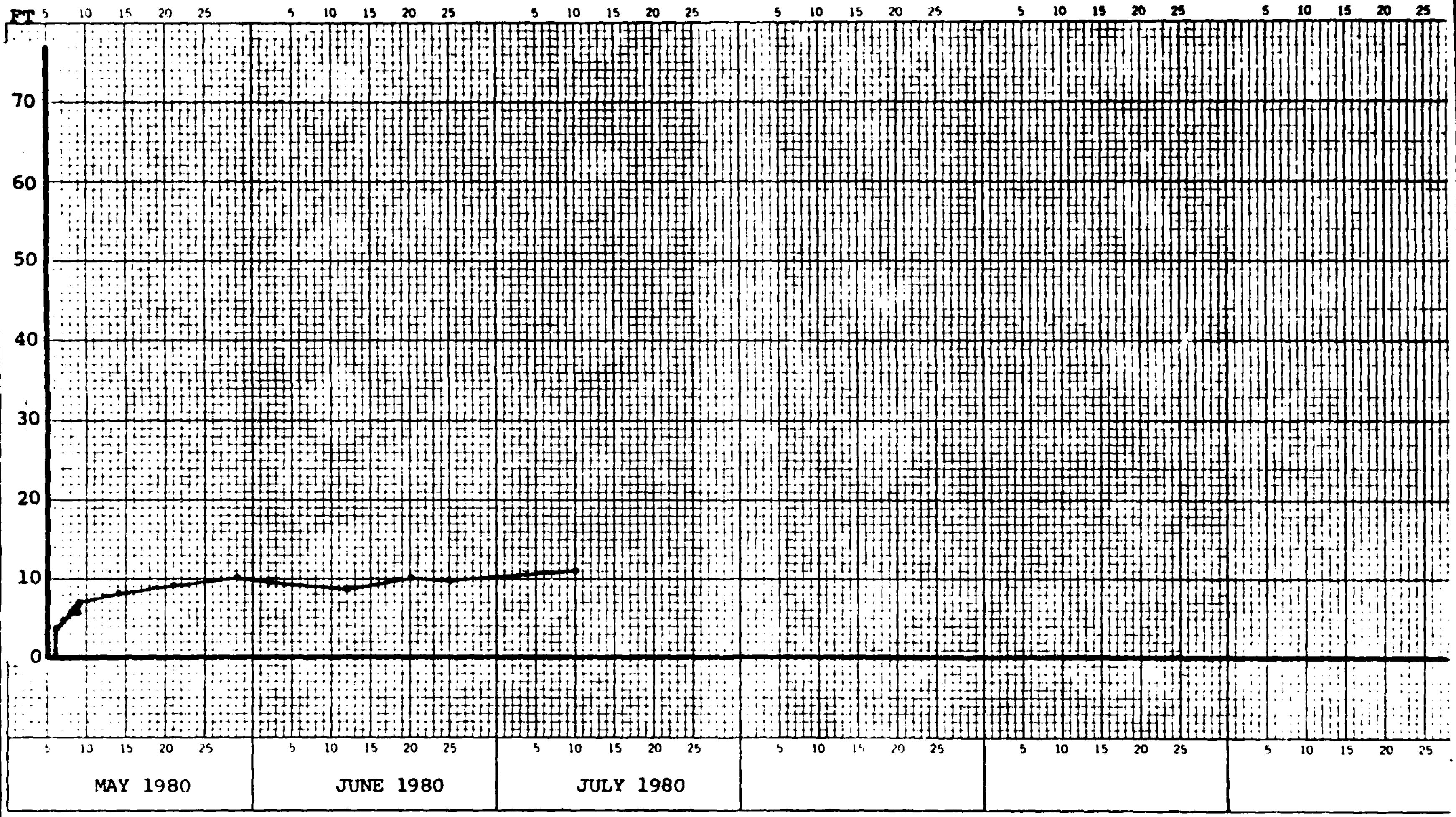
NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≈ -131

Roger J. Au & Son, Inc. Mansfield, Ohio GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Park River Auxiliary Tunnel Hartford, CT	TUNNEL PIEZOMETER DATA Ring # 275 TS5 STA 27+00
	Project 77382	July 1980 Fig. E-4



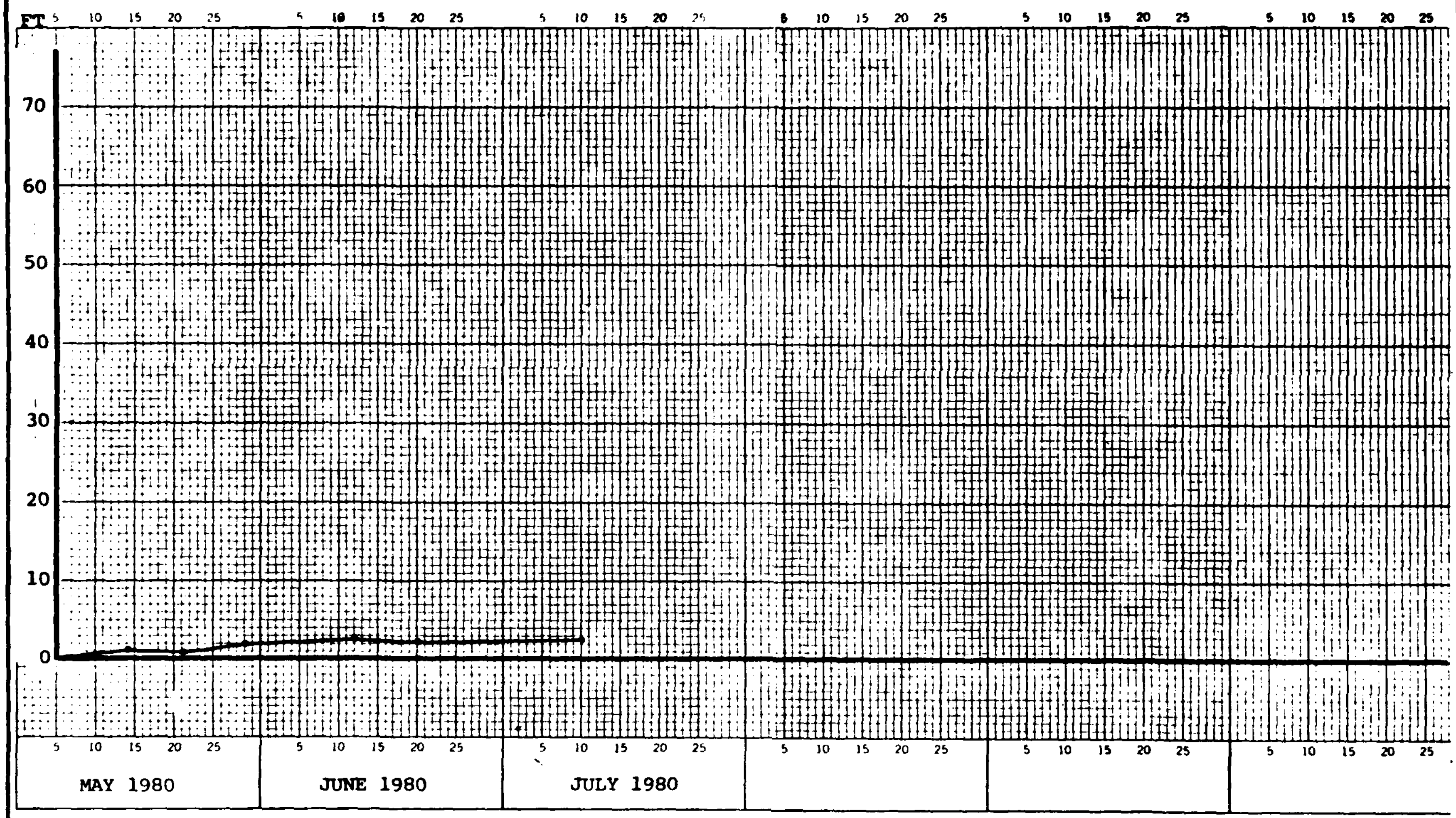
NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≌ -121

Roger J. Au & Son, Inc. Mansfield, Ohio GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Park River Auxiliary Tunnel Hartford, CT	TUNNEL PIEZOMETER DATA Ring # 548 TS6 STA 43+50
	Project 77382	July 1980 Fig. E-5



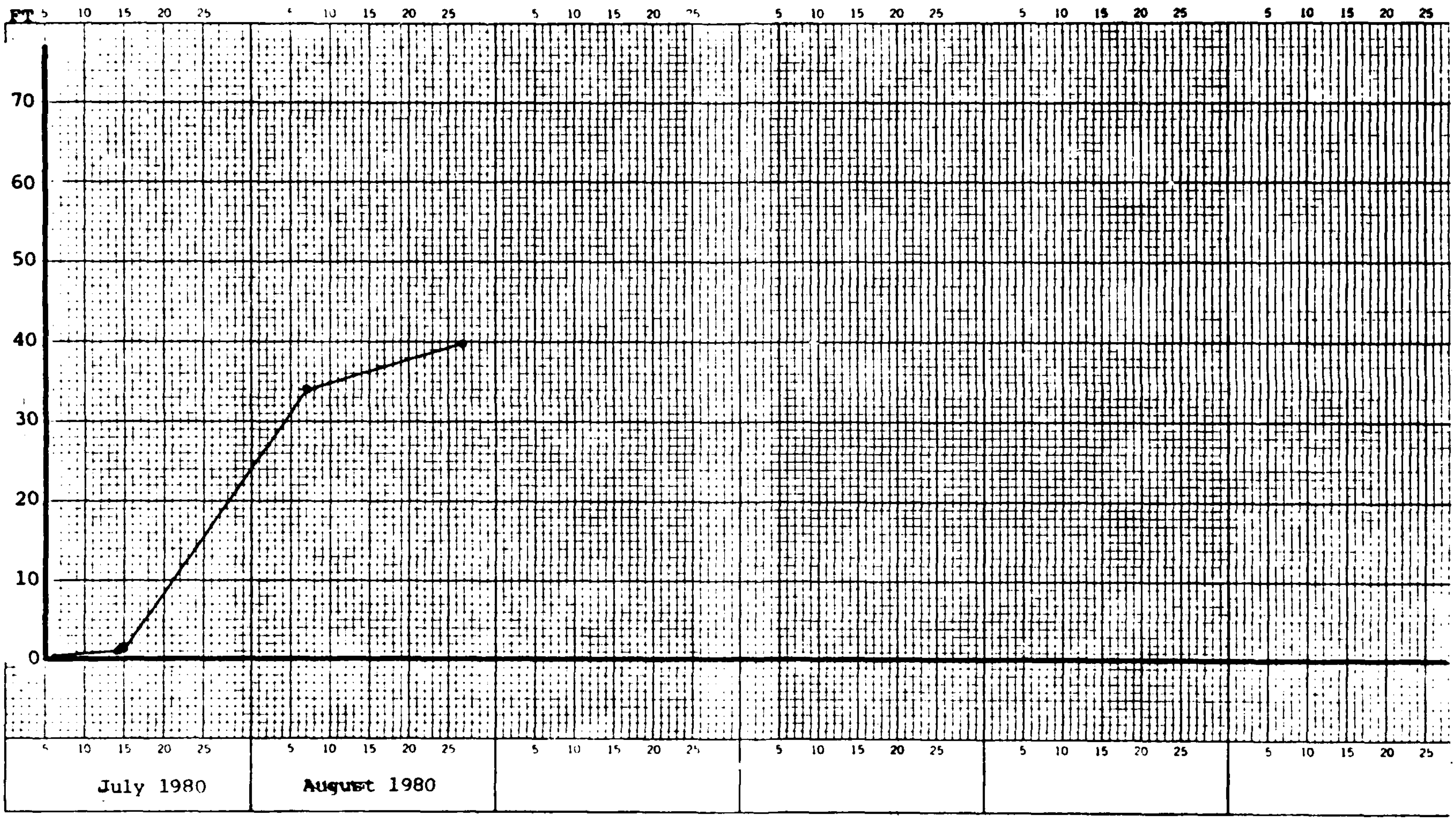
NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≅ -112

Roger J. Au & Son, Inc. Mansfield, Ohio GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Park River Auxiliary Tunnel Hartford, CT Project 77382	TUNNEL PIEZOMETER DATA Ring # 797 TS7 STA 58+55 July 1980 Fig. E-6



NOTE: Piezometric level given in feet of H_2O above instrument. Instrument elevation ≈ -111

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	TUNNEL PIEZOMETER DATA Ring # 840 TS8 STA 61+19
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	July 1980 Fig. E-7



NOTE: 1 Piezometric level given in feet of H_2O above instrument. Instrument elevation ≈ -93

2 Data for 8/7/70 and 8/27/80 supplied by COE.

Roger J. Au & Son, Inc.
Mansfield, Ohio

GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS

Park River Auxiliary Tunnel
Hartford, CT

Project 77382

TUNNEL PIEZOMETER DATA
Ring #1324
TS9 STA 91+00

August 1980 Fig. E-8

APPENDIX F

SEE VOLUME I PLATE NO. 31

FIGURE F-1

APPENDIX G

HERMAN G. PROTZE
— INC. —
MATERIALS TECHNOLOGIST 36 JACONNET STREET, NEWTON HIGHLANDS, MASS. 02161 (617) 332-8460

December 14, 1979

GEOTECHNICAL ENGINEERS, INC.
WINCHESTER, MASSACHUSETTS

MODULUS OF ELASTICITY OF CONCRETE
HARTFORD TUNNEL PROJECT

REFERENCE NUMBER 79L-1376

DATE RECEIVED 9-17-79

DATE FABRICATED 7-18-79

DATE TESTED 9-19-79 (age 2 months)

IDENTIFICATION Three nominal 4x8" concrete cylinders as fabricated and delivered (minus their molds) by you. Specimens were identified as I, II and II. Directions from you were to test I and one II; holding the second II until results were checked.

METHOD OF TEST Secant modulus of elasticity to 35% of ultimate load using two 4½" electrical strain gages and balanced Wheatstone bridge, in conformity with standard methods

RESULTS

	Strain, inches/inch x 10^5	
Applied Load	I	II
Zero, lb.	0.0	0.0
5,000	7.6	7.5
10,000	15.0	15.0
15,000	26.5	25.5
20,000	36.0	35.3
25,000	46.0	45.0
30,000	57.8	55.1
35,000	70.1	67.4
Modulus of Elasticity	3,700,000ψ	3,800,000ψ
Compressive Strength	7630ψ	7990ψ

Respectfully submitted,

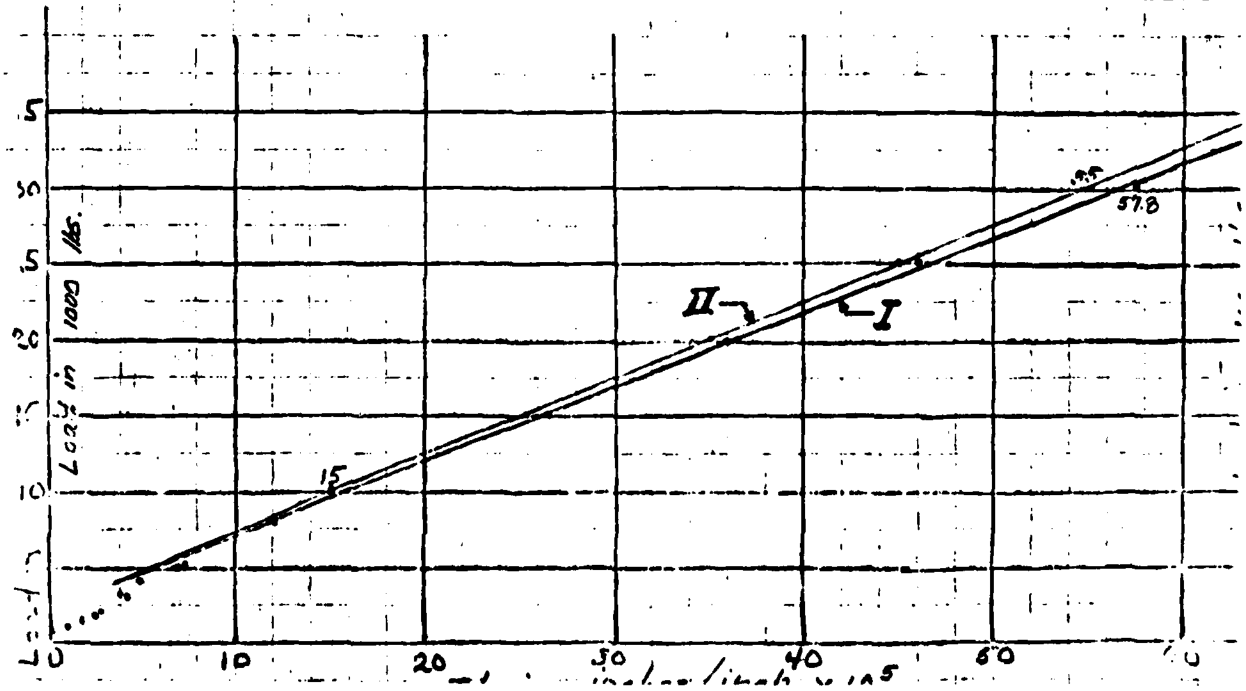
David R. Mitchell

David R. Mitchell

Geotechnical 1

Tested 9-19-79

	I	II
Zero	0	0
500	0	0
1000	2.0	1.0
2000	2.5	2.4
3000	4.0	4.1
4000	5.0	5.0
5000	7.6	7.5
10,000	15.0	15.0
15,000	26.5	25.5
20,000	36.0	35.3
25,000	46.0	45.0
30,000	57.8	55.1
35,000	70.1	67.4
P	102,100	107,000
f'c	7630 psi	7990 psi



HERMAN G. PROTZE
— INC. —
MATERIALS TECHNOLOGIST 36 JACONNET STREET, NEWTON HIGHLANDS, MASS. 02161 (617) 332-8460

December 14, 1979

GEOTECHNICAL ENGINEERS, INC.
WINCHESTER, MASSACHUSETTS

MODULUS OF ELASTICITY OF CONCRETE
<u>HARTFORD TUNNEL PROJECT</u>

REFERENCE NUMBER	79L-1524
DATE RECEIVED	10-16-79
DATE FABRICATED	7-18-79
DATE TESTED	10-19-79 (age 3 months)
IDENTIFICATION	Two nominal 4x8" concrete cylinders as fabricated and delivered by you. Specimens were identified as I and II. Both specimens were tested per your instructions.
METHOD OF TEST	Secant modulus of elasticity to 35% of ultimate load using two 2½" electrical strain gages and balanced Wheatstone bridge, in conformity with standard methods.

RESULTS

Applied Load	Strain, inches/inch x 10^5 I	Strain, inches/inch x 10^5 II
Zero, lb.	0.0	0.0
5,000	8.0	7.9
10,000	15.8	15.8
15,000	24.9	24.0
20,000	34.2	33.2
25,000	43.6	42.8
30,000	53.8	51.8
35,000	64.0	61.5
Modulus of Elasticity	4,000,000ψ	4,000,000ψ
Compressive Strength	7780ψ	7730ψ

Respectfully submitted,

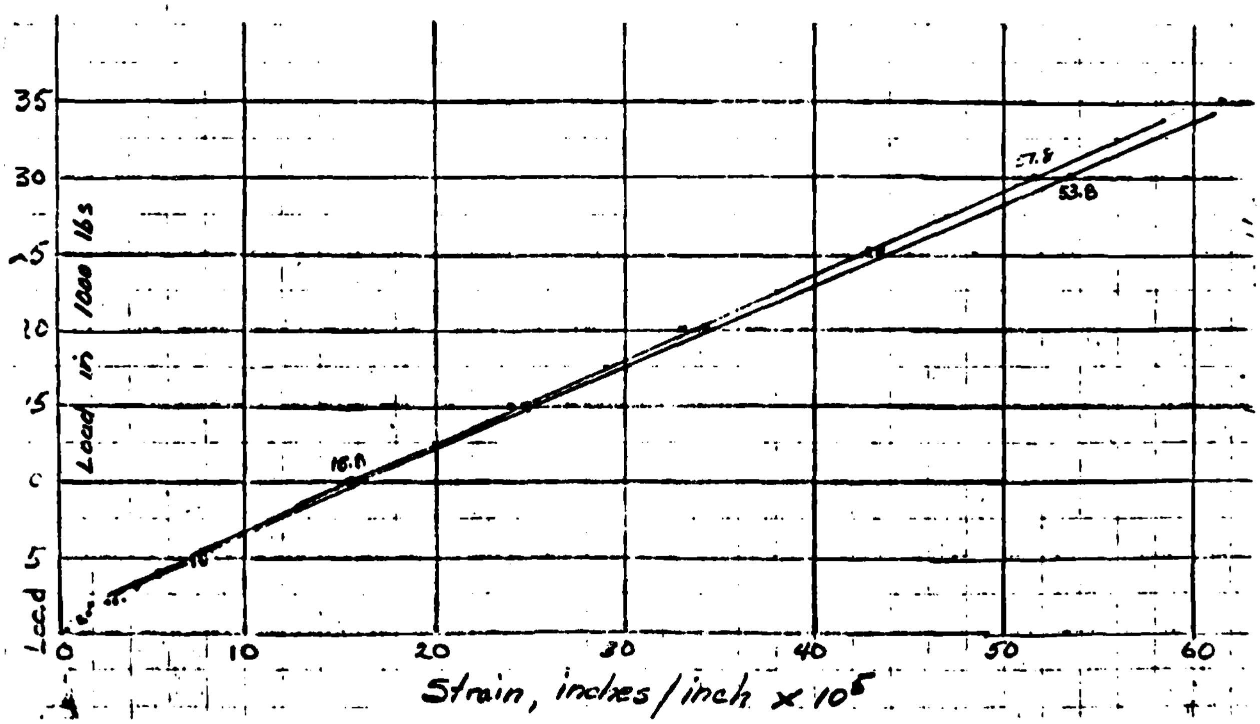
David R. Mitchell

David R. Mitchell

Geotechnical

Tested 10-19-79

	I	II
Zero	0	0
500	1.4	1.0
1,000	1.9	1.9
2000	3.2	2.9
3000	4.5	4.4
4000	6.0	5.8
5000	8.0	7.9
10,000	15.8	15.8
15,000	24.9	24.0
20,000	34.2	33.2
25,000	43.6	42.8
30,000	53.8	51.8
35,000	64.0	61.5
P	104,250	103,500
f'c	7780#	7730#



HERMAN G. PROTZE
— INC. —
MATERIALS TECHNOLOGIST 36 JACONNET STREET, NEWTON HIGHLANDS, MASS. 02161 (617) 332-8460

December 14, 1979

GEOTECHNICAL ENGINEERS, INC.
WINCHESTER, MASSACHUSETTS

MODULUS OF ELASTICITY OF CONCRETE
HARTFORD TUNNEL PROJECT

REFERENCE NUMBER	79L-1792
DATE RECEIVED	12-11-79
DATE FABRICATED	7-18-79
DATE TESTED	12-13-79 (age 5 months)
IDENTIFICATION	Two nominal 4x8" concrete cylinders as fabricated and delivered by you. Specimens were identified as I and II. Specimens tested per your instructions.
METHOD OF TEST	Secant modulus of elasticity to 35% ultimate load using two 4½" electrical strain gages and balanced Wheatstone bridge, in conformity with standard methods.

RESULTS

Applied Load	Strain, inches/inch x 10^5 I	Strain, inches/inch x 10^5 II
Zero, lb.	0.0	0.0
5,000	6.4	7.0
10,000	15.0	14.8
15,000	23.6	23.7
20,000	32.1	33.0
25,000	42.1	43.6
30,000	52.2	53.6
35,000	62.1	65.5
Modulus of Elasticity	4,100,000ψ	4,000,000ψ
Compressive Strength	8200ψ	8480ψ

Respectfully submitted,

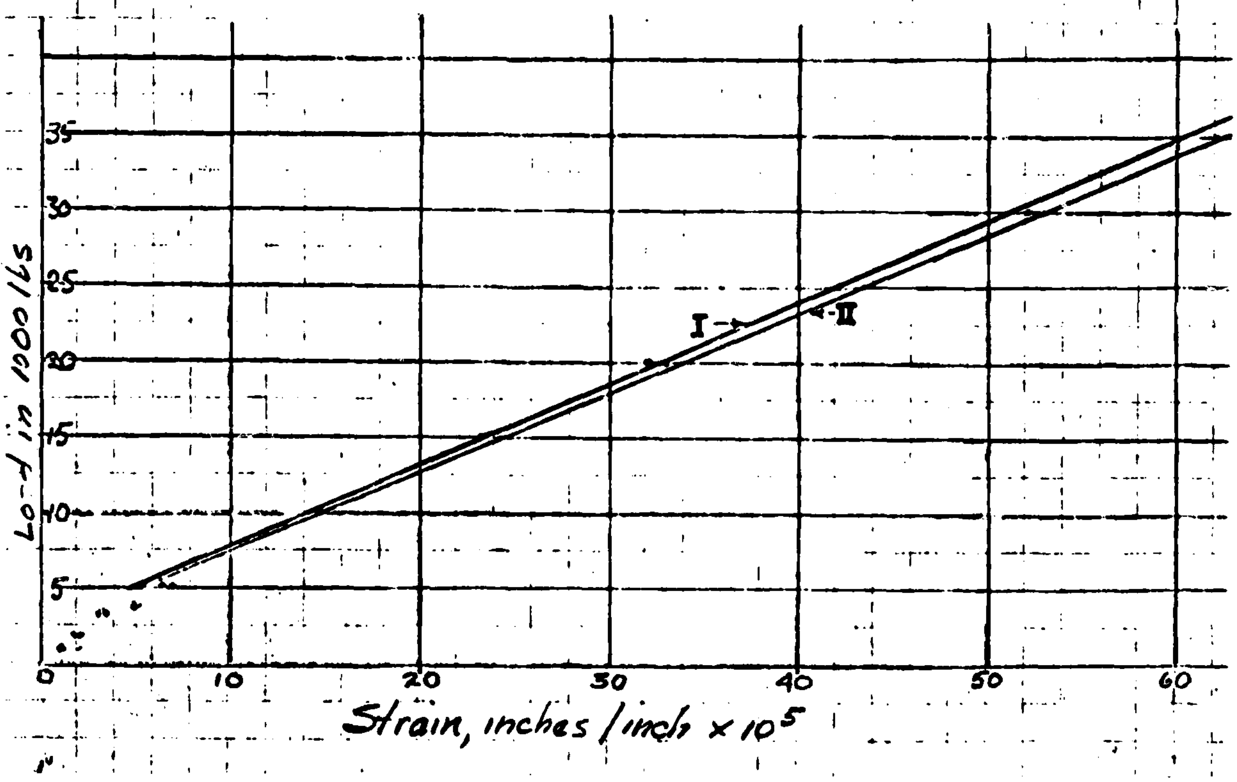
David R Mitchell

David R. Mitchell

Tested
12-13-79
5 months

Geotechnical

	I	II
Zero	0	0
1000	1.1	1.0
2000	2.2	2.0
3000	3.7	3.2
4000	5.0	5.2
5000	6.4	7.0
10,000	15.0	14.8
15,000	23.6	23.7
20,000	32.1	33.0
25,000	42.1	43.6
30,000	52.2	53.6
35,000	62.1	65.5
P	109,750	113,500
f'c	8200#	8480#



HERMAN G. PROTZE
— INC —
MATERIALS TECHNOLOGIST 36 JACONNET STREET, NEWTON HIGHLANDS, MASS. 02161 (617) 332-8460

April 14, 1980

GEOTECHNICAL ENGINEERS, INC.
WINCHESTER, MASSACHUSETTS

MODULUS OF ELASTICITY OF CONCRETE
HARTFORD TUNNEL PROJECT

REFERENCE NUMBER	80L-249
DATE RECEIVED	3-12-80
DATE FABRICATED	1-4-80
DATE TESTED	4-11-80 (age 14 weeks)
IDENTIFICATION	Two nominal 4x8" cylinders as fabricated and delivered (minus their molds) by you. Specimens were identified as III and IV.
METHOD OF TEST	Specimens stored in laboratory air until tested. Secant modulus of elasticity determined at 35% of ultimate load using two 2½" electrical strain gages and balanced Wheatstone bridge, in conformity with standard methods.

RESULTS

Applied Load	Strain, inches/inch x 10^5 III	IV
Zero, lb.	0.0	0.0
5000	9.0	7.0
10000	19.8	16.8
15000	31.4	25.2
20000	41.6	34.8
25000	52.4	44.4
30000	64.8	54.6
35000	76.2	65.0
Modulus of Elasticity	3,430,000ψ	3,970,000ψ
Compressive Strength	8210ψ	7470ψ

Respectfully submitted,

David R. Mitchell

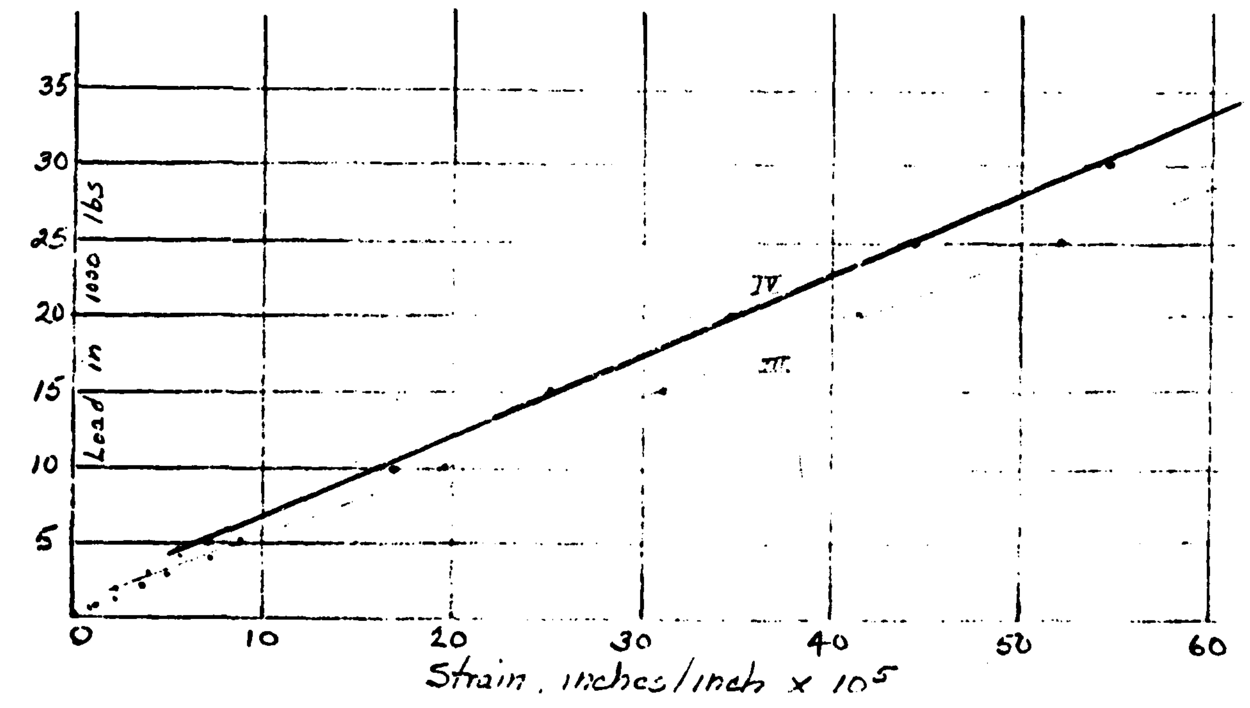
David R. Mitchell

3 Geotechnical

Geotechnical

Tested: 4-11-[illegible]
Ref No: 8[illegible]L [illegible]49

	III	IV
Zero	0.0	0.0
500	1.2	0.8
1000	2.4	1.1
2000	3.8	[illegible]
3000	5.2	4.0
4000	7.2	5.8
5000	9.0	7.0
10000	19.8	16.8
15000	31.4	25.2
[illegible]	41.6	34.8
25000	52.4	44.4
30000	61.8	[illegible]4.6
35000	76.7	[illegible]
[illegible]	110,000	[illegible]
f'c	8200	7470



HERMAN G. PROTZE
– INC. –
MATERIALS TECHNOLOGIST 36 JACONNET STREET, NEWTON HIGHLANDS, MASS. 02161 (617) 332-8460

April 14, 1980

GEOTECHNICAL ENGINEERS, INC.
WINCHESTER, MASSACHUSETTS

MODULUS OF ELASTICITY OF CONCRETE
HARTFORD TUNNEL PROJECT

REFERENCE NUMBER	80L-132
DATE RECEIVED	2-12-80
DATE FABRICATED	1-4-80
DATE TESTED	3-7-80 (age 9 weeks)
IDENTIFICATION	Two nominal 4x8" cylinders as fabricated and delivered (minus their molds) by you. Specimens were identified as ~~I~~ III and ~~II.~~ IV
METHOD OF TEST	Specimens stored in laboratory air until tested. Secant modulus of elasticity determined at 35% of ultimate load using two 2½" electrical strain gages and balanced Wheatstone bridge, in conformity with standard methods.

RESULTS

Applied Load	Strain, inches/inch x 10^5 III	IV
Zero, lb.	0.0	0.0
5000	7.3	11.8
10000	17.2	21.1
15000	26.1	31.0
20000	36.0	39.4
25000	45.9	52.0
30000	57.3	58.1
35000	67.8	64.8

	III	IV
Modulus of Elasticity	3,840,000ψ	3,940,000ψ
Compressive Strength	8550ψ	8480ψ

Respectfully submitted,

David R. Mitchell

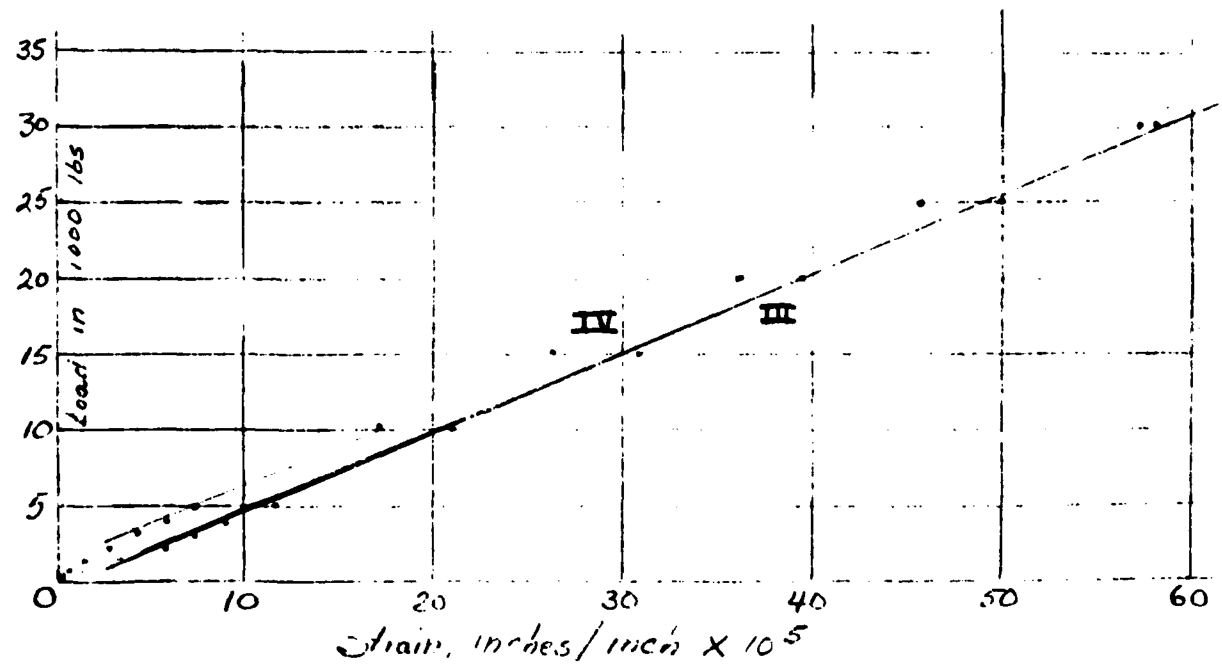
David R. Mitchell

3 Geotechnical

Geotechnical

Tested: [illegible]
Ref. No: [illegible]-132

Load	III	IV
Zero	0.0	0.0
500	0.6	1.5
1000	1.1	3.4
2000	[illegible]	5.8
3000	[illegible]	[illegible]
4000	[illegible]	[illegible]
5000	[illegible]	[illegible]
10 000	[illegible]	[illegible]
15 000	[illegible]	[illegible]
20 000	[illegible]	[illegible]
25000	[illegible]	[illegible]
30 000	[illegible]	[illegible]
35 000	[illegible]	[illegible]
P	[illegible]	[illegible]
f'c	[illegible]	[illegible]



HERMAN G. PROTZE
— INC. —
MATERIALS TECHNOLOGIST 36 JACONNET STREET, NEWTON HIGHLANDS, MASS. 02161 (617) 332-8460

November 12, 1980

GEOTECHNICAL ENGINEERS, INC.
WINCHESTER, MASSACHUSETTS

MODULUS OF ELASTICITY OF CONCRETE
HARTFORD TUNNEL PROJECT

REFERENCE NUMBER	80L-614
DATE RECEIVED	6-19-80
DATE FABRICATED	1-4-80
DATE TESTED	7-25-80 (age 6 months, 3 weeks)
IDENTIFICATION	Two nominal 4x8" concrete cylinders as fabricated and delivered by you. Specimens were identified as III and IV.
METHOD OF TEST	Specimens stored in laboratory air after receipt until tested. Secant modulus of elasticity to 35% of ultimate load using two 2½" electrical strain gages and balanced Wheatstone bridge, in conformity with standard methods. Both specimens were tested per your instructions.

RESULTS

Applied Load	Strain, inches/inch x 10^5 III	IV
Zero, lb.	0.0	0.0
5000	8.2	5.8
10000	15.6	14.4
15000	23.8	25.6
20000	32.2	34.0
25000	40.0	43.2
30000	48.6	53.0
35000	58.2	62.4
Modulus of Elasticity	4,430,000ψ	3,980,000ψ
Compressive Strength	8700ψ	8030ψ

Respectfully submitted,

David R. Mitchell

David R. Mitchell

	III	IV
Zero	0	0
500	0.2	0
1000	2.7	1.2
2000	4.3	2.2
3000	5.0	3.1
4000	6.8	4.0
5000	8.2	5.8
10000	15.6	14.4
15000	23.8	25.6
20000	32.2	34.0
25000	40.0	43.2
30000	48.6	53.0
35000	58.2	62.4
P	116,000	107000
f'c	8700	8030

Rec'd 6-19-80

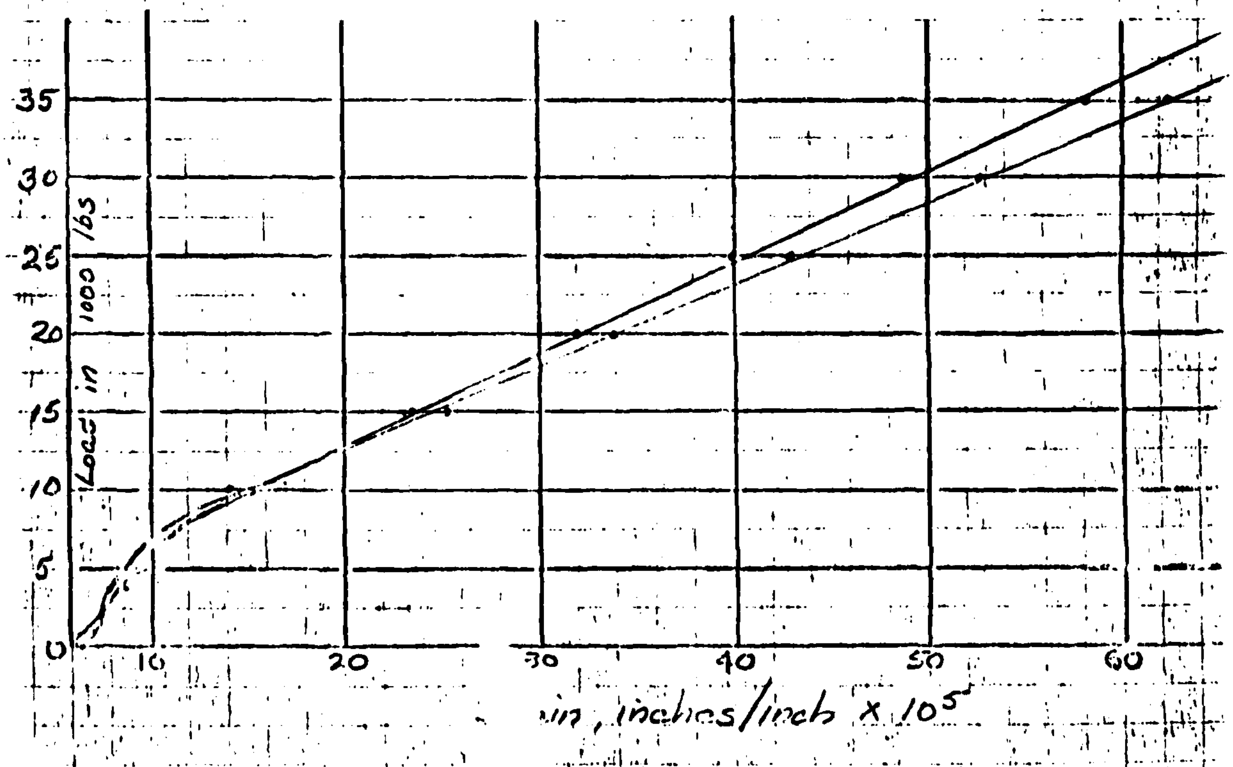
Tested 7-25-80

Made 1-4-80

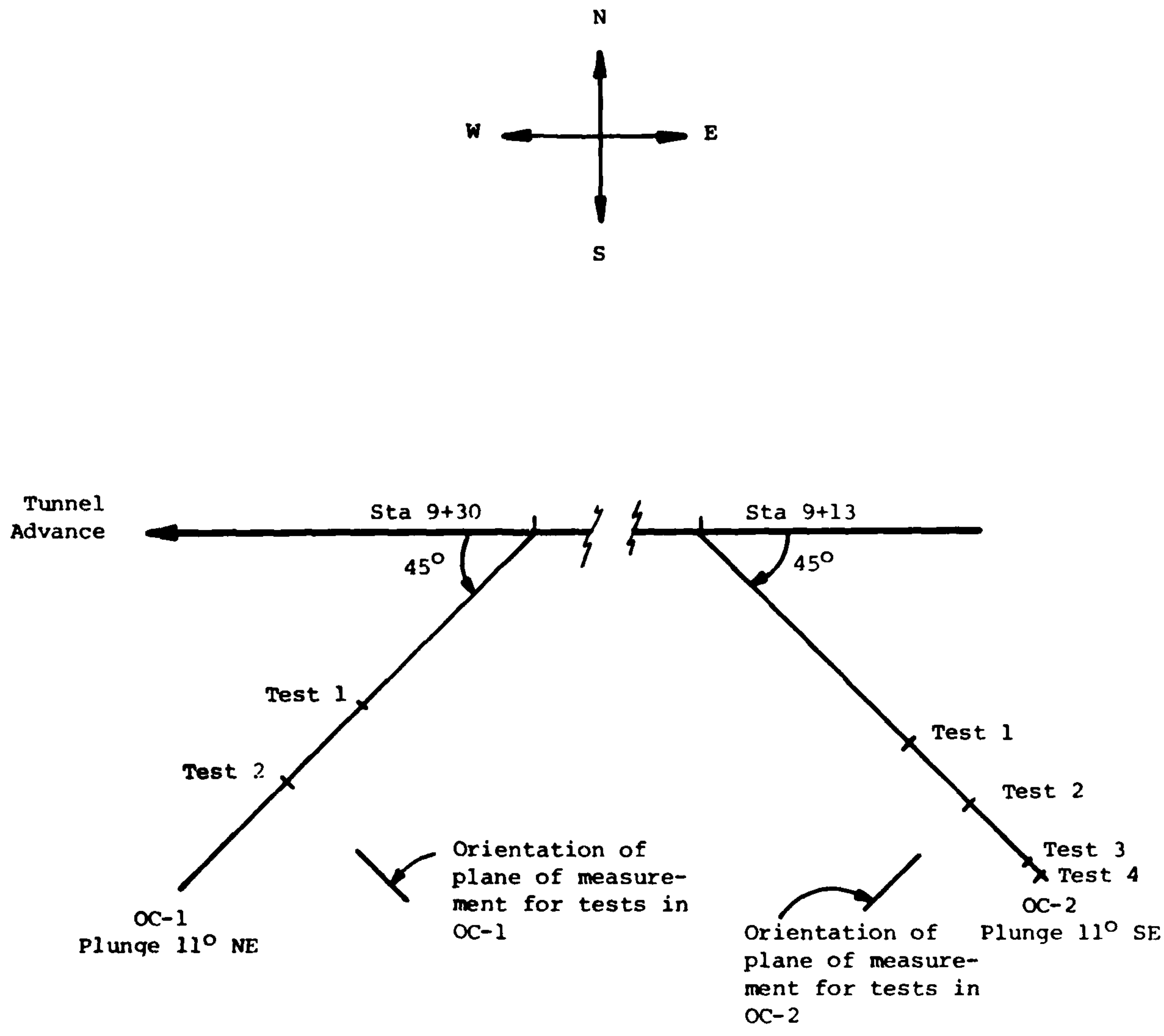
Age 6 Mo. 3 wks

Ref. No. 80L-614

III IV



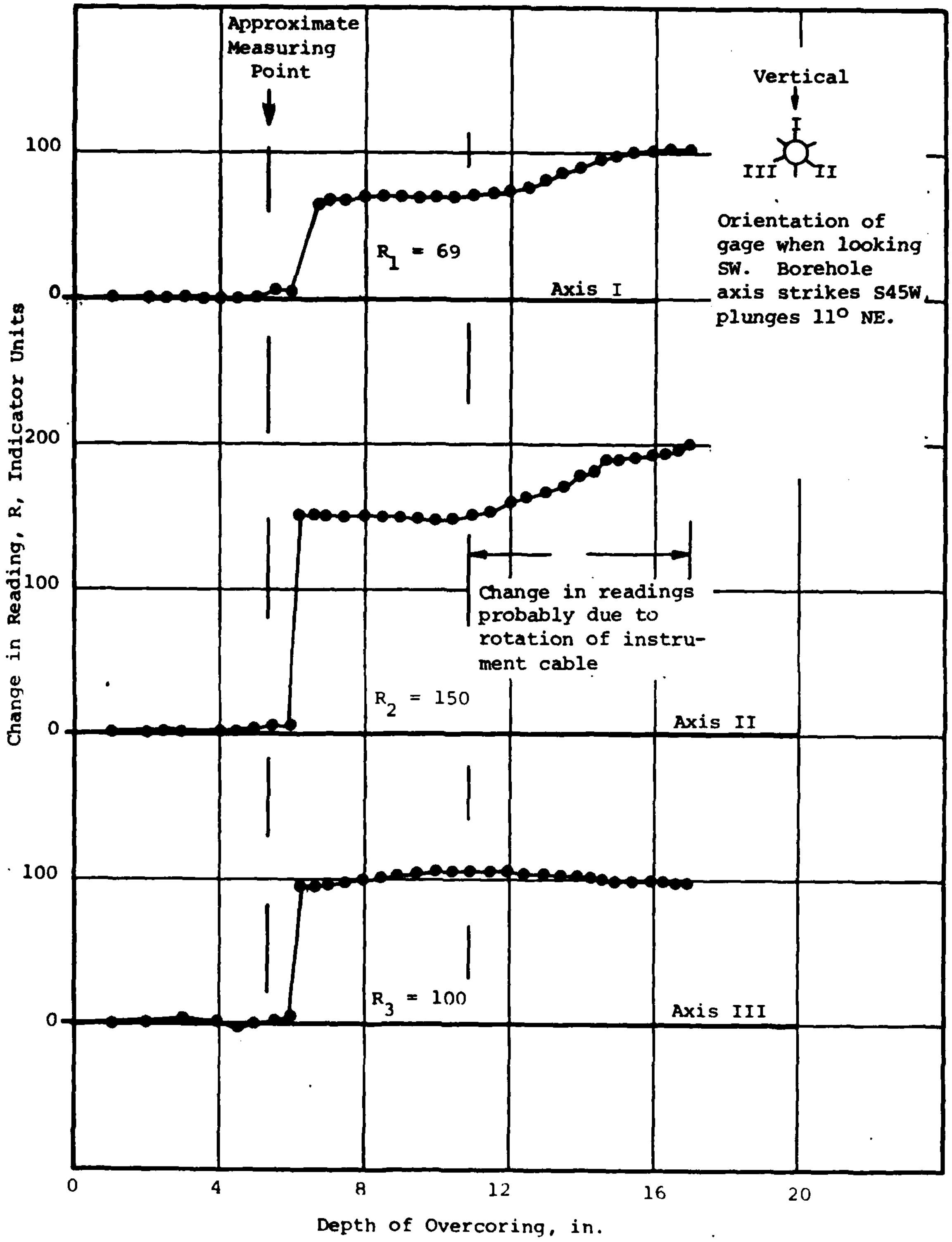
APPENDIX N



Sandstone bed dips 16° E

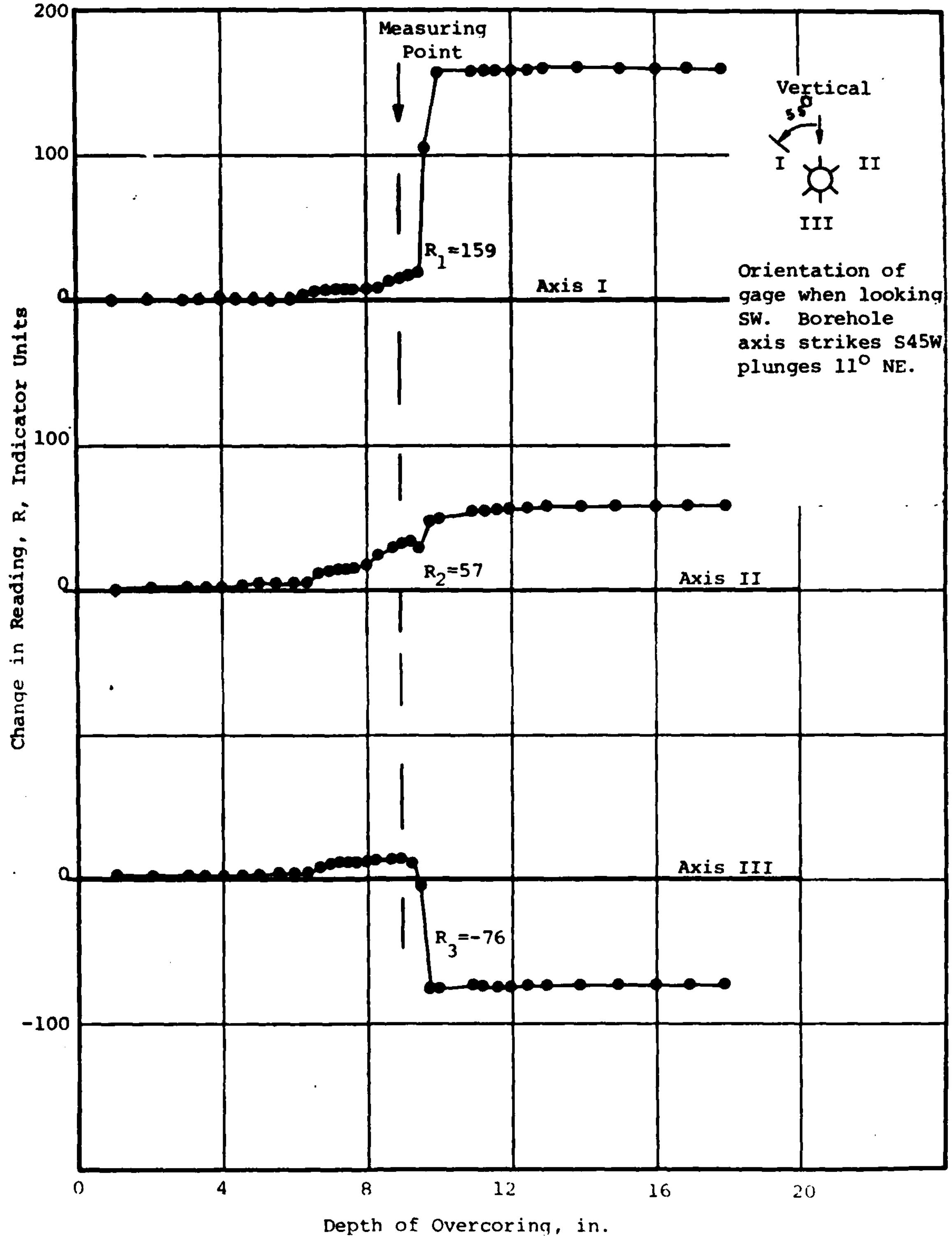
1 in. = 10 ft

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	PLAN OF OVERCORE BORINGS
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 77382	August 1980 Fig. H-1



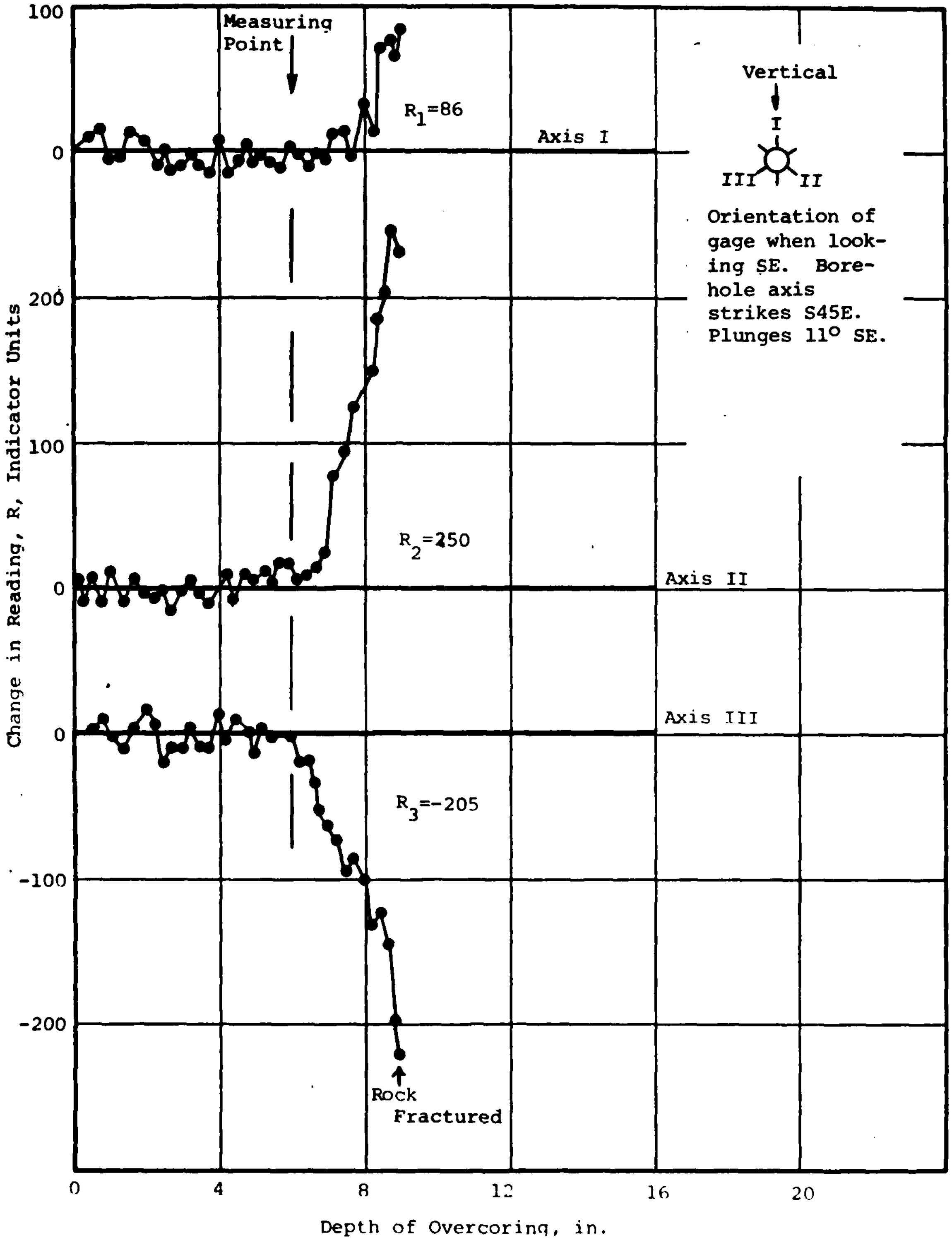
Measuring point is 14 ft 4 in. from tunnel wall along the borehole.

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	DATA FROM STRAIN MEASUREMENTS Hole No. OC-1, Test 1
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-2



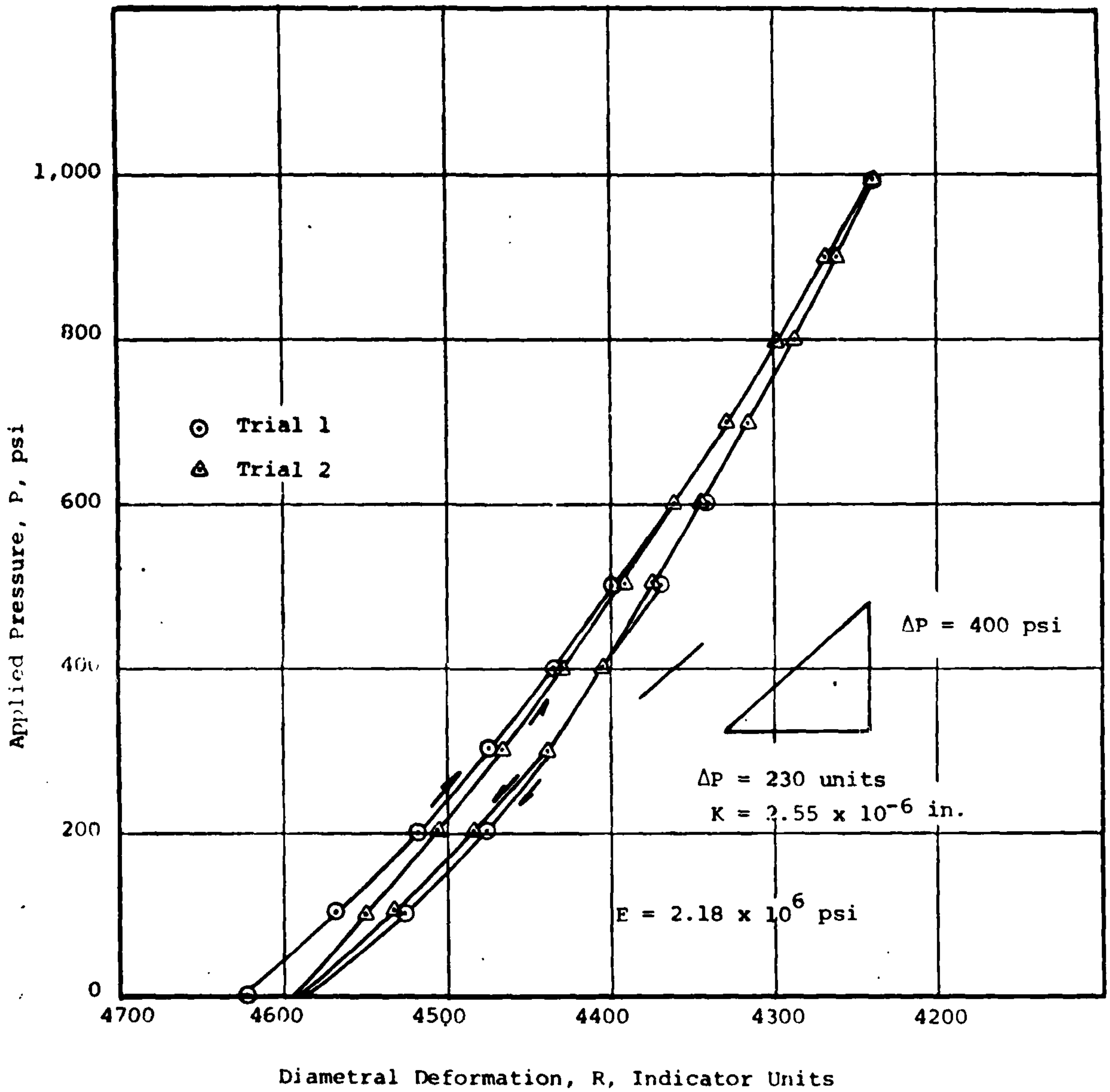
Measuring point is 21 ft from tunnel wall along the borehole.

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	DATA FROM STRAIN MEASUREMENTS Hole No. OC-1, Test 2
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-3



Measuring point is 23 ft 1 in. from tunnel wall along the borehole.

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	DATA FROM STRAIN MEASUREMENTS Hole No. OC-2, Test 2
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-4



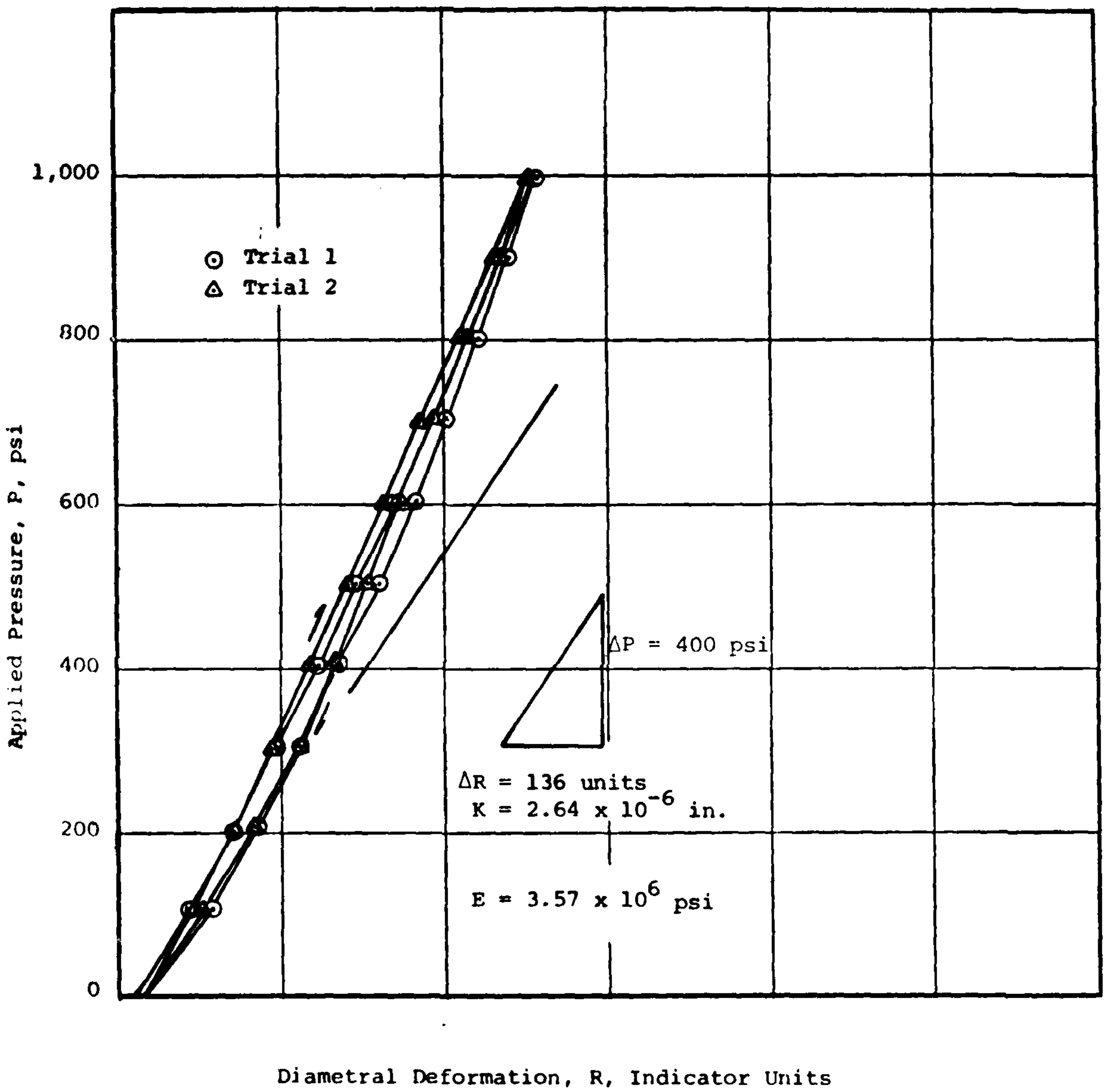
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 13¼ inches

Depth of Measuring Point = 14ft, 4 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-1 Test No. 1 Axis I
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-5



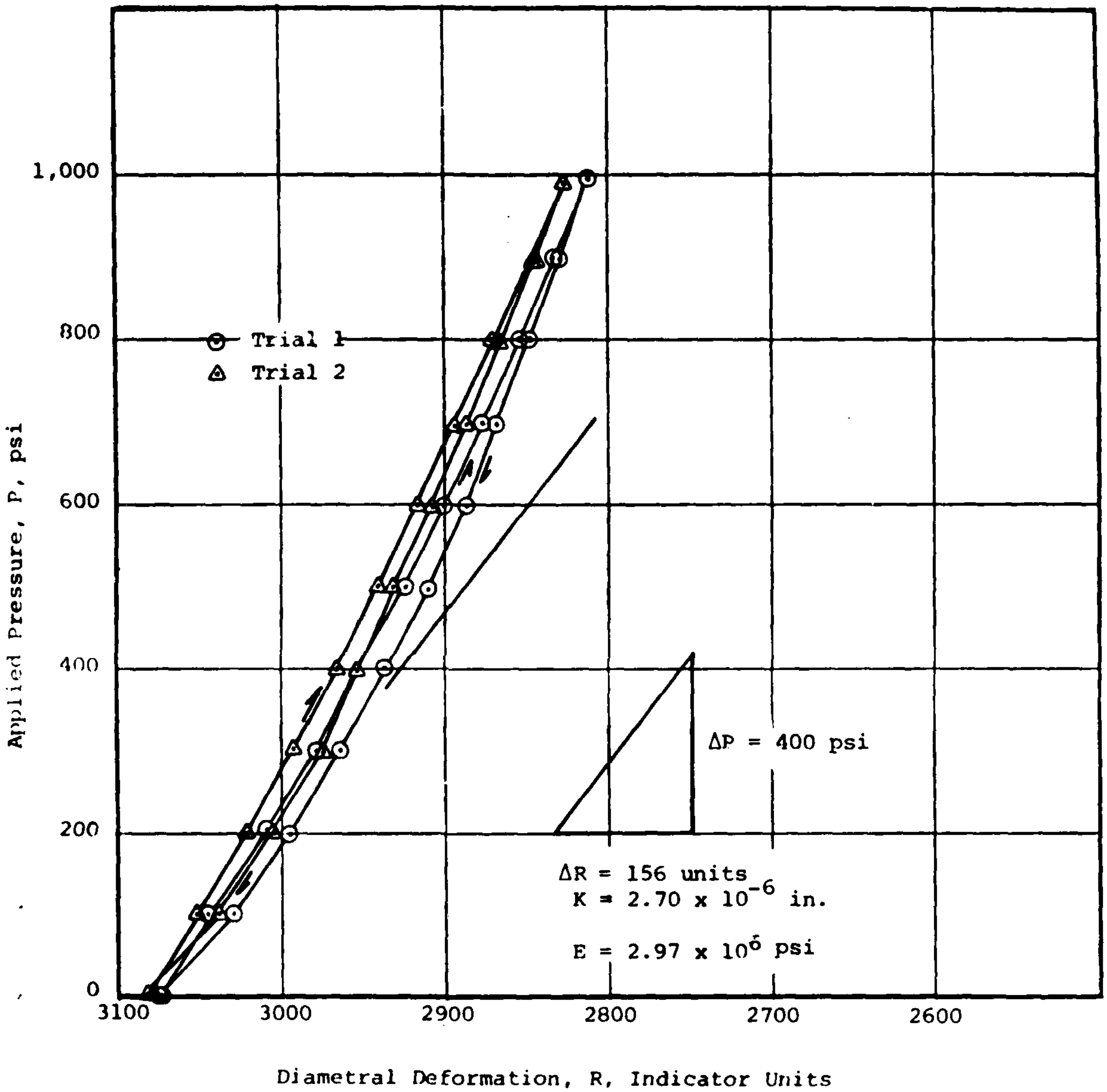
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 13¼ inches

Depth of Measuring Point = 14 ft, 4 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-1 Test No. 1 Axis II
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-6



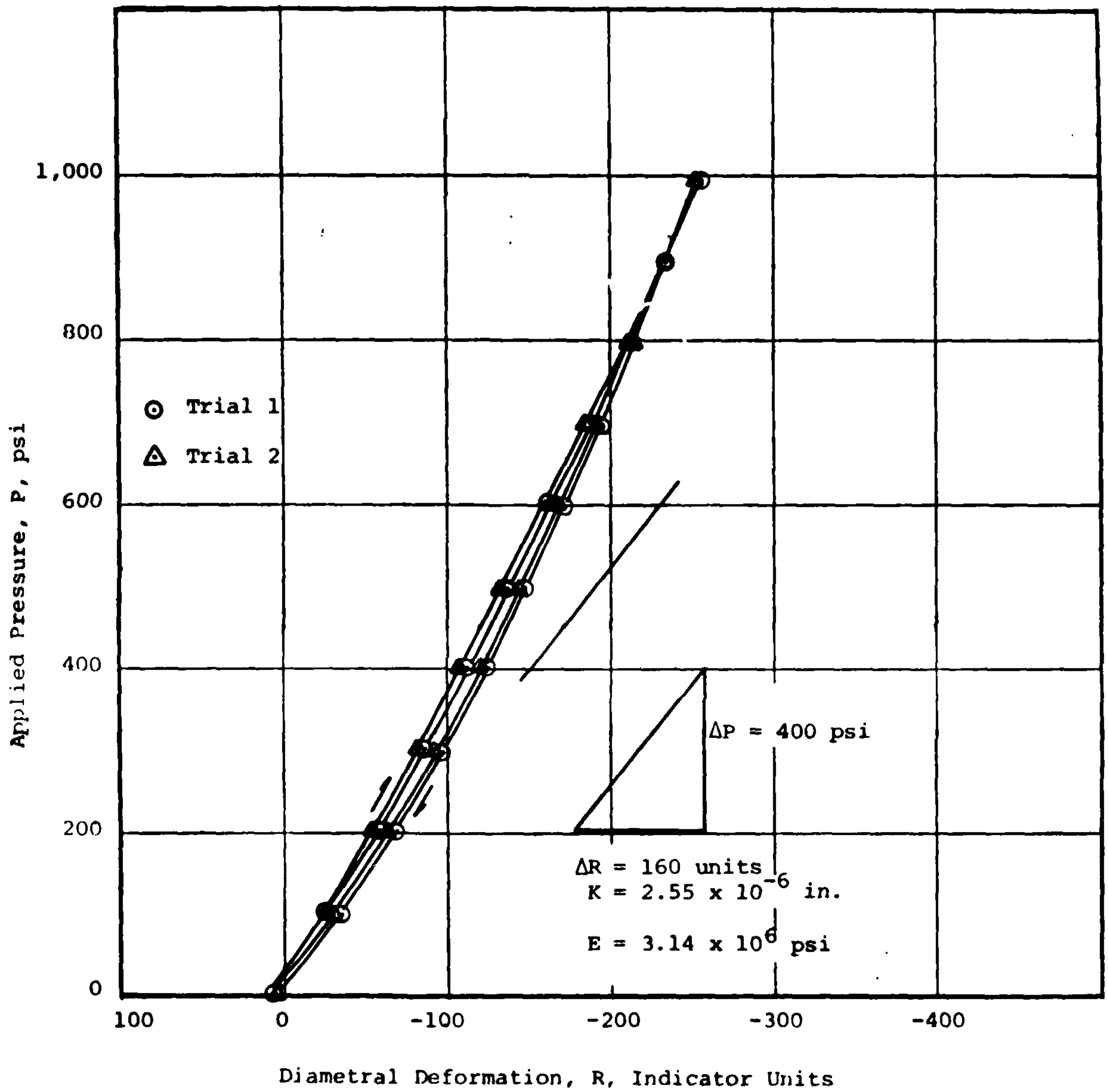
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 13¼ inches

Depth of Measuring Point = 14 ft, 4 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-1 Test No. 1 Axis III
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-7



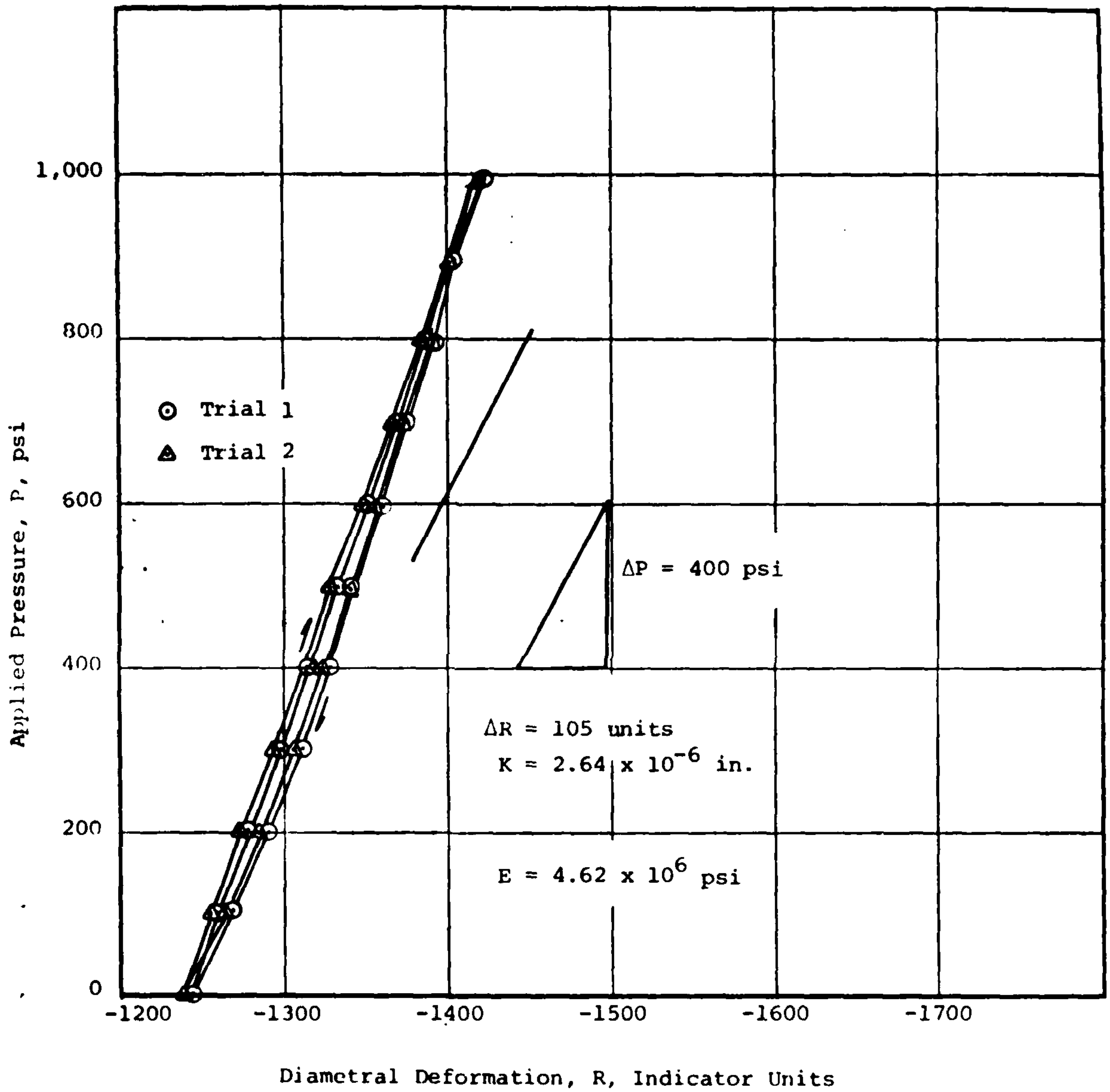
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 13 inches

Depth of Measuring Point = 21 ft, 0 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-1 Test No. 2 Axis I
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-8



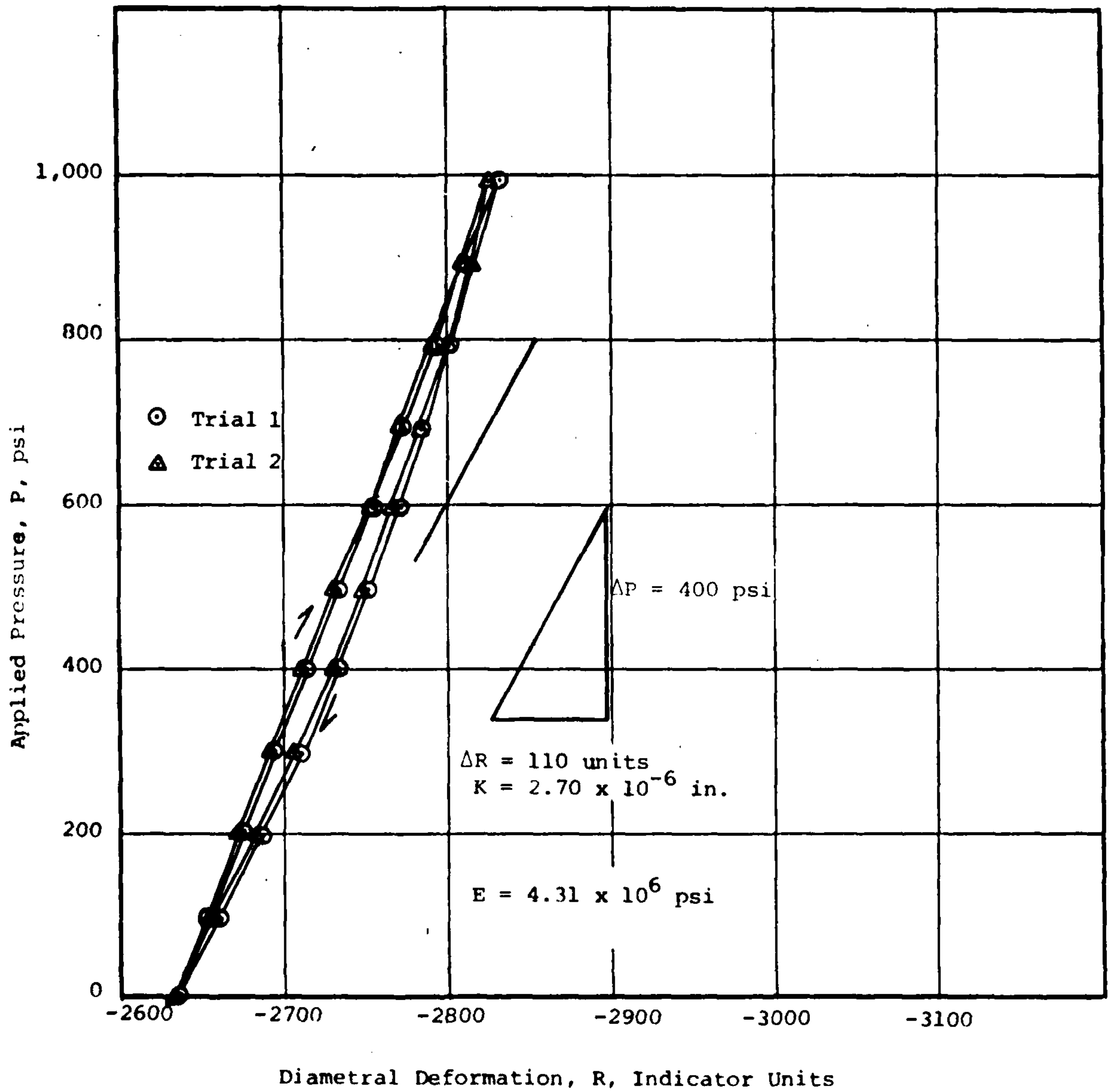
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 13 inches

Depth of Measuring Point = 21 ft, 0 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-1 Test No. 2 Axis II
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-9



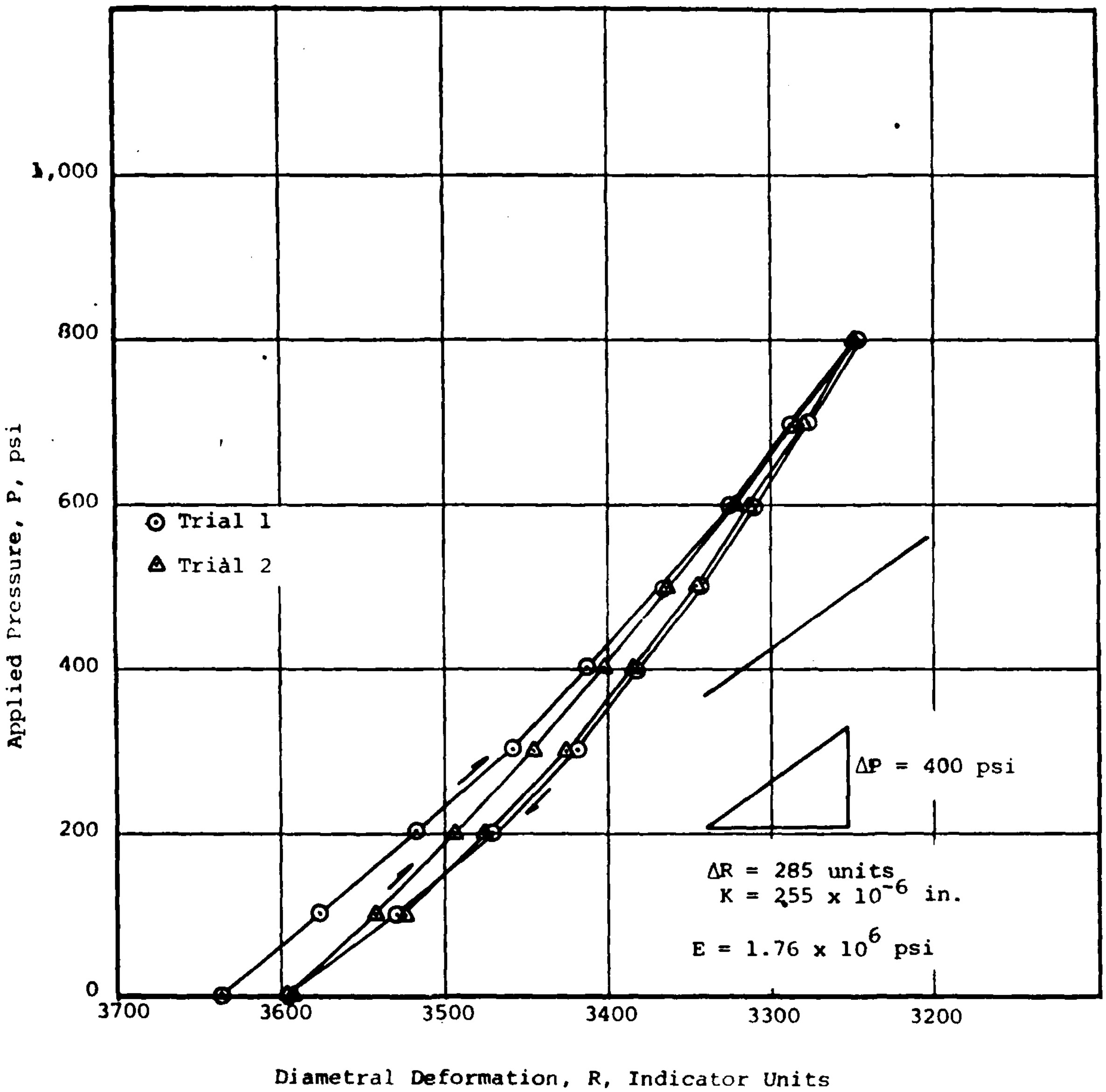
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 13 inches

Depth of Measuring Point = 21 ft, 0 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-1 Test No. 2 Axis III
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-10



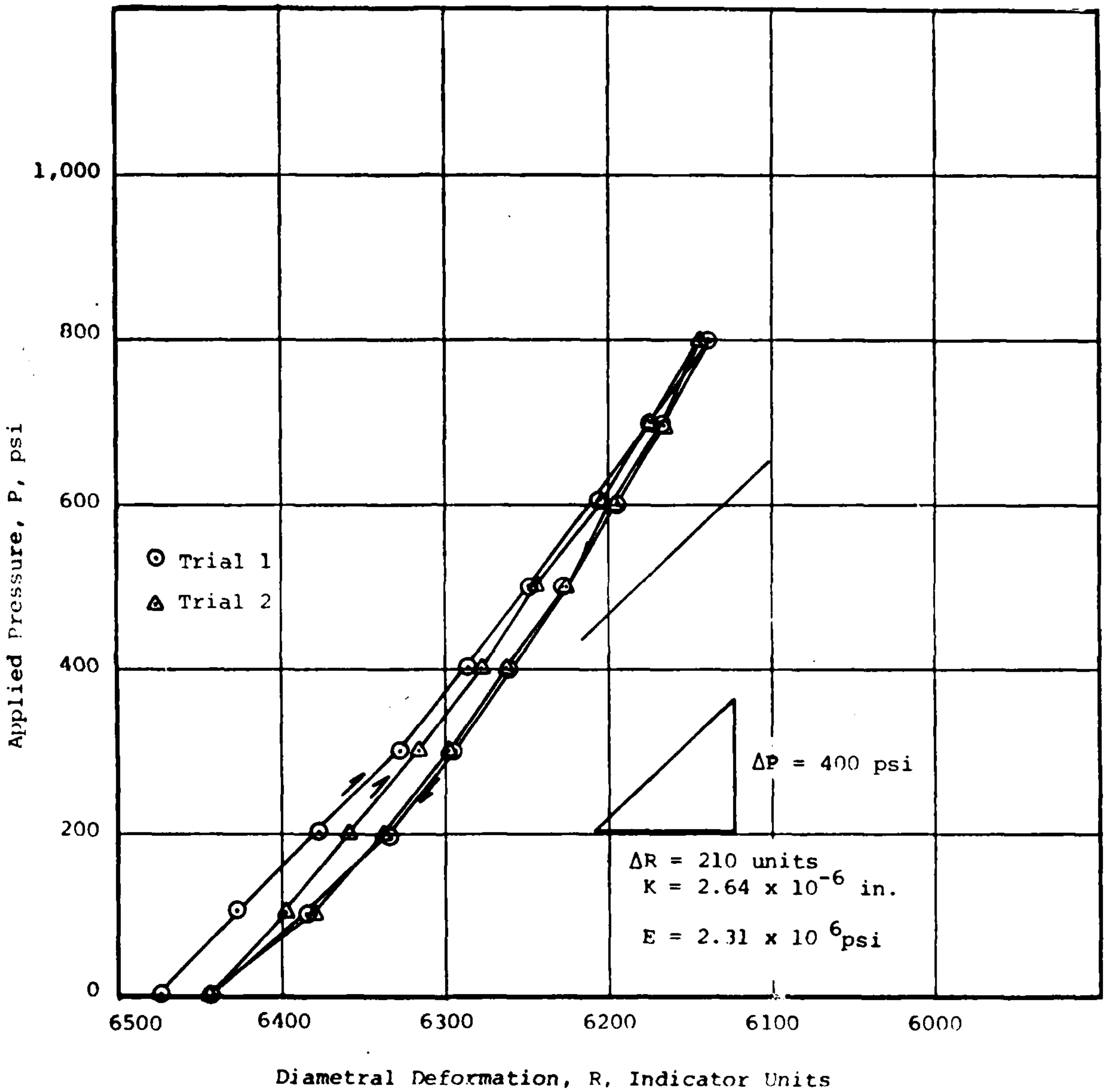
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 11 inches

Depth of Measuring Point = 23 ft, 11 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-2 Test No. 2 Axis I
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-11



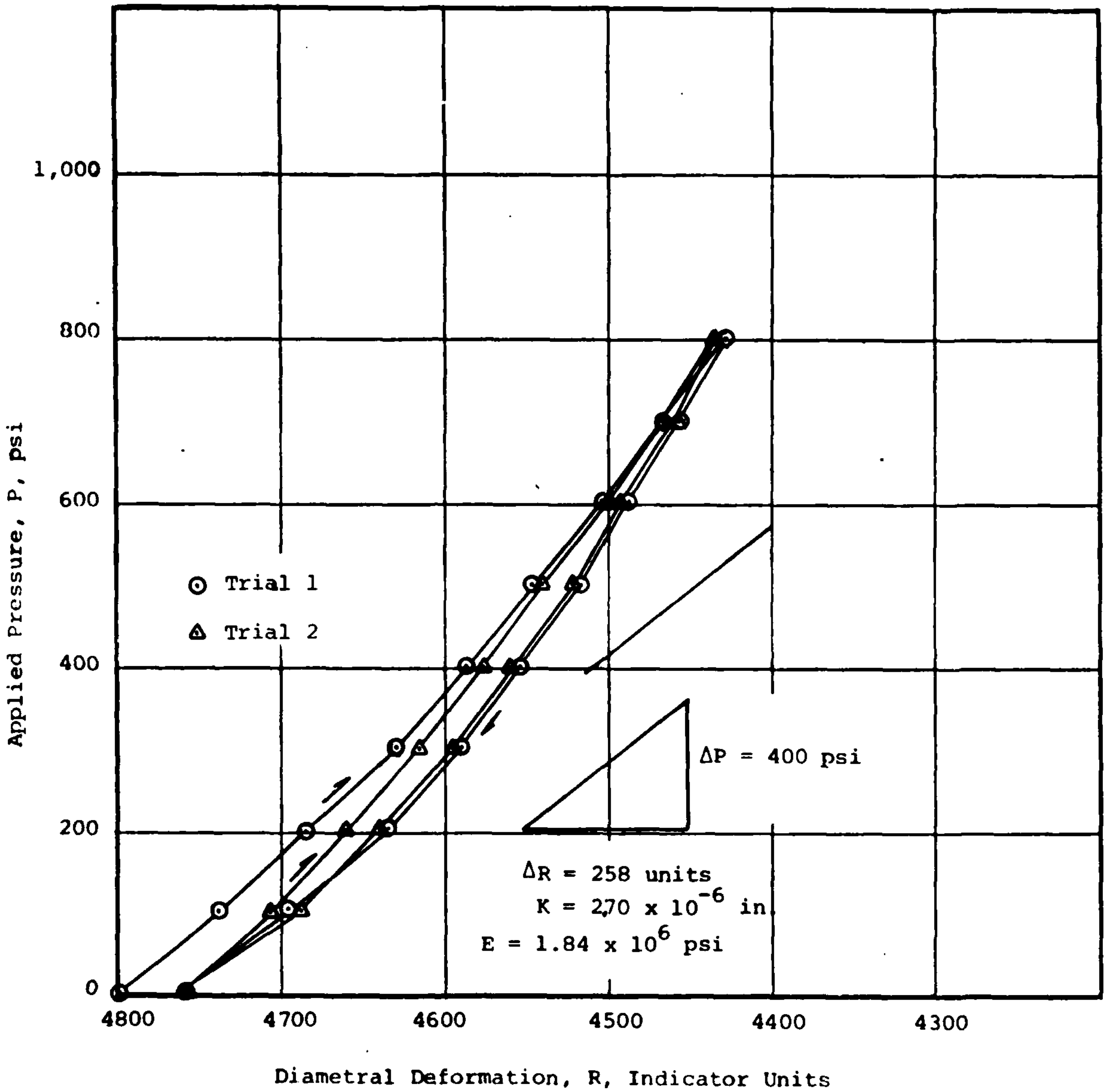
Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 11 inches

Depth of Measuring Point = 23 ft, 11 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-2 Test No. 2 Axis II
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. M-12



Sample O.D = 5.97 inches

Ex Hole I.D. = 1.50 inches

Length of Core = 11 inches

Depth of Measuring Point = 23 ft, 11 inches

Roger J. Au & Son, Inc. Mansfield, Ohio	Park River Auxiliary Tunnel Hartford, CT	BIAXIAL TEST RESULTS Hole No. OC-2 Test No. 2 Axis III
Geotechnical Engineers Inc. Winchester, Massachusetts	Project 77382	August 1980 Fig. H-13

PHOTOGRAPHIC GEOLOGIC DOCUMENTATION

EXAMPLES

APPENDIX - I

PHOTO I-1 Gray shale, fractured, with fines present on the brecciated surfaces. Photo obtained at Station 96+01, near the fault zone encountered at Station 96+65.

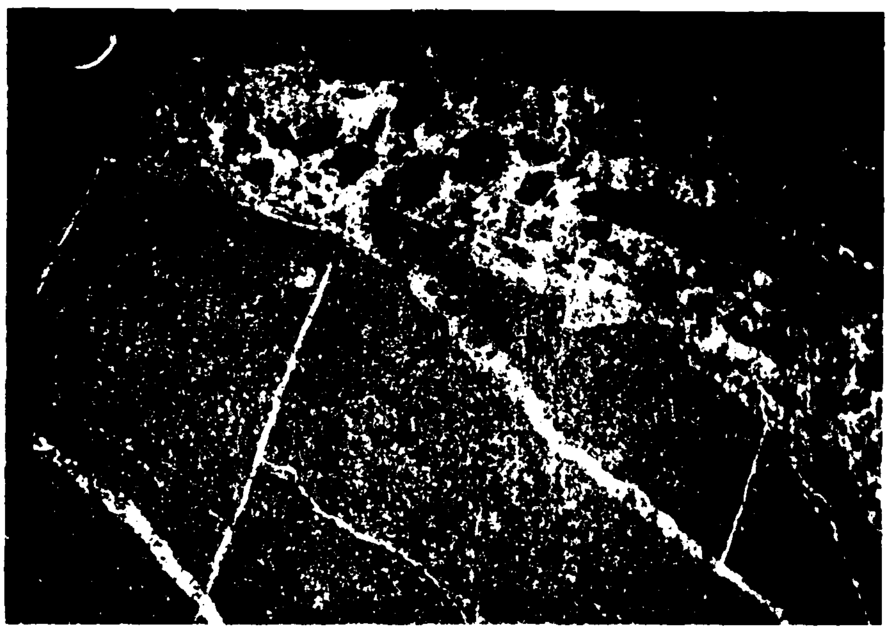


PHOTO I-2 Calcite-healed breccia in contact with gray sandstone. Two parallel calcite-healed joint sets appear within the gray sandstone. Photo obtained at Station 91+32.

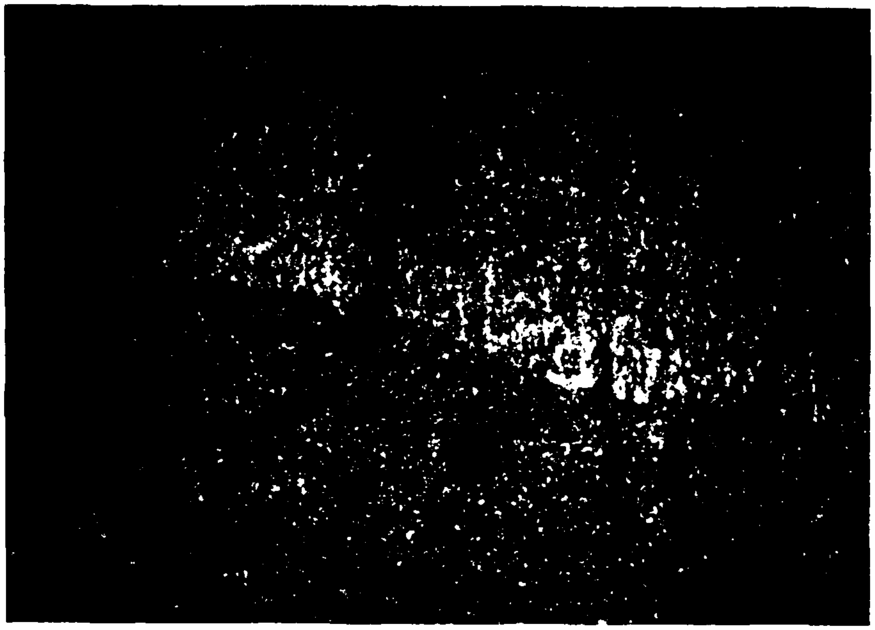


PHOTO I-3 Interbedded sandstone in contact with red shale. Cross-bedded jointing is exhibited within the interbedded sandstone. Photo obtained at Station 38+72.

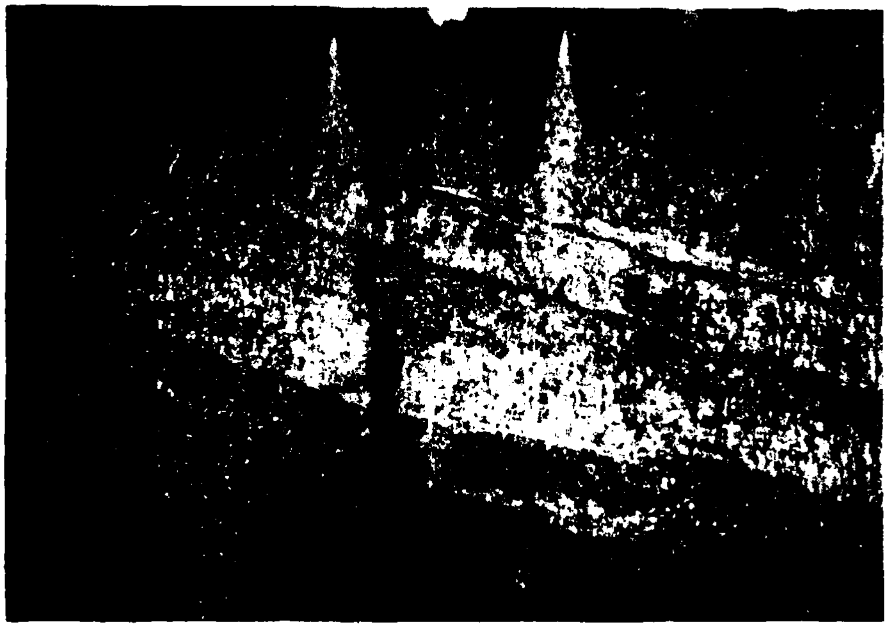


PHOTO I-4 Interbedded sandstone/gray shale in contact with gray shale. The third and fifth water nozzles are spraying water and there is a streak on the glass between camera and exposed bedrock to the left of center on the photo. Photo obtained at Station 65+66.

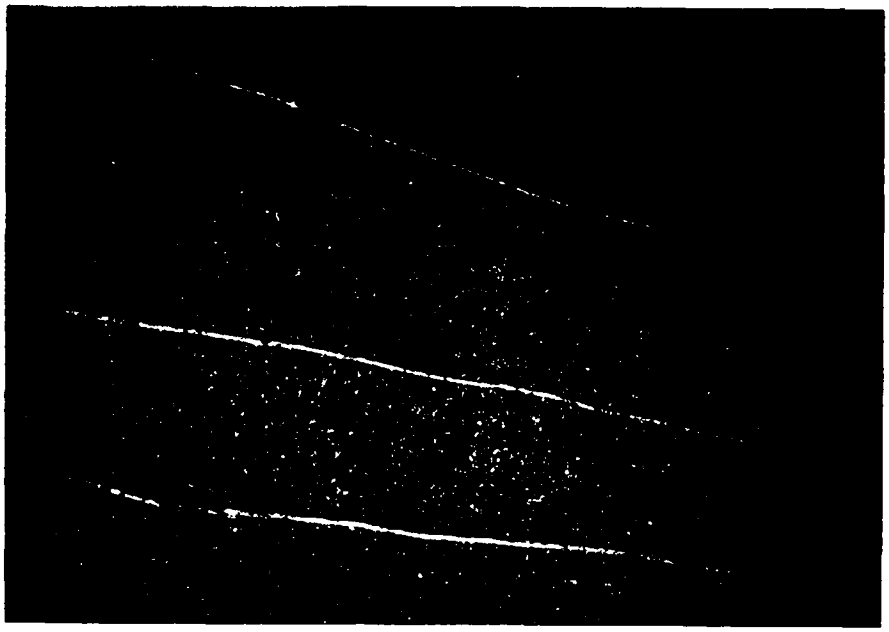


PHOTO I-5 Maroon shale with calcite stringers parallel to bedding. Photo obtained at Station 62+01.

PHOTO I-6 Red sandstone or red shale. Dark spots are accumulated dirt on the glass separating camera and tunnel wall. Photo obtained at Station 53+43.

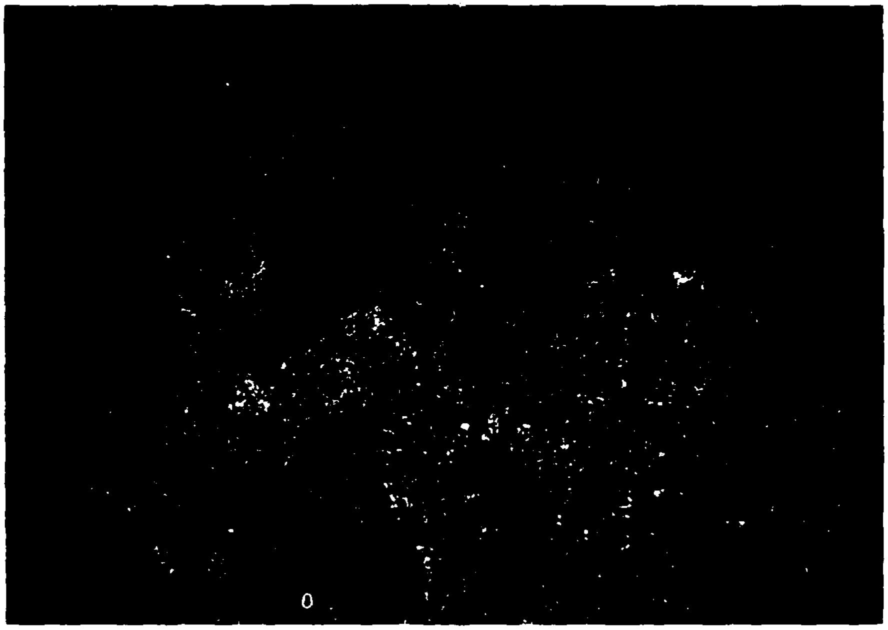


PHOTO I-7 Gray sedimentary rock (shale or sandstone) with darker interbeds, probably gray shale. The glass separating the camera and tunnel wall ap)ears to have condensation on it. Photo obtained at Station 21+66.

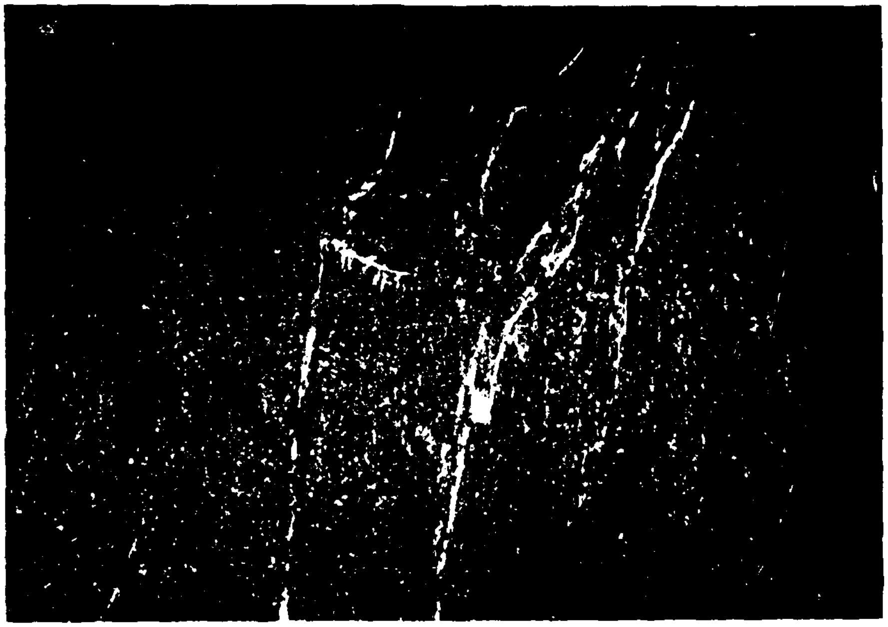


PHOTO I-8 Gray bedrock where the geologic origin and lithological type cannot be visually identified. Calcite-healed stringers appear near vertical and somewhat parallel. Photo obtained at Station 86+76.

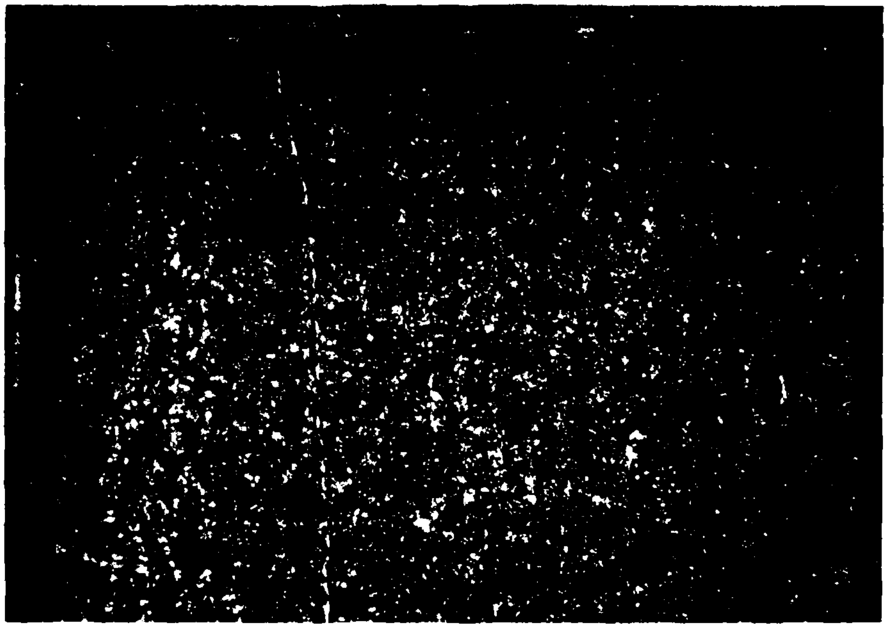


PHOTO I-9 Red shale with closely spaced near-vertical striations due to gage cutter. Photo obtained at Station 23+36.

PHOTO I-10 Muck smeared against the tunnel wall. Several gage cutter striations appear to the left of the smeared material. Photo obtained at Station 45+06.

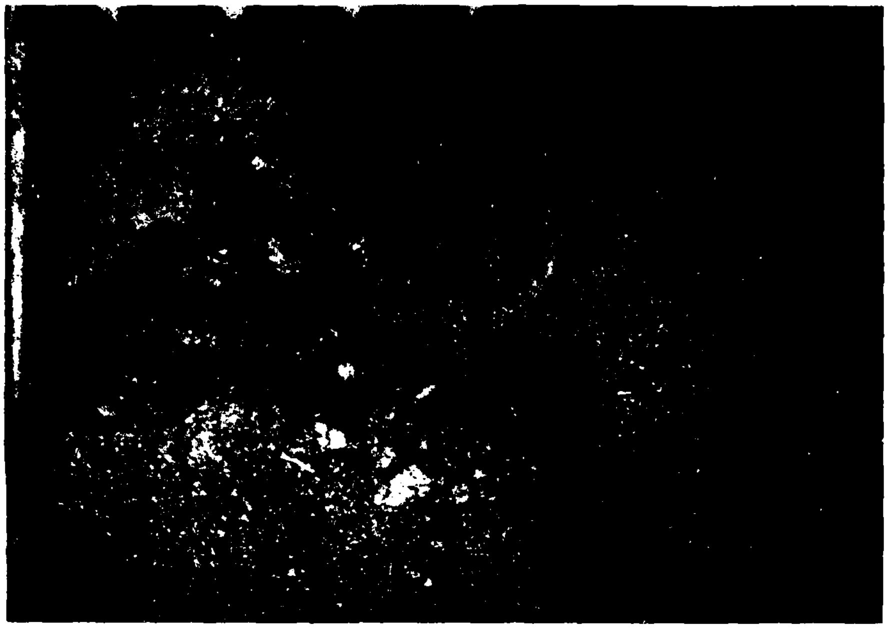


PHOTO I-11 Red shale exhibiting overbreak. The dimensions of the overbreak are approximately 18-in.-long, 9-in.-high, and perhaps 6-in.-deep. Photo obtained at Station 15+62.

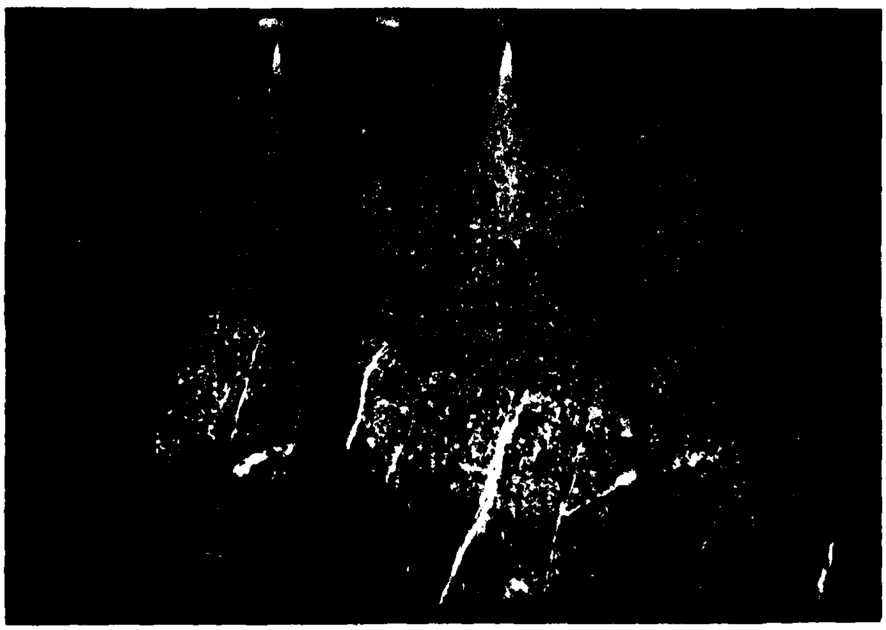


PHOTO I-12 Bed of sandstone in contact with red shale. The sandstone exhibits crossbedded jointing which has been calcite healed. The third and fifth water jets are washing the fines from the exposed bedrock surface revealing greater detail. Photo obtained from Station 72+44.

PHOTO I-13 The exposed bedrock appears interbedded. However, the glass between the camera and bedrock has been blurred by condensation preventing lithologic identification. Photo obtained at Station 19+69.

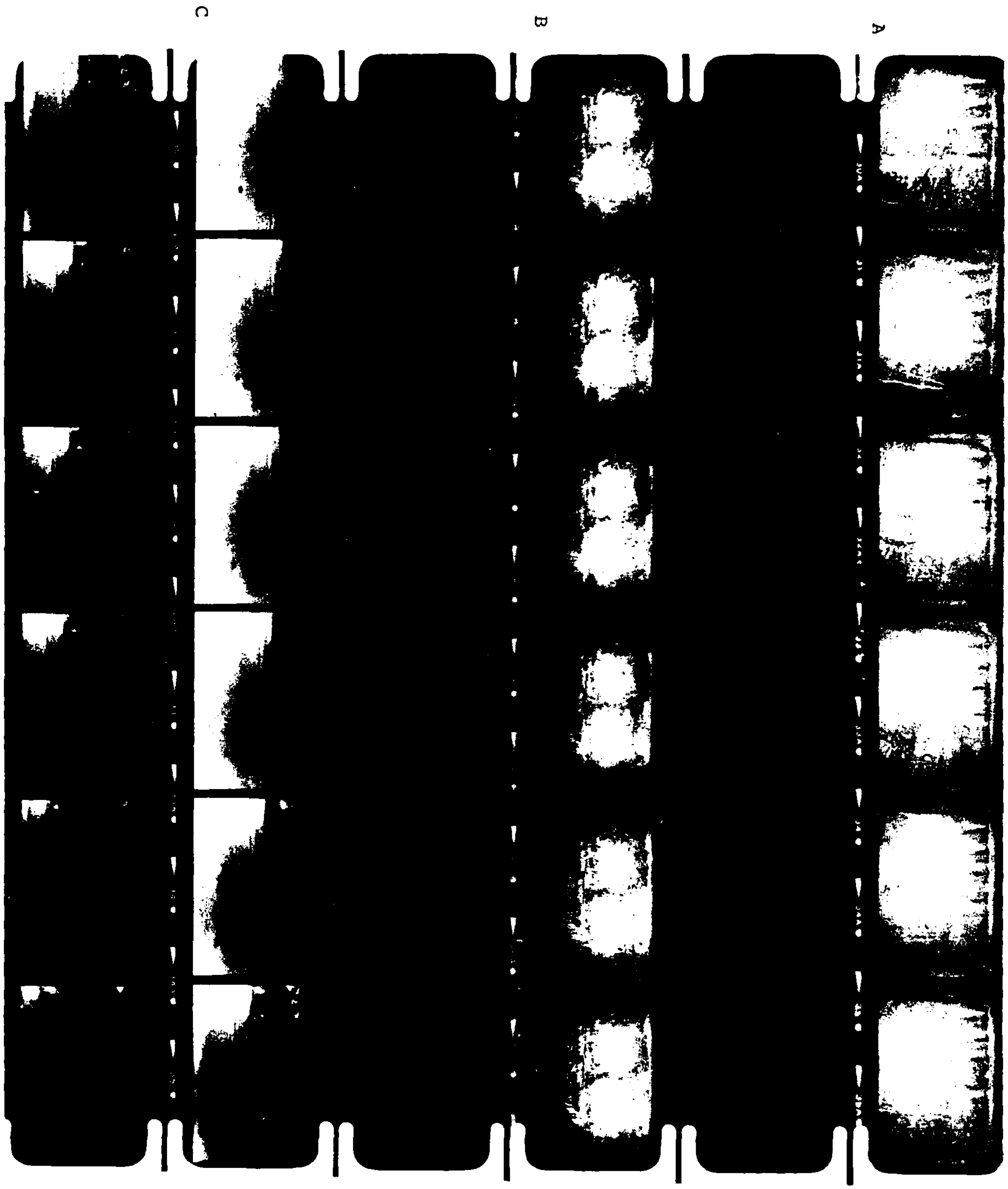


PHOTO I-14

Top Contact Set 14A Exposed bedrock in this series appears to be interbedded sandstone with some crossbedded jointing which has been calcite-healed. Exposures 32 and 33 appear to be duplicates. Exposures 34 and 35 appear to be overlapped based upon the position of the calcite-healed crossbedded joint.

Center Contact Set 14B A series of exposures of the protective door. The two brighter areas on each exposure are due to the reflection of the flashes off of the door.

Lower Contact Sets 14C Appears to be a progressive series of exposures obtained while the clay outside the door is gradually forcing the door open and eventually fracturing the glass between the camera and exposed bedrock. Series taken at fault zone near Station 96 + 60.

APPENDIX B

SUBSIDENCE SURVEY REPORT

14 AUG 1981

Roger J. Au & Son, Inc.
P.O. Box 1488
Mansfield, Ohio 44901
(419)-529-3213

August 13, 1981

U. S. Army Corps of Engineers
75 Laurel Street
Hartford, Connecticut 06106

Attention: Mr. Terry Wilford

RE: Contract No. DACW33-77-C-0099
Construction of Park River
Local Protection Project
Auxiliary Conduit - Part II
Hartford, Connecticut
HO-177

Subject: Settlement Instrumentation
Final Report

Gentlemen:

Enclosed herewith are three (3) copies of our Final Settlement Instrumentation Report, as per Section 13B Subsection 5.4 of the Contract Specifications.

Sincerely,

ROGER J. AU & SON, INC.

John J. Duffy,
Project Manager

JJD/clcs

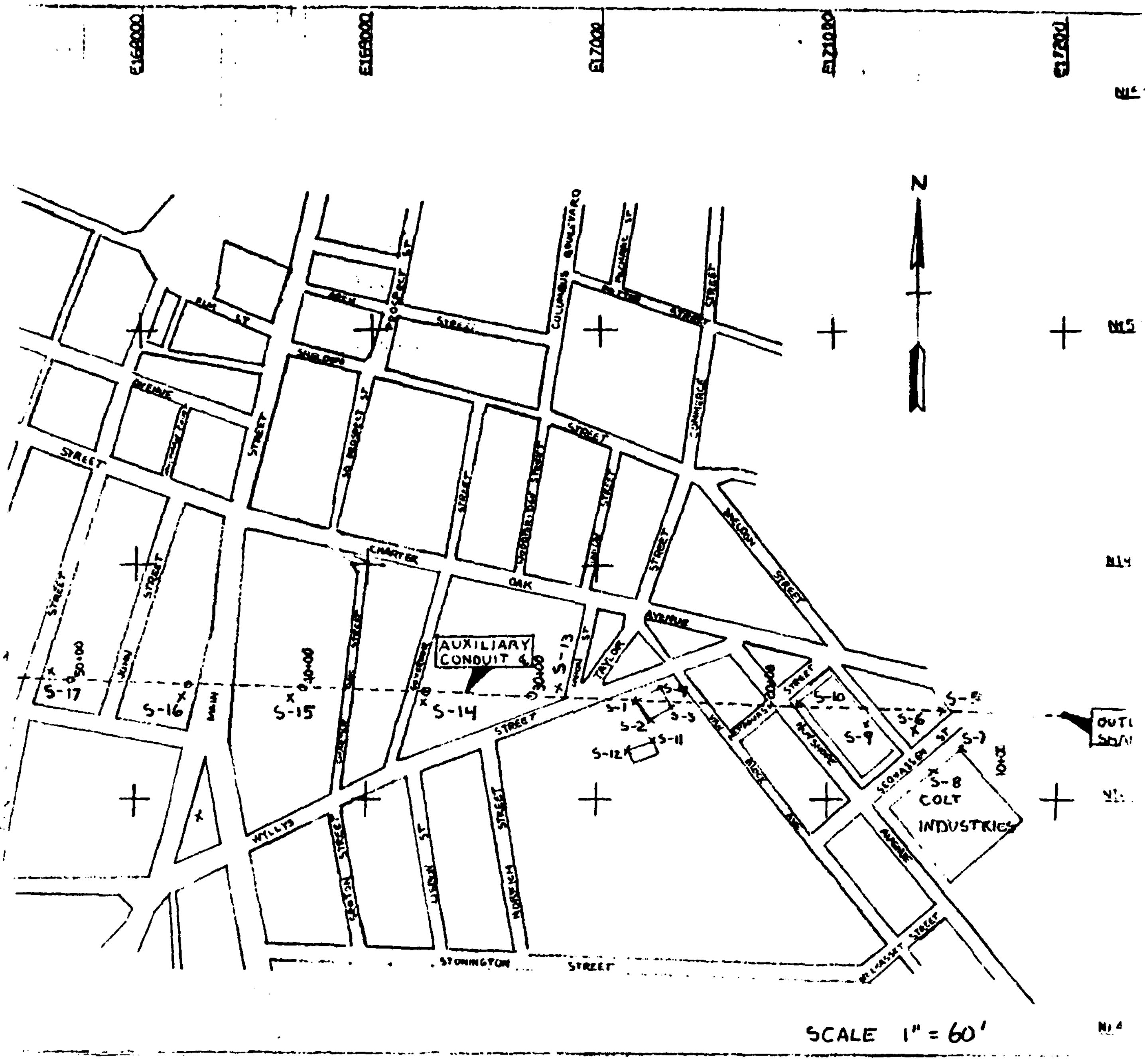
enclosures (3)

cc: Mr. Chuck Au
Mr. Roger Au
Mr. F. Moehle

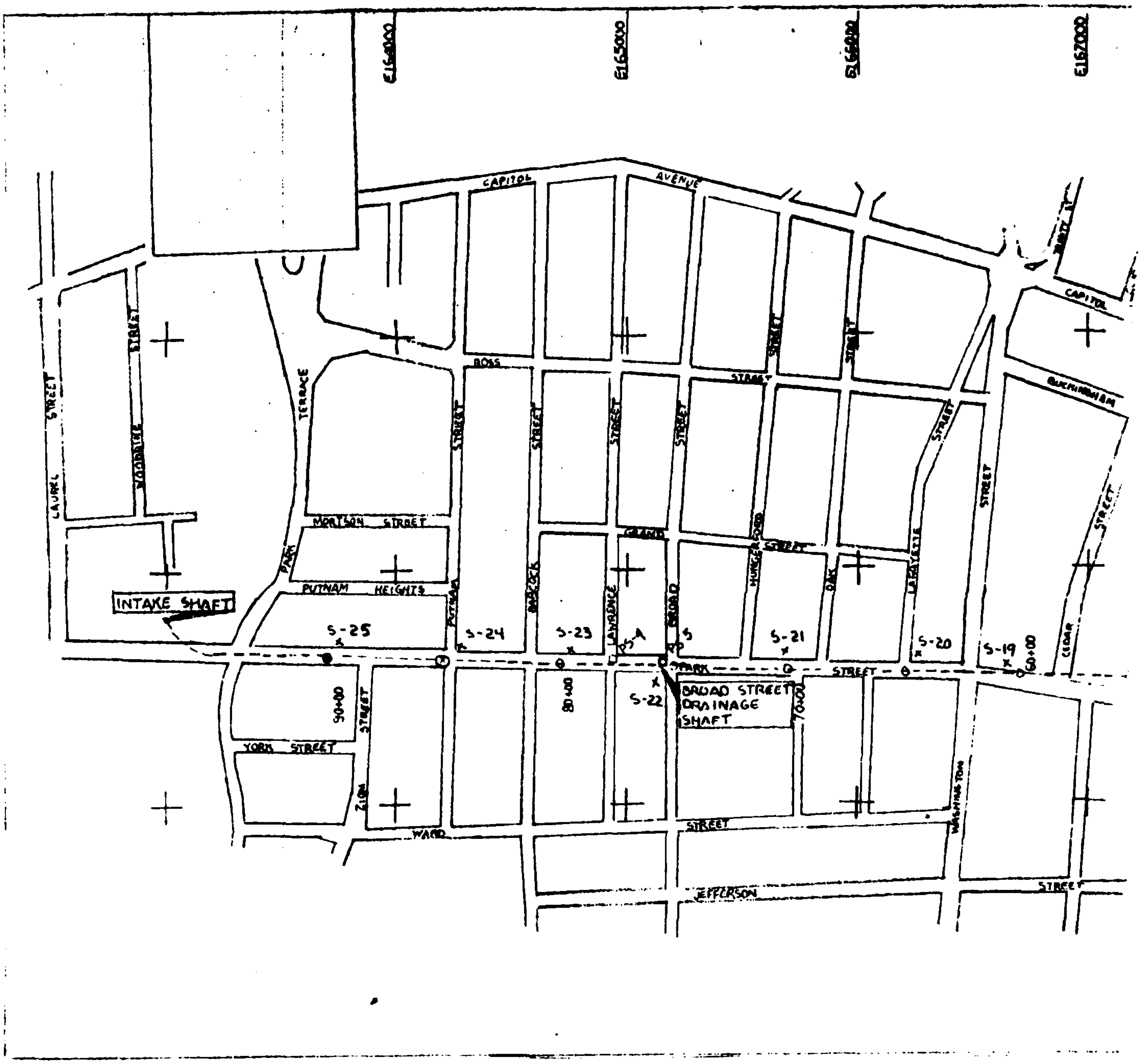
GENERAL CONTRACTORS

Sta.	Elev.	Description
S-1	23.80	Cut on ledge at Northeast side of Church Spire, East of North Entrance to Spire. Previous Point #6
S-2	25.15	Cut on ledge at Southwest corner of Church. Previous point #22
S-3	25.05	Cut on ledge at Southeast corner of Church. 15'± east of Southeast entrance door. Previous point #17
S-4	25.36	Cut on ledge at Northwest corner Good Shepharc Church. North and East of West Entrance to Church. Previous point #12
S-5	36.25	Top Northeast corner of plaque depicting high water mark of 1936 flood-located on Northeast corner State Department of Transportation, Building #17, Van Dyke Street.
S-6	28.50	Southwest corner of bottom corner stone on Colt complex building at Northeast corner Van Dyke and Sequassen Street.
S-7	23.90	Top of angle iron on colt complex building at East side Sequassen Street, 112' South of S-6 and on same building, just North of red entrance door and opposite fire hydrant.
S-8	25.12	Top of concrete window sill at Southwest corner Connecticut Department of Transportation building, same building as S-5.
S-9	22.26	On ledge 1'-9" ± above sidewalk on Northeast corner, second building West of Sequassen Street and Vredendale Avenue intersection on building with high brick stack marked Colt.
S-10	22.23	On ledge 1'-[illegible]" [illegible] at Northwest corner of same building as S-9.
S-11	28.46	Top North edge of ledge of East foundation supporting marble piller in front of Colt Memorial Parish.
S-12	28.46	Top North edge of ledge of West foundation supporting Marble piller in front of Colt Memorial Parish.

Sta.	Elev.	Description
S-13	22.39	Top Northwest corner brick window sill on North side abandoned car wash at corner Wyllys and Union, window farthest west. By centerline Sta. 30+00
S-14	35.57	Top first floor ledge at Southwest corner Stanadyne/Capewell Division Executive office Building. East side Governor Street approximately centerline Sta. 35+00.
S-15	70.16	Top ledge Northwest corner foundation of brick building behind St. Peters Church - 25' Lt. ± centerline Sta. 40+25+.
S-16	60.95	Top of top foundation stone at Southeast corner #171 Main Street at approximately 45+00.
S-17	59.98	Top corner second brick above concrete foundation at Southeast corner of operating laundry - 380 Hudson Street - near centerline Sta. 50+00.
S-18	72.64	Top East corner brick window sill on North side #12 Wadsworth Street, window farthest East. Near centerline Sta. 55+00.
S-19	82.11	Top first brick (vertical) above concrete foundation at Southeast corner building at Northeast corner Park & Washington (Nemrow) near centerline Sta. 60+00.
S-20	82.66	Top concrete at Southwest corner entrance to C.B.T. Co. on Northwest corner Washington and Park, by square marble column. Near centerline Sta. 65+00.
S-21	71.77	Top concrete ledge East of entrance to U.S. Post Office, just north of flag pole near centerline Sta. 70+00.
S-22	69.21	Top stone foundation at Southeast corner #598 Park - near centerline Sta. 75+00.
S-23	77.99	Top stone ledge 2' West of Southeast corner building at 712 Park Street, near centerline Sta. 80+00.
S-24	90.79	Top Southwest corner of corner stone at Southwest corner of Sainte Anne Church at Northeast corner Putnam and Park, near centerline Sta. 85+00.
S-25	92.09	Southwest corner first step to hse #896-898 on North side Park Street near centerline Sta. [illegible]0+00.



E16000
E16500
E17000
N
AUXILIARY CONDUIT ℄
S-17
S-16
S-15
S-14
S-13
S-10
S-9
S-8
S-7
S-6
S-5
S-3
S-2
S-11
S-12
COLT INDUSTRIES
CHARTER
OAK
AVENUE
WYLLYS
STREET
STONINGTON
STREET
COLUMBUS BOULEVARD
COMMERCE
SCALE 1" = 60'



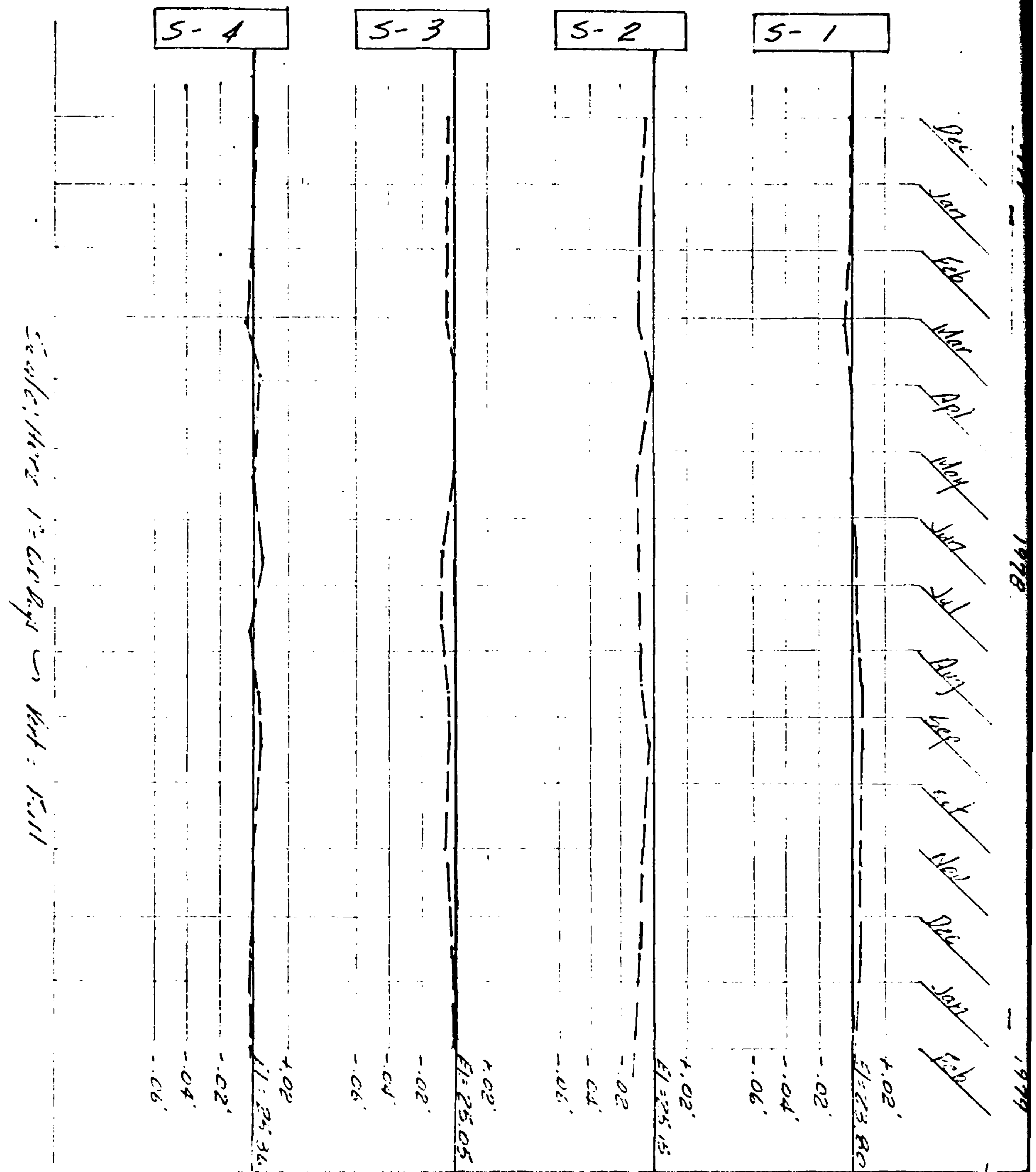
E164000
E165000
E166000
E167000
CAPITOL
AVENUE
CAPITOL
BOSS
STREET
BUCKINGHAM
LAUREL STREET
WOODBINE STREET
PARK TERRACE
MORTSON STREET
PUTNAM HEIGHTS
INTAKE SHAFT
PUTNAM STREET
BABCOCK STREET
LAWRENCE STREET
BROAD STREET
GRAND STREET
HUNGERFORD STREET
OAK STREET
LAFAYETTE STREET
CEDAR STREET
S-25
S-24
S-23
S-21
S-20
S-19
S-22
PARK STREET
BROAD STREET DRAINAGE SHAFT
90+00
80+00
70+00
60+00
YORK STREET
ZION STREET
WARD STREET
WASHINGTON
JEFFERSON STREET

Roger J. Au & Son, Inc

Park River Project
Hartford Ct.
Subsidence
Survey

Dec 1977 to
Feb 1979

1/7

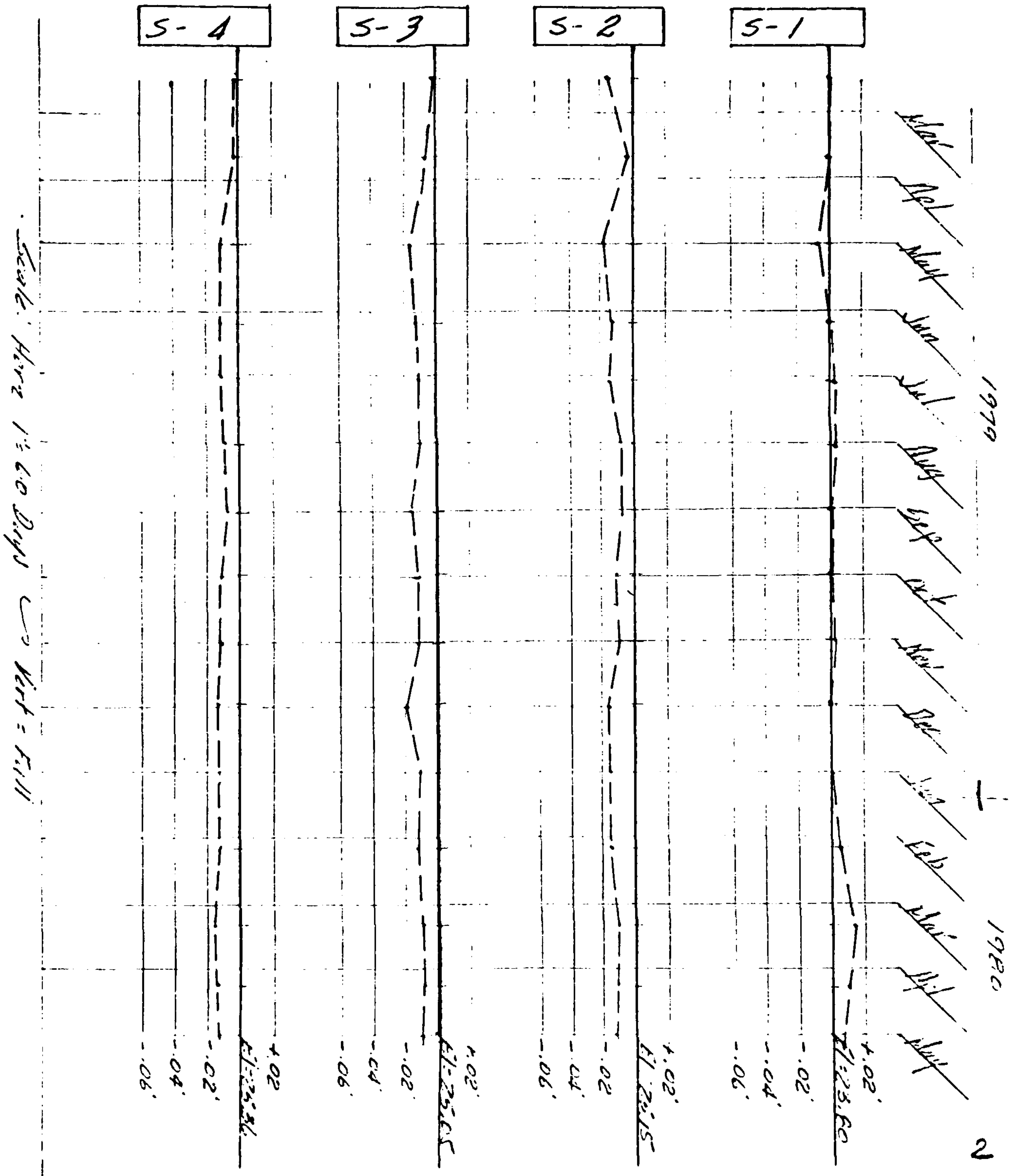


Roger J. Au & Son, Inc

Park River Project
Hartford Ct.
Subsidence
Survey

Mar 1979 to
May 1980

1/7

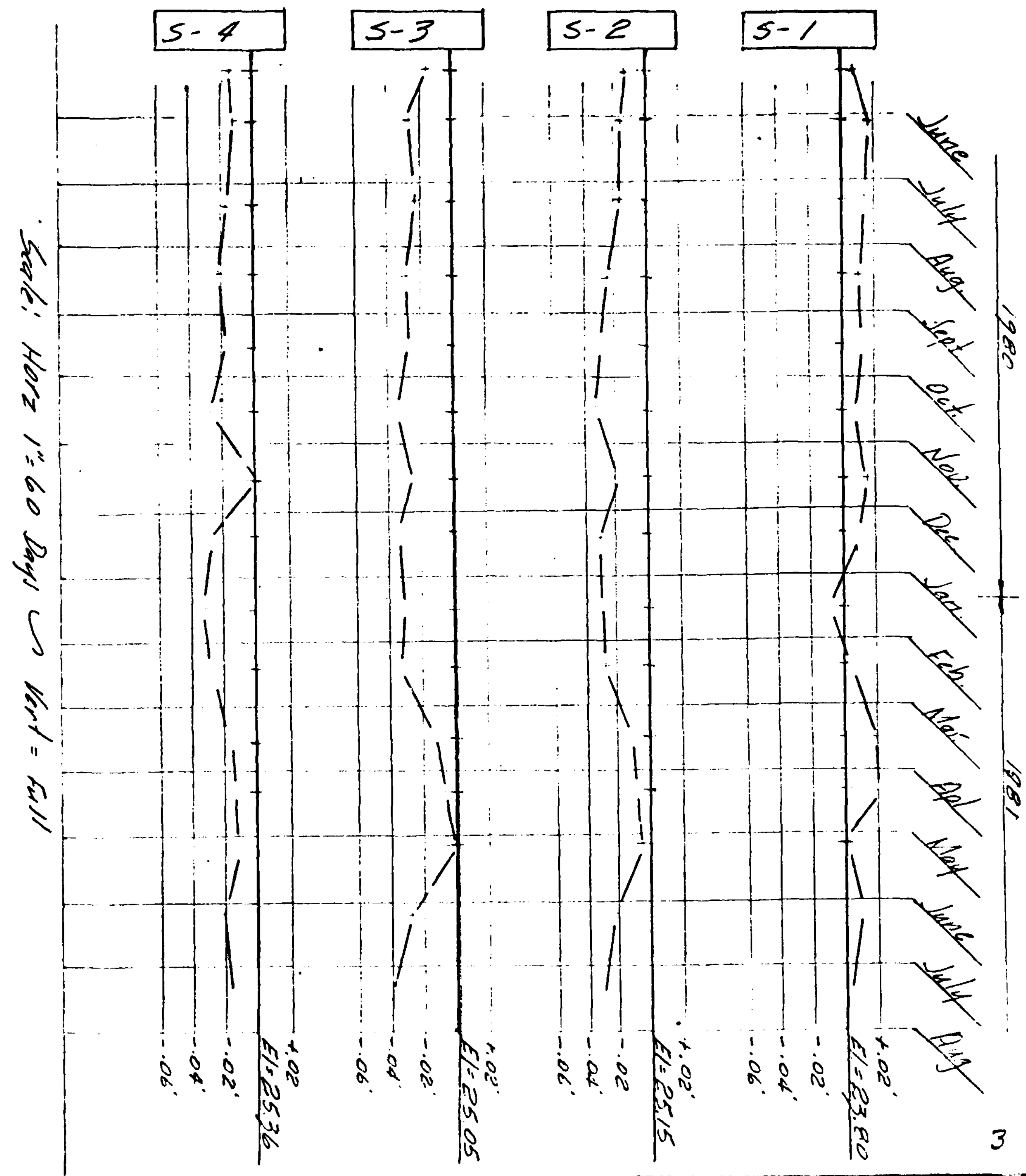


Roger J. Au & Son, Inc.
P.O. Box 148E
Mansfield, Ohio 44901

Park River Project
Hartford Ct.
Subsidence Survey

June 1980 to Aug 1981

1/7



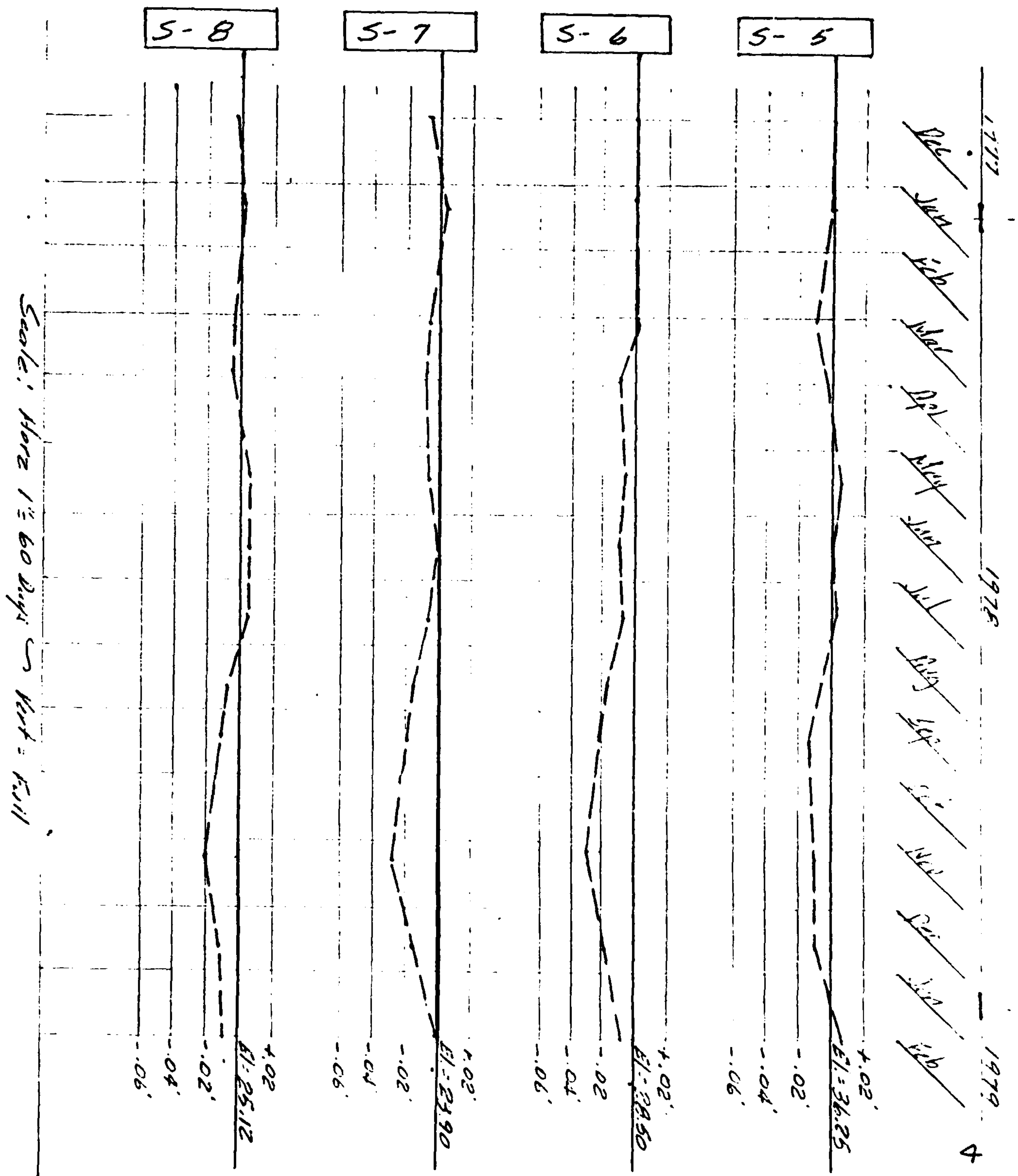
VALUABLE

Roger J. Au & Son, Inc

Park River Project
Hartford Ct.
Subsidence
Survey

Dec 1977 to
Feb 1979

2/7



VALUABLE

Roger J. Au & Son, Inc.

Park River Project
Hartford Ct.
Subsidence
Survey

Mar 1979 to
May 1980

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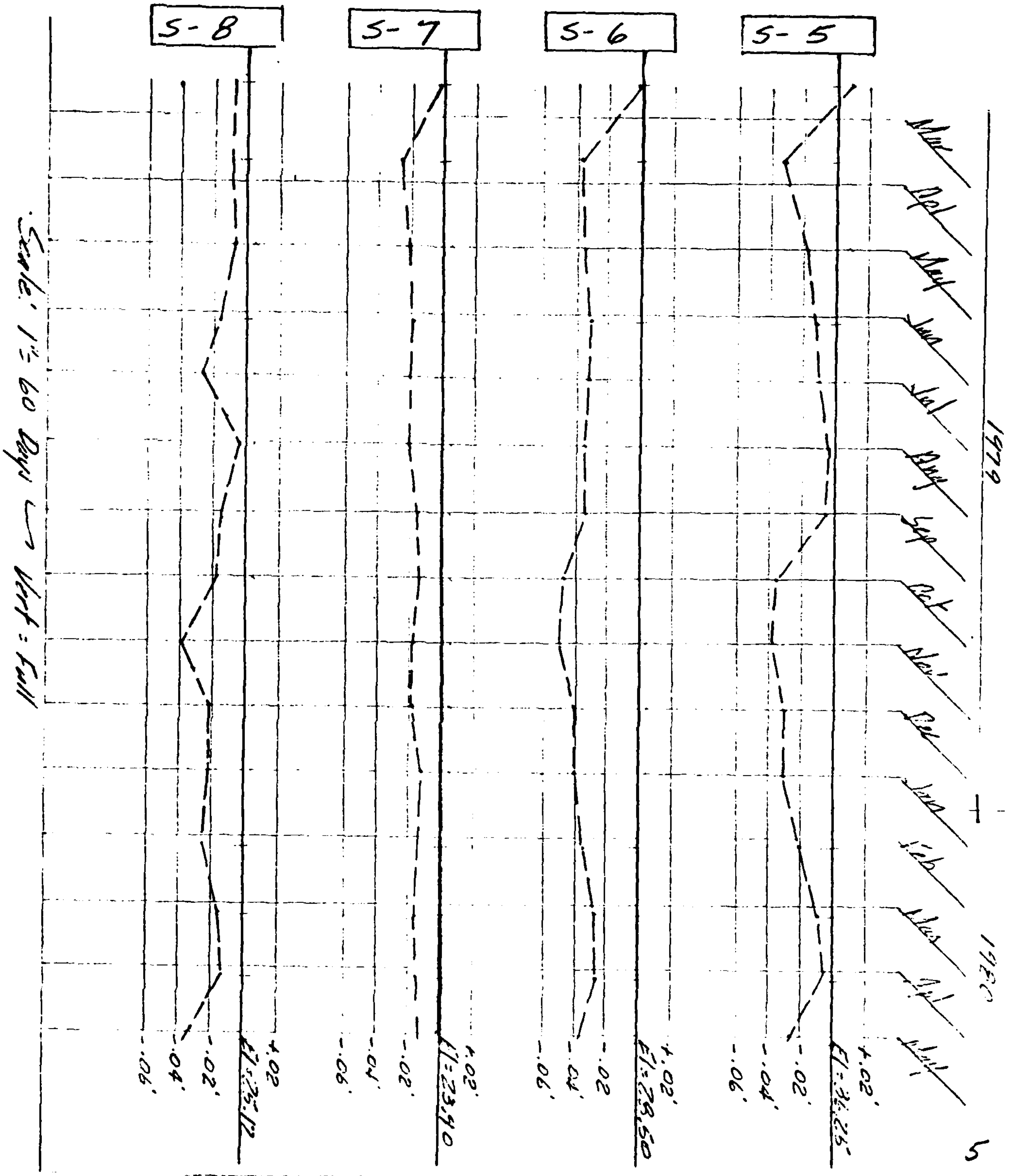

Roger J. Au & Son, Inc.
P.O. Box 1489
Mansfield, Ohio 44901

Park River Project
Hartford Ct.
Subsidence
Survey

June 1980 to
Aug 1981

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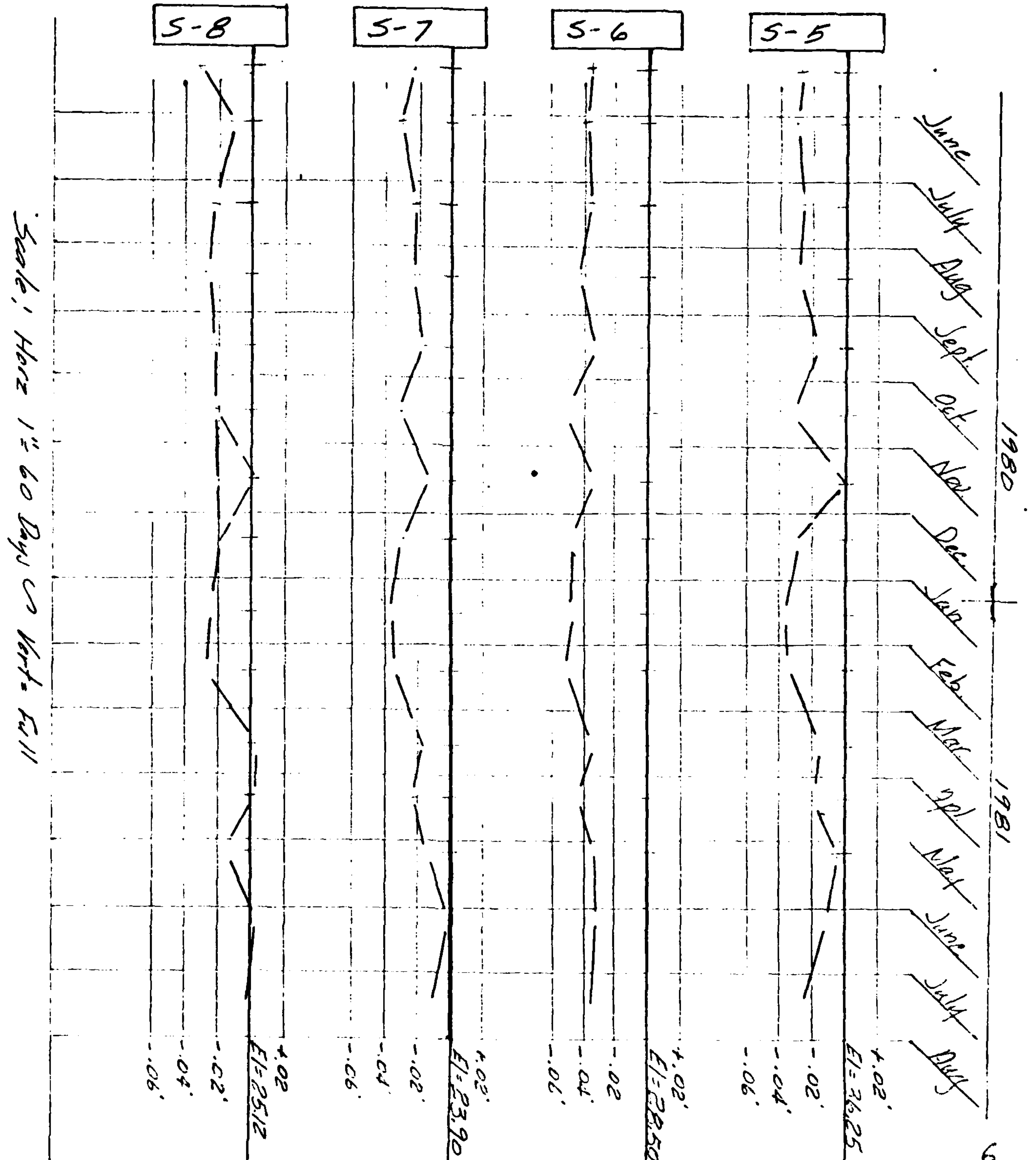

LUABLE

ger J. Au & Son, Inc.

SUBJECT Park River Project

Hartford Ct.

Subsidence

Survey

Dec 1977 to
Feb 1979

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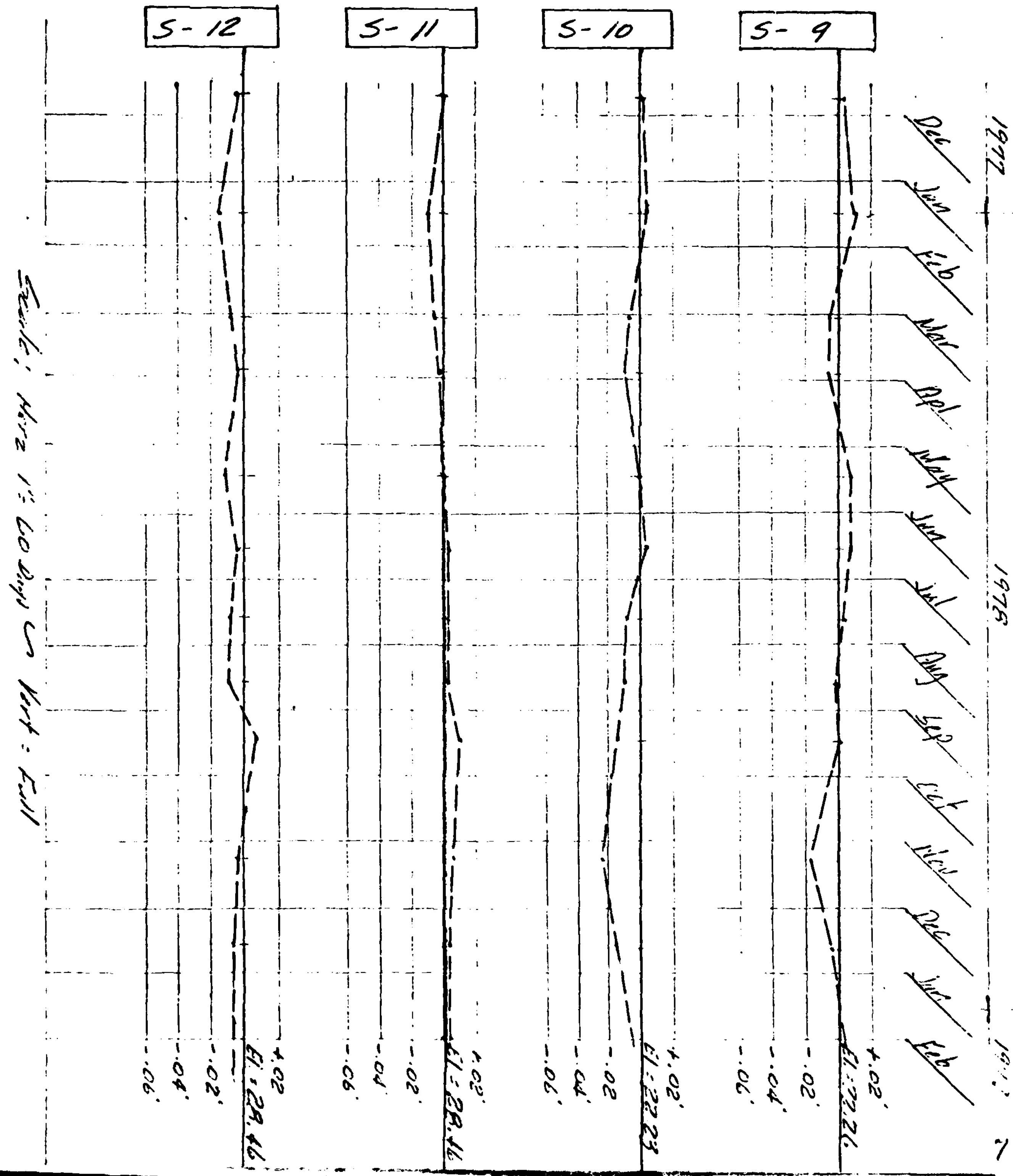

VALUABLE

Roger J. Au & Son, Inc.
P.O. Box 1488
MANSFIELD, Ohio 44901

SUBJECT Park River Project
Hartford Ct.
Subsidence
Survey

Mar 1979 to
May 1980

3
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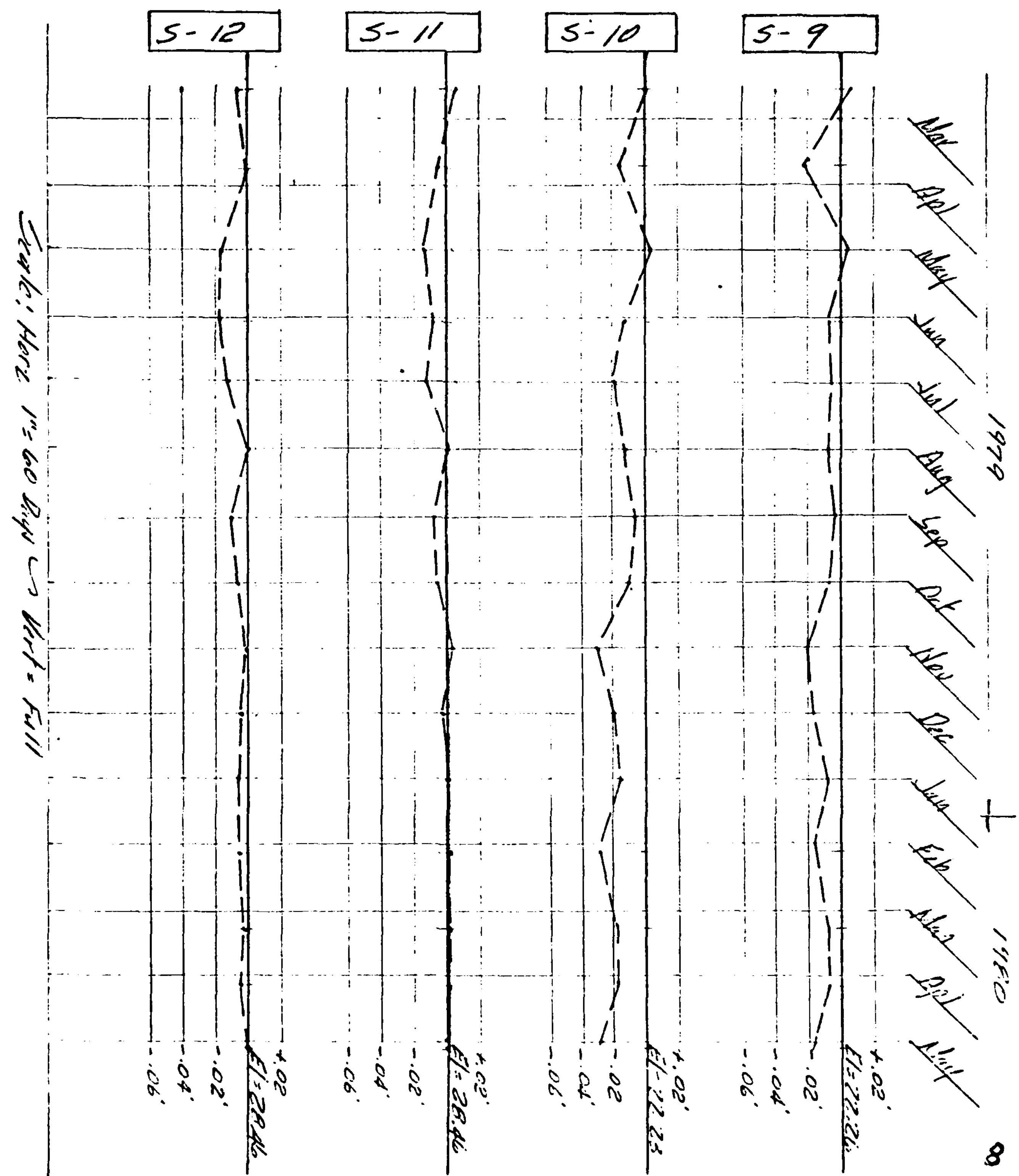

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Au & Son, Inc

Park River Project
Hartford Ct.
Subsidence
Survey

June 1980 to Aug 1981

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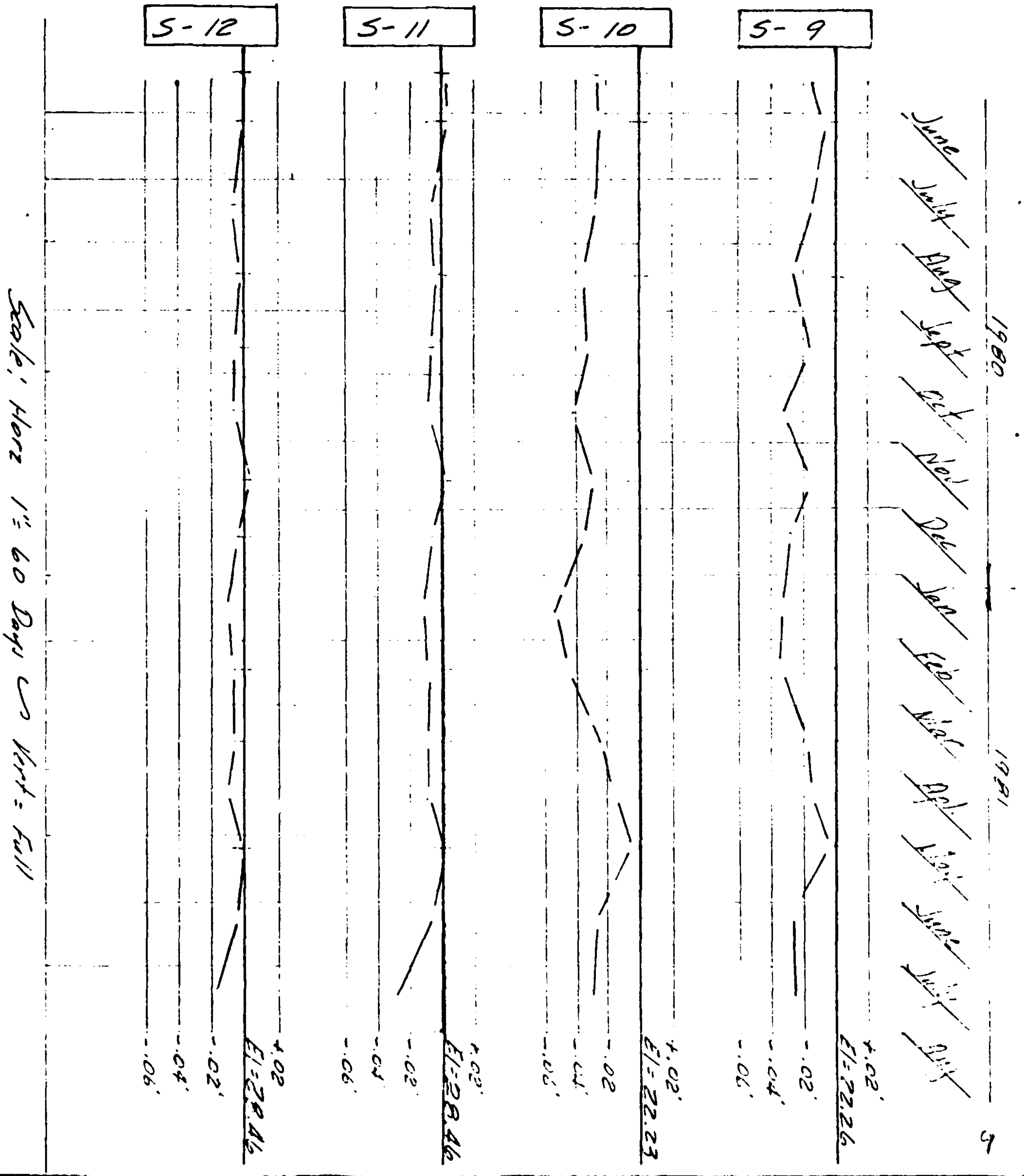

Roger J. Au & Son, Inc.

Park River Project
Hartford Ct.
Subsidence
Survey

Dec 1977 to
Feb 1979

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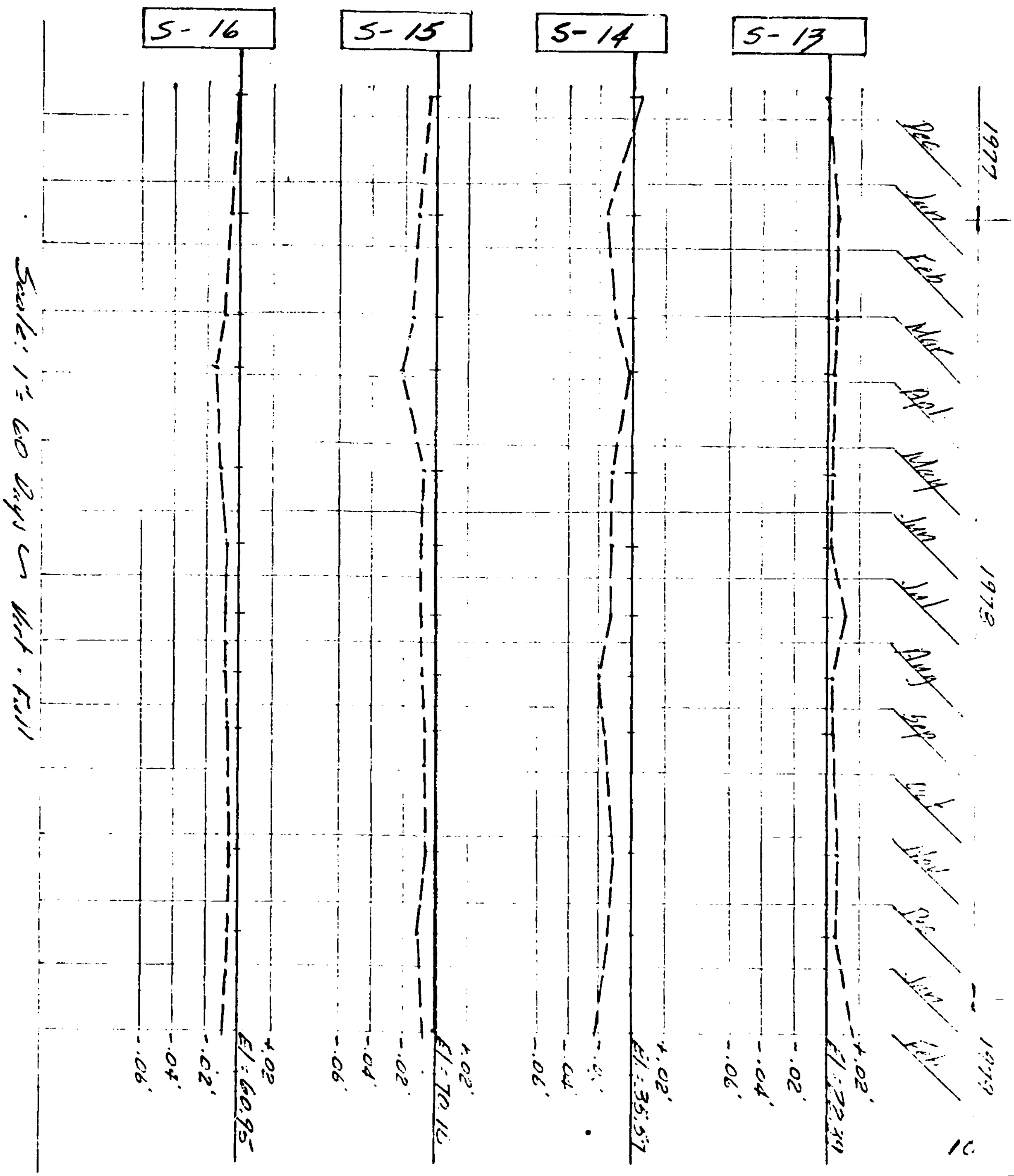


10

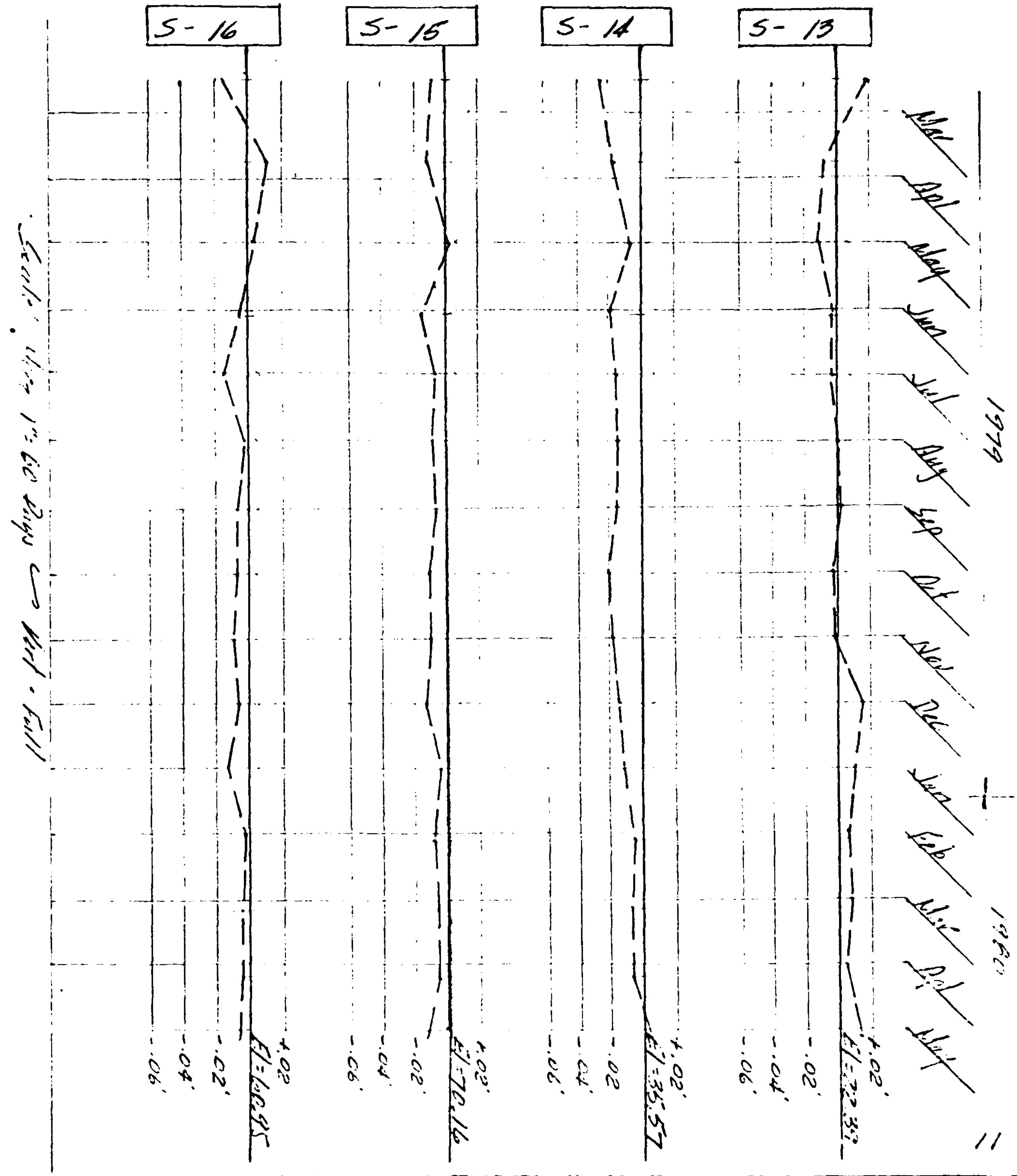
VALUABLE

Roger J. Au & Son, Inc.
P.O. Box 14??
??? Ohio 44??

Park River Project
Hartford Ct.
Subsidence
Survey

Mar 1979 to
May 1980

4
7



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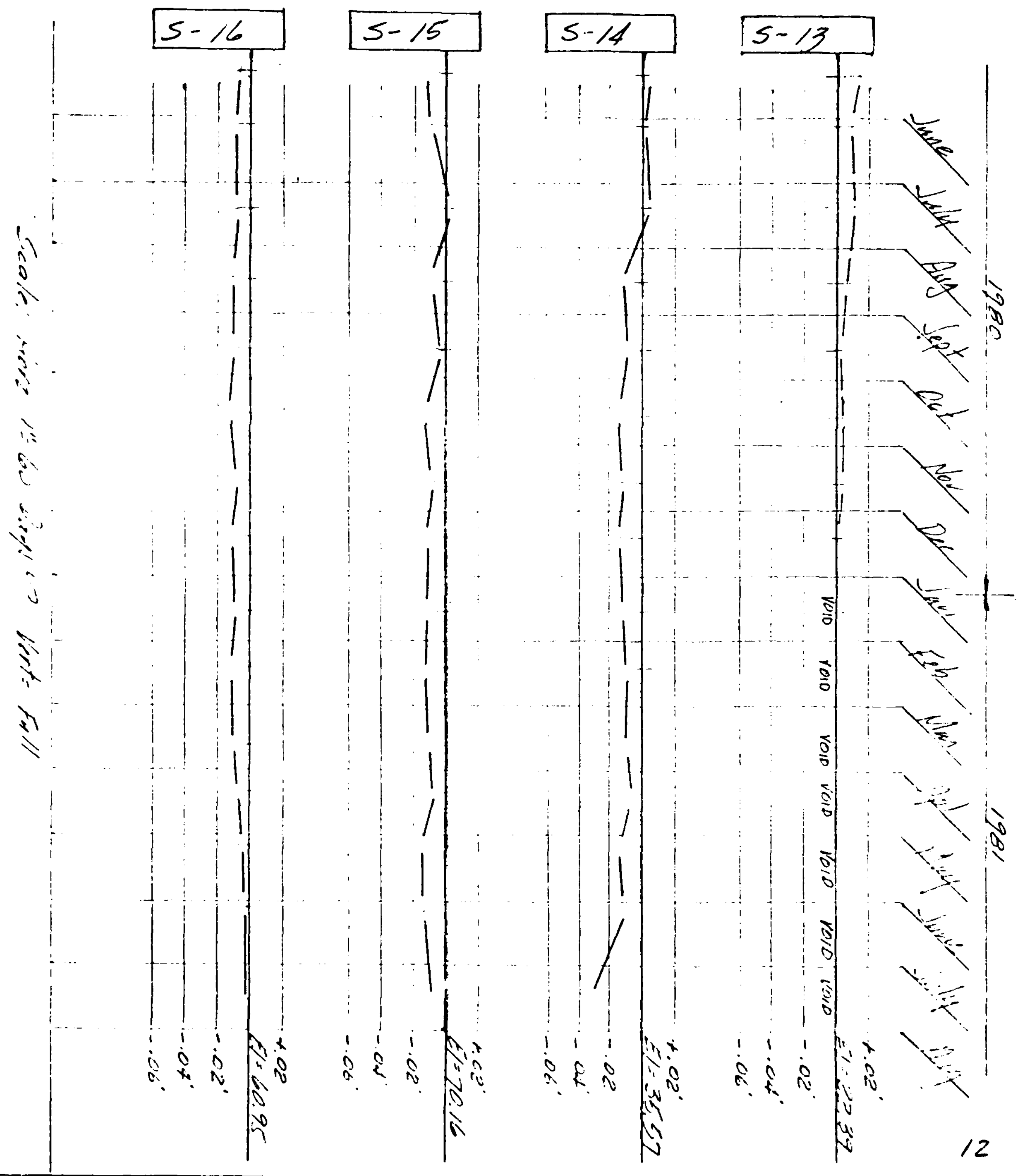
Roger J. Au & Son Inc

Park River Project
Hartford Ct.
Subsidence
Survey

June 1980 to
Aug 1981

4
7



Park River Project
Hartford Ct.
Subsidence
Survey

Dec 1977 to
Feb 1979

5

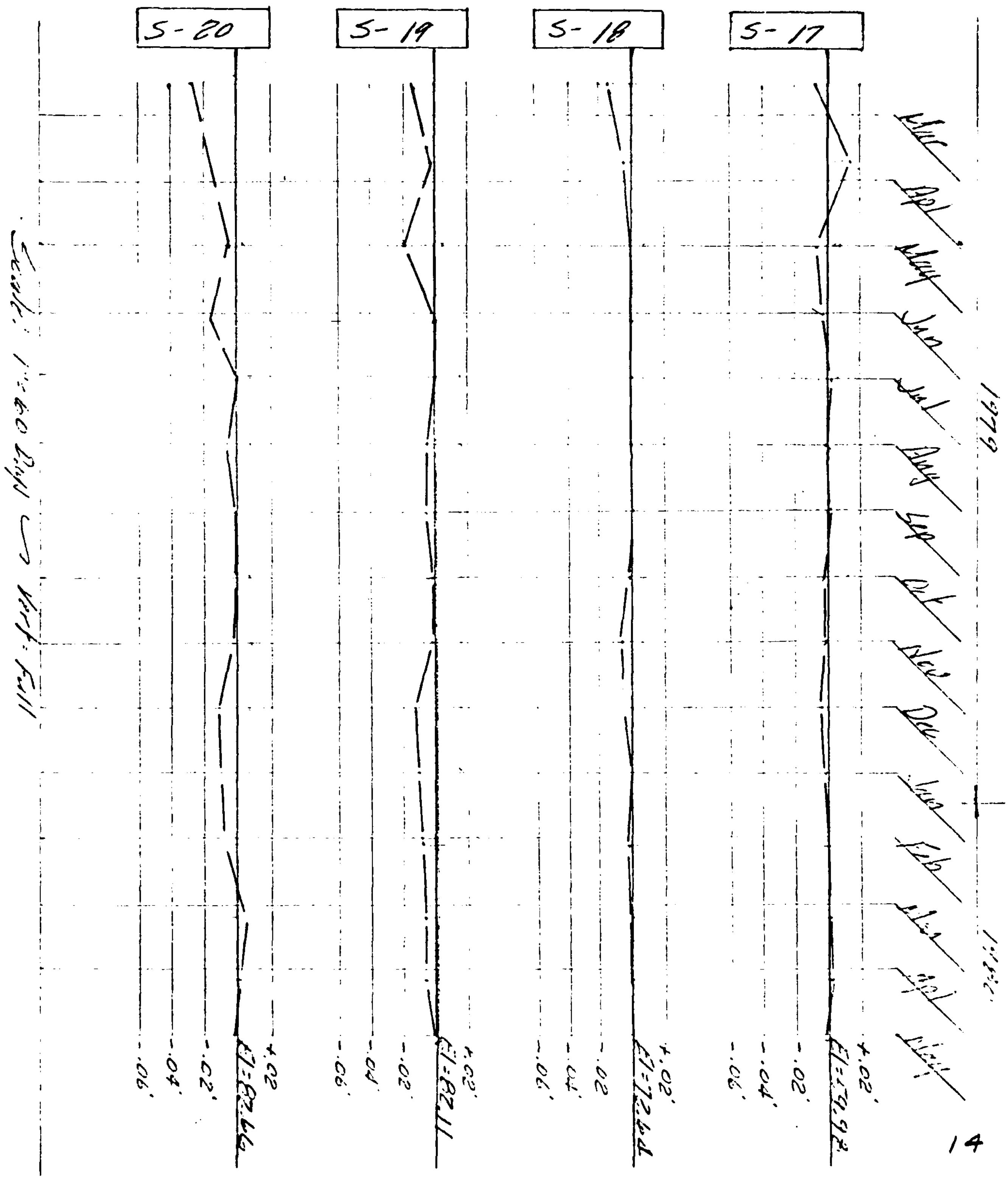
7

Roger J. Au & Son Inc

Park River Project
Hartford Ct.
Subsidence
Survey

March 1979 to
May 1980

5
7

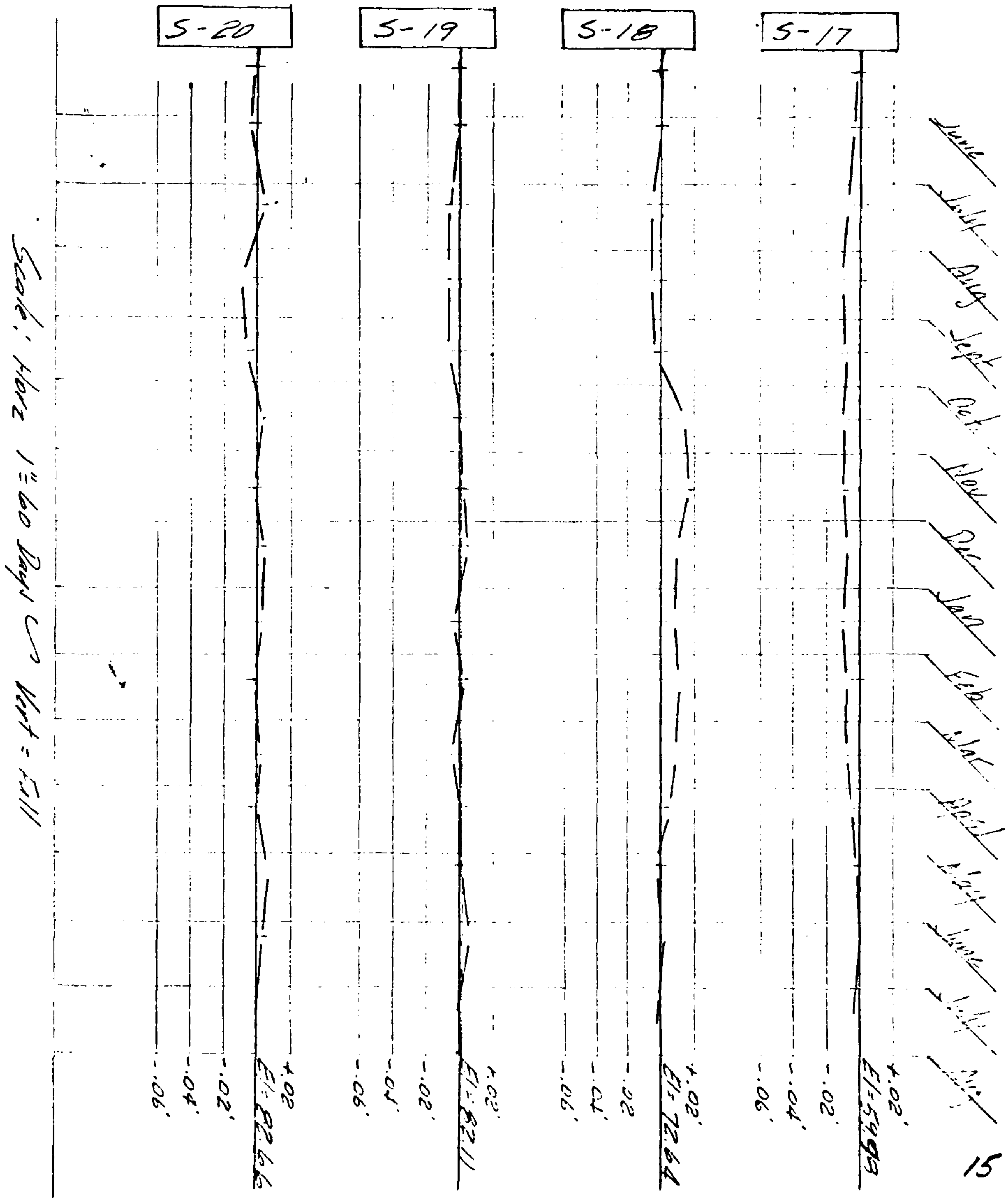


Roger J. Au & Son Inc.

Park River Project
Hartford Ct.
Subsidence
Survey

June 1980 to
Aug 1981

5
7



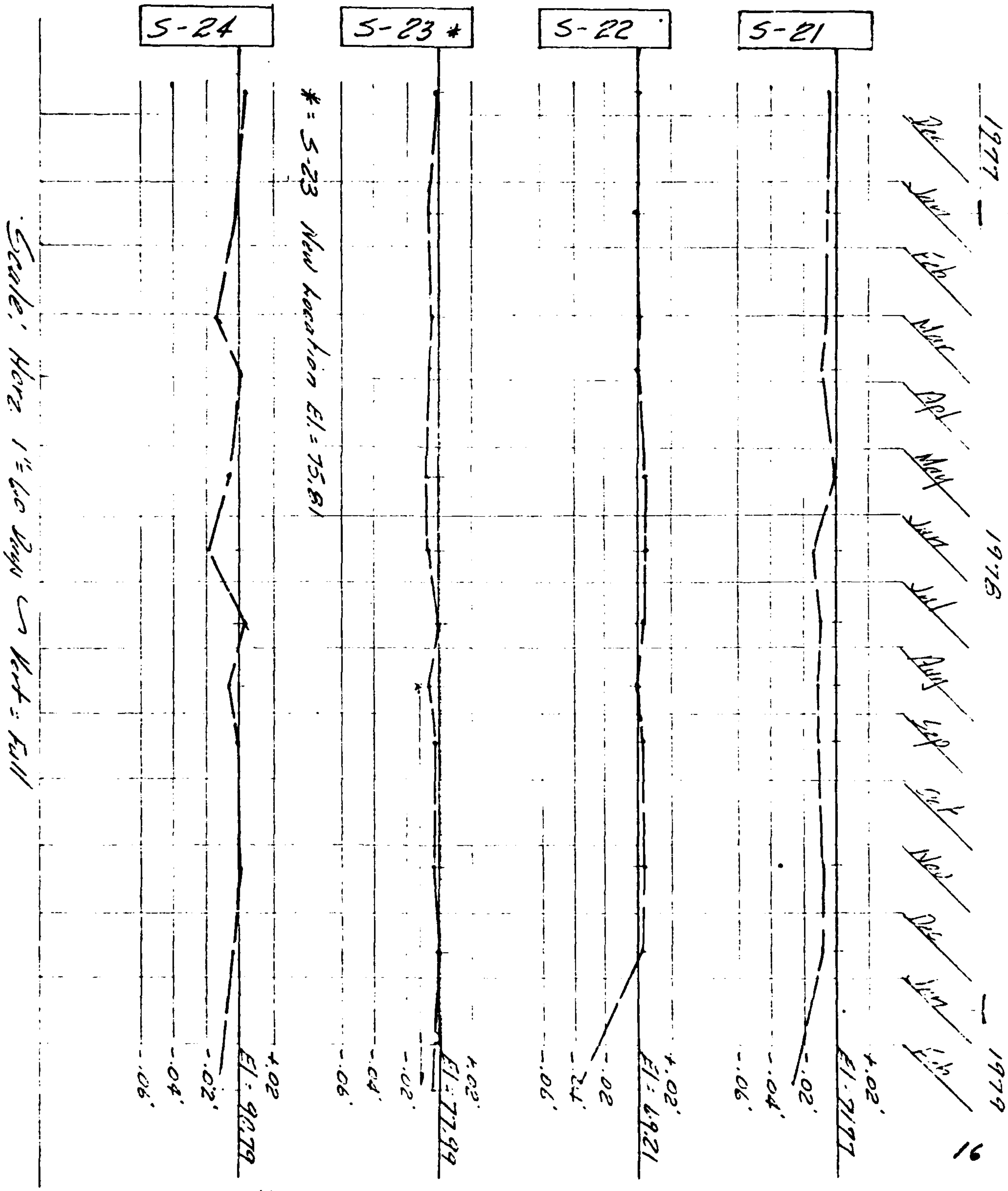
VALLABLE

Park River Project
Hartford Ct.
Subsidence
Survey

Dec 1977 to
Feb 1979

Roger J. Au & Son, Inc.

6
7



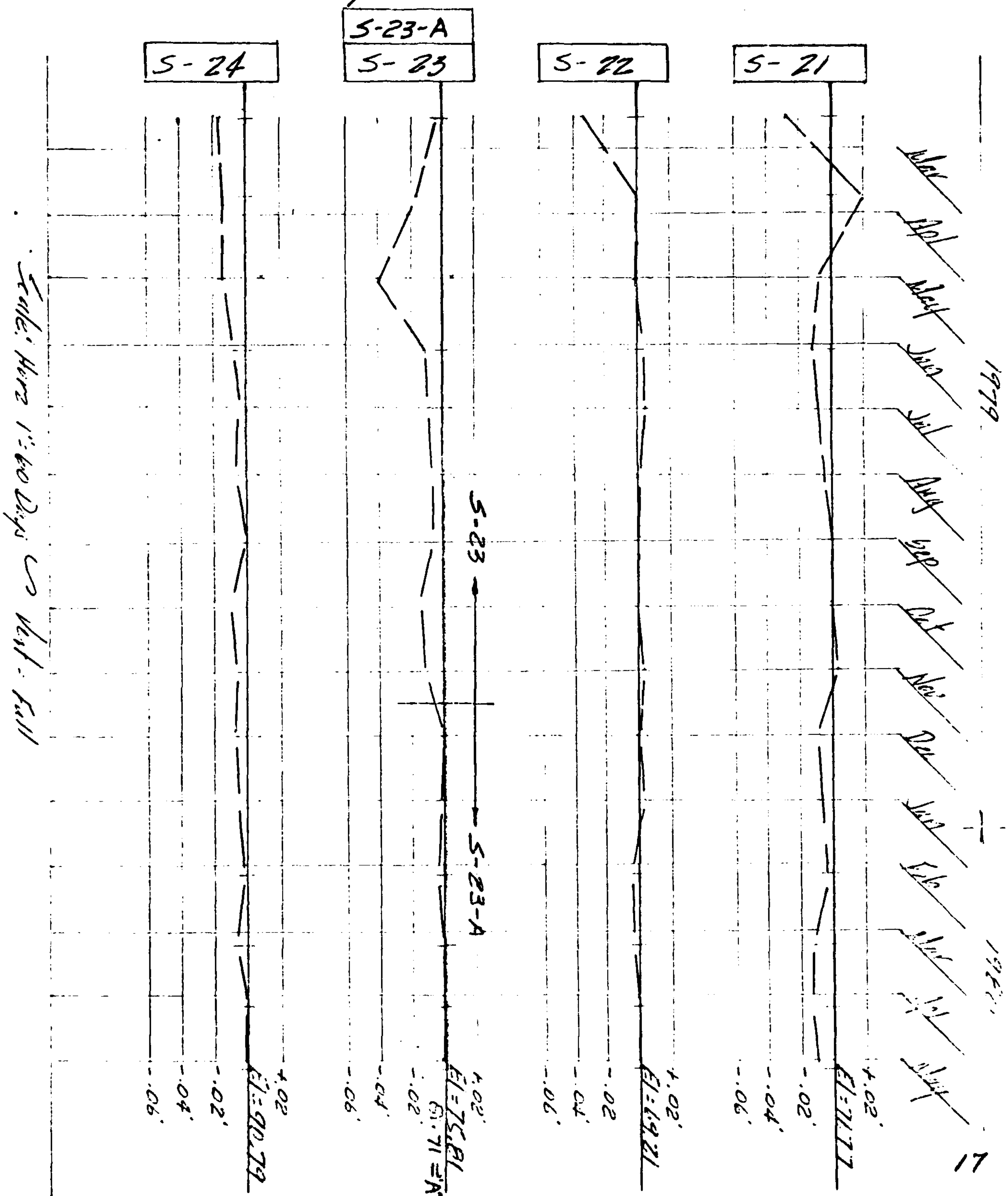
RELIABLE

Roger J. Au & Son, Inc

Park River Project
Hartford Ct.
Subsidence
Survey

March 1979 to
May 1980

6
7

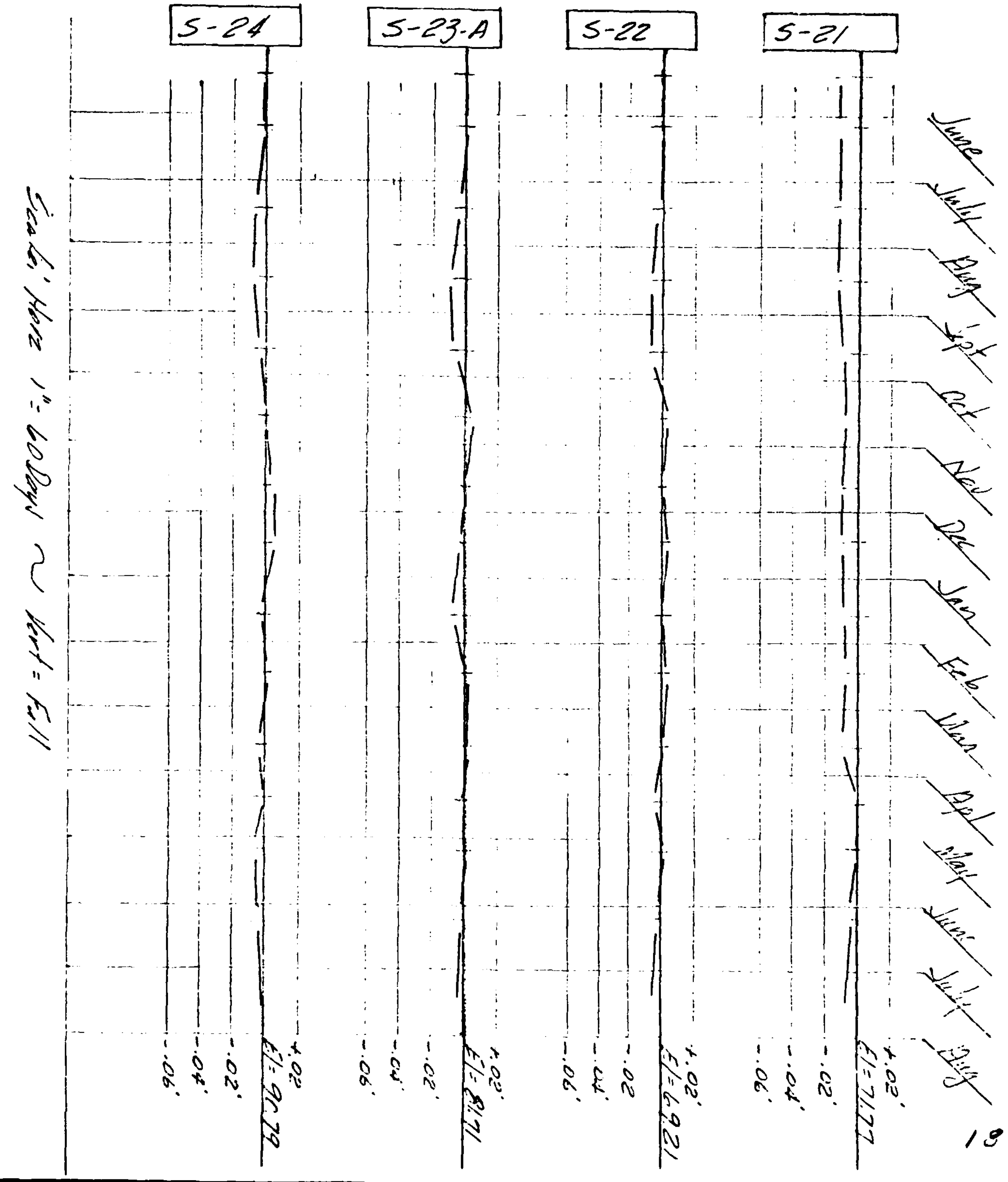


Roger J. Au & Son, Inc.
P.O. Box 1488
Massillon, Ohio

Park River Project
Hartford Ct.
Subsidence
Survey

JUNE 1980 to
Aug 1981

6
7

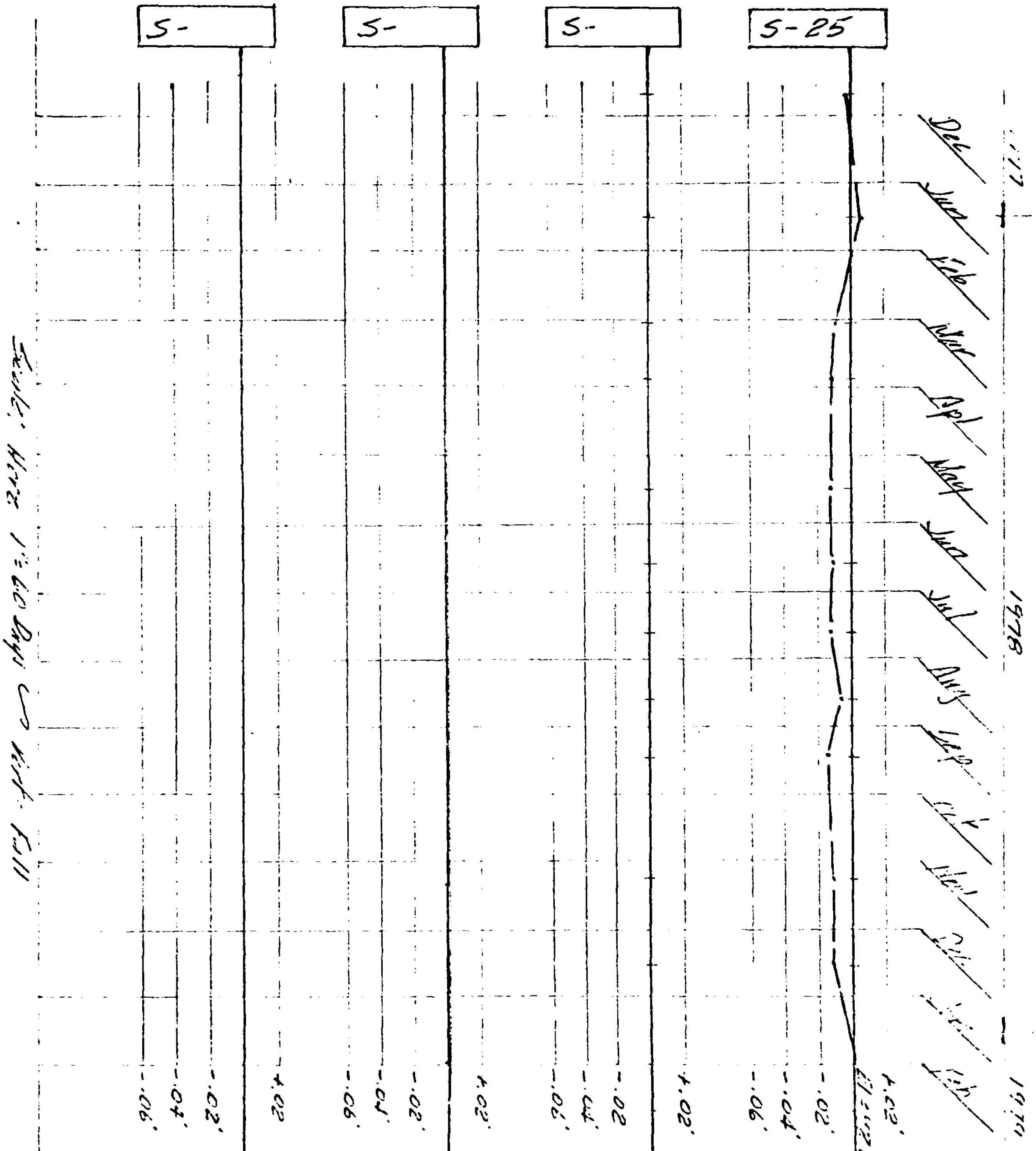


Roger J. Au & Son, Inc
P.O. Box ...
Mansfield, Ohio 44...

Park River Project
Hartford Ct.
Subsidence
Survey

Dec 1977 to
Feb 1979

7
7



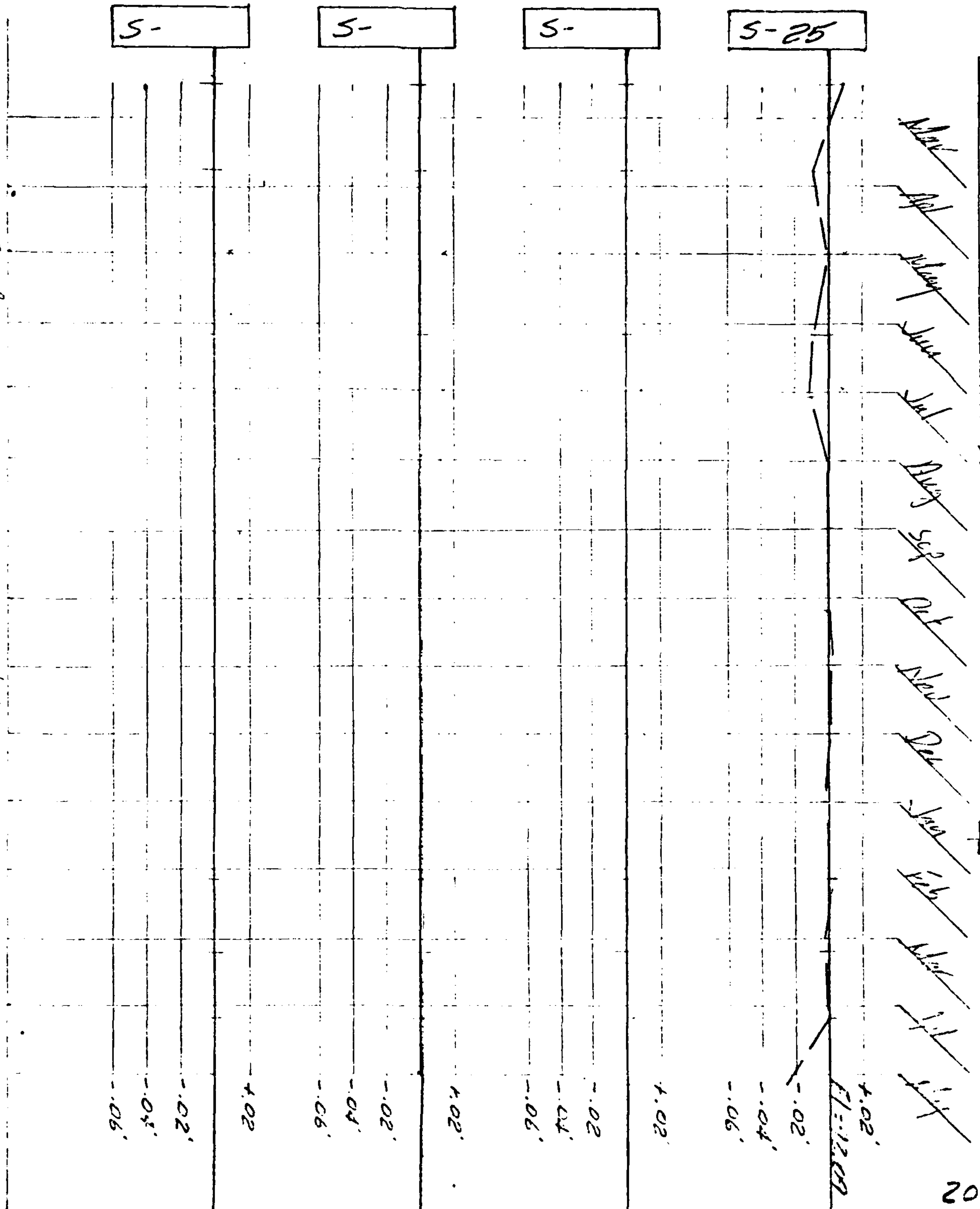
Roger J. Au & Son, Inc.

Park River Project
Hartford Ct.
Subsidence
Survey

March 1979 to
May 1980

7

7



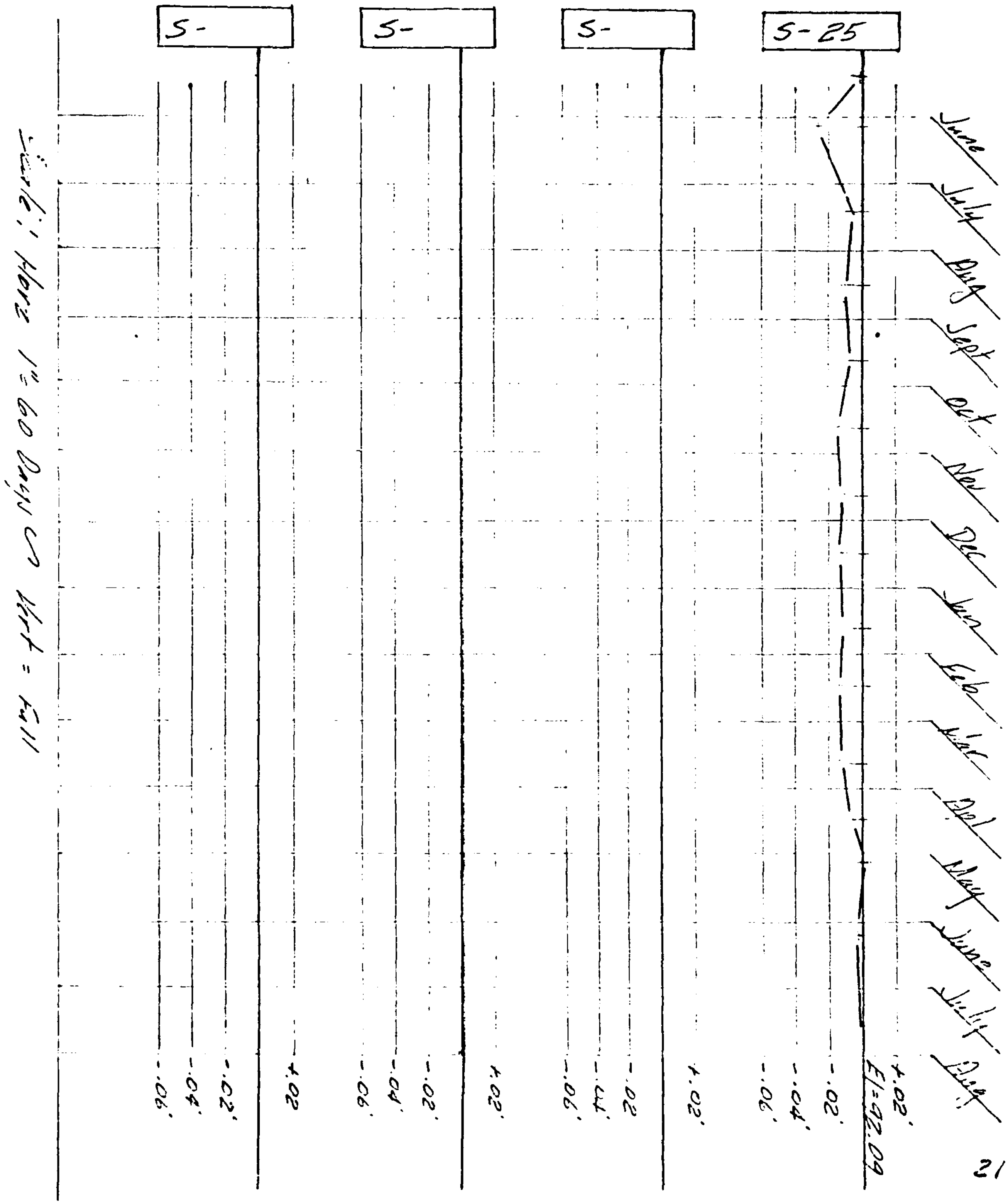
20

Roger J. Au & Son, Inc.

Park River Project
Hartford Ct.
Subsidence
Survey

JUNE 1980 to
Aug 1981

7
7



TABLE

Roger J. Au & Son, Inc.
Firelands Sewer & Water Construction Co. Inc.
P.O. Box 1488
Mansfield, Ohio 44901

SUBJECT Reference + location B.M. set @ S.W. Cor. Park St. + Park Terrace @ Pope Park, Hartford, Conn

C.M. IN CHARGE

R.T. INSTRUMENT

D.T. ROD

ROD

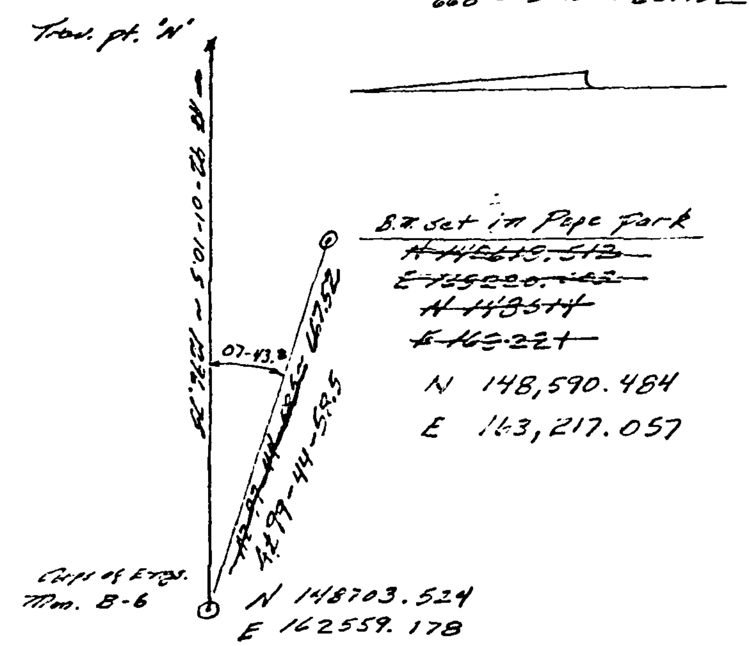
12/13/77 DATE

NOTES FIG

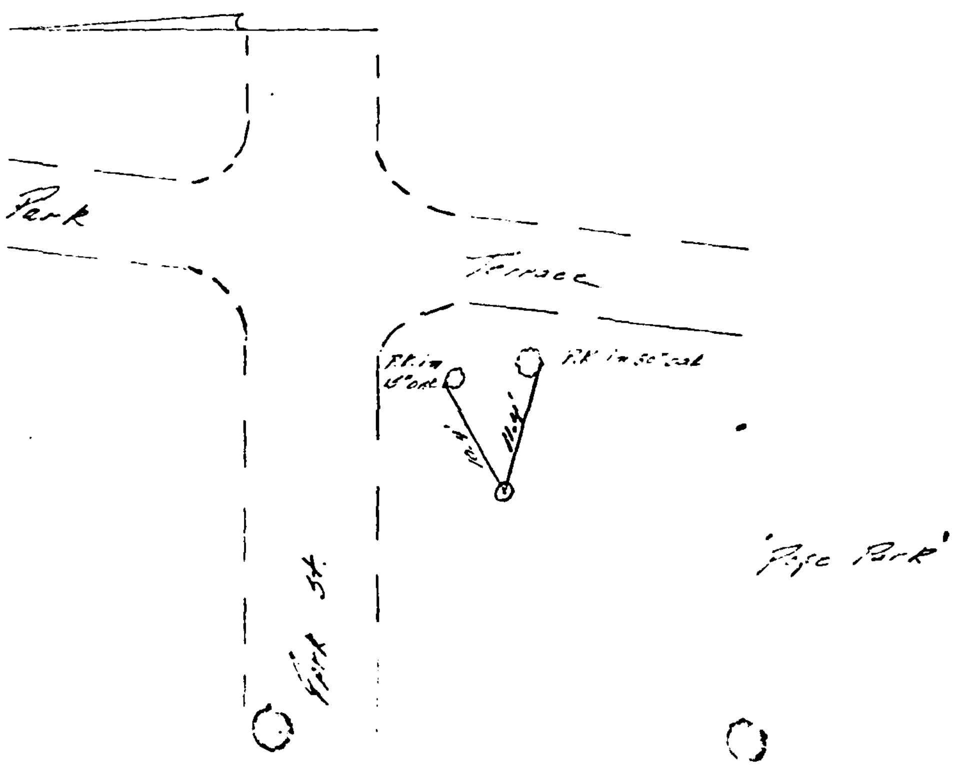
NOTES CH'D

1/3

668' @ 2°-10' = 667.52



62.47' LT. STA.
~~37.34' Lt. Sta.~~
~~95+31.8~~
95+34.7



TABLE

Roger J. Au & Son, Inc.

Firelands Sewer & Water Construction Co., Inc

P O Box 1488

Mansfield Ohio 44901

SUBJECT ELEVATIONS OF PERMANENT BENCH MARKS

PARTY

S.M. IN CHARGE

K.T. INSTRUMENT

D.T. ROD

ROD

12/14/77 DATE

NOTES FIGURED

NOTES CHECKED

STATION	+	HI	−	ELEVATION	DESCRIPTION
T.B.M. #4	4.088	23.446		19.358	TOP OF NUT ON SOUTH ANCHOR BOLT @ N.W. END OF CONC [illegible], N.W. OF OUTLET CHANNEL
PERMANENT B.M.	8.000	23.149	8.297	15.149	PERMANENT B.M. [illegible] OF [illegible], STA. [illegible], [illegible] RT. (TO [illegible])
T.B.M. #4			3.792	19.357	([illegible])
T.B.M. #12	5.120	63.537		[illegible]	N.E. BOLT [illegible] HYD @ MAIN [illegible] [illegible] @ N. END OF [illegible]
PERMANENT B.M.	[illegible]	[illegible]	5.043	[illegible]	PERMANENT B.M. SET [illegible] OF 5', STA. 44+29.8, [illegible]
T.B.M. #12			3.601	54.514	([illegible])
T.B.M. #25	3.525 ~~[illegible]~~	33.657 ~~[illegible]~~		[illegible]	N.W. COR. [illegible] LIGHT POLE # [illegible] @ S.E. COR. [illegible]
PERMANENT B.M.	[illegible]	82.505 ~~[illegible]~~	[illegible] ~~[illegible]~~	77.172 ~~[illegible]~~	PERMANENT B.M. SET TO [illegible] OF [illegible], STA. [illegible], [illegible]
T.B.M. #25			[illegible]	30.129 ~~[illegible]~~	([illegible])

Roger J. Au & Son, Inc.
ELANDS SEWER & WATER CONSTRUCTION CO. INC
P O BOX 1488
MANSFIELD, OHIO 44901

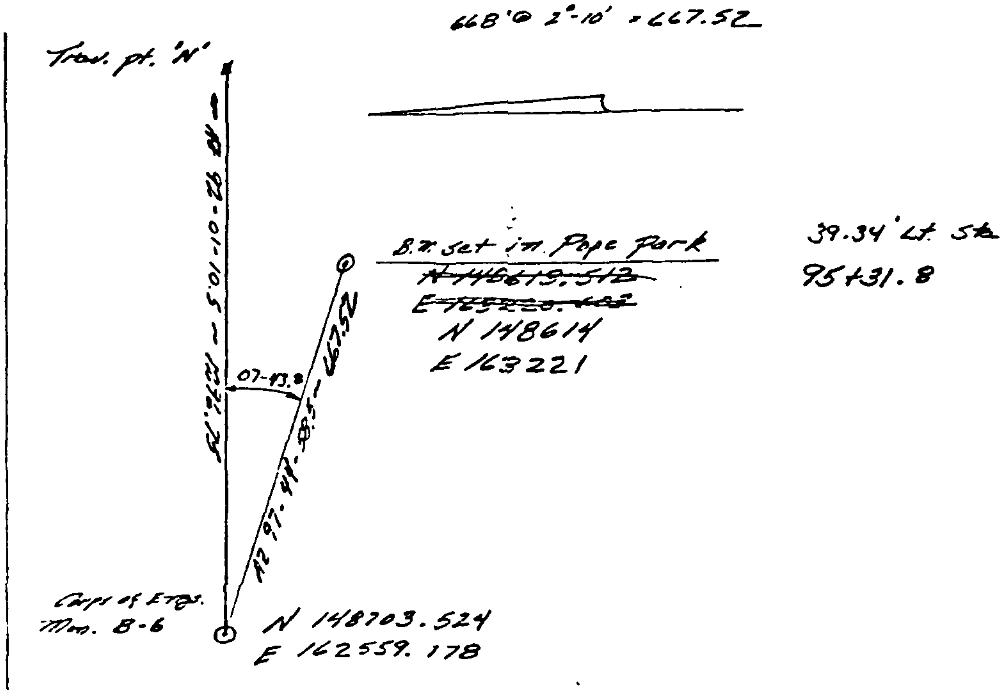
SUBJECT: References + location
B.M. set @ S.W. Cor.
Park St. + Park Terrace
in Pope Park, Hartford, Conn

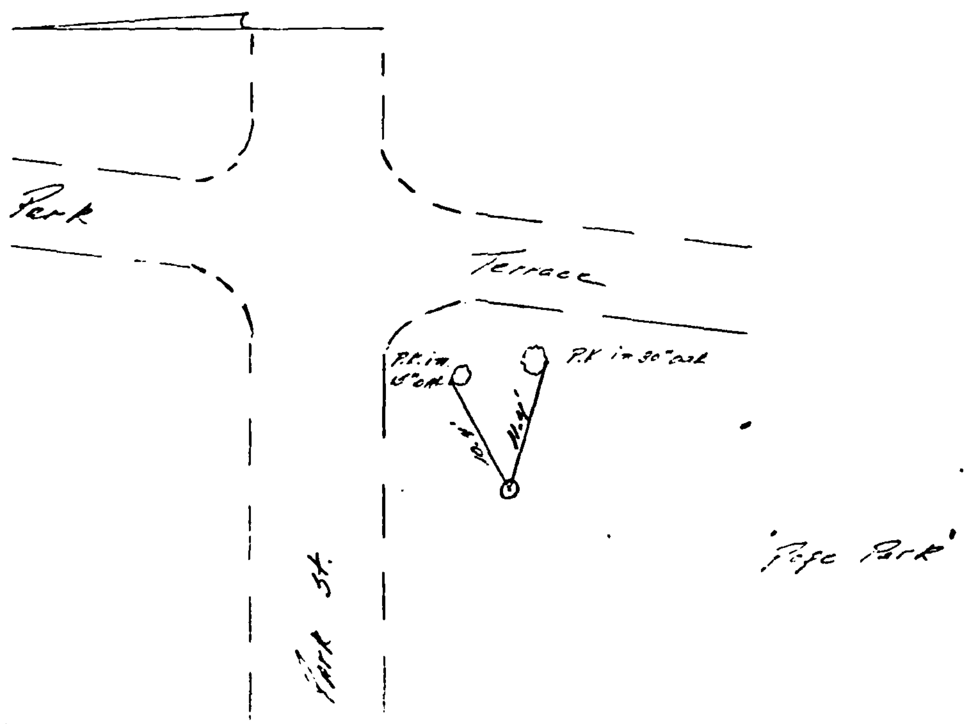
C.M. IN CHARGE
R.T. INSTRUMENT
D.T. ROD
ROD

12/13/77 DATE

NOTES FIGURED
NOTES CHECKED

1/3





...er J. Au & Son, Inc.
...LANDS SEWER & WATER
...NSTRUCTION CO. INC
P.O. BOX 1488
...ANSFIELD OHIO 44901

SUBJECT Reference Location BM set in Brainard Park - Main St + Park St Hartford, Conn.

PARTY

C.H. IN CHARGE

R.T. INSTRUMENT

D.T. ROD

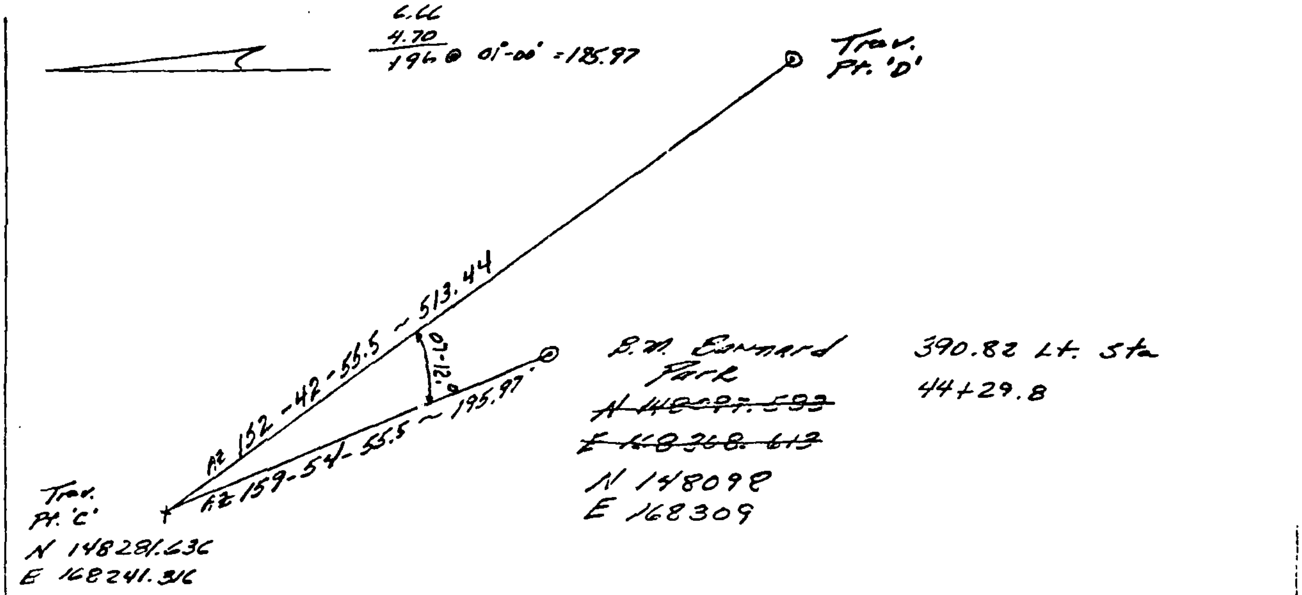
_______ ROD

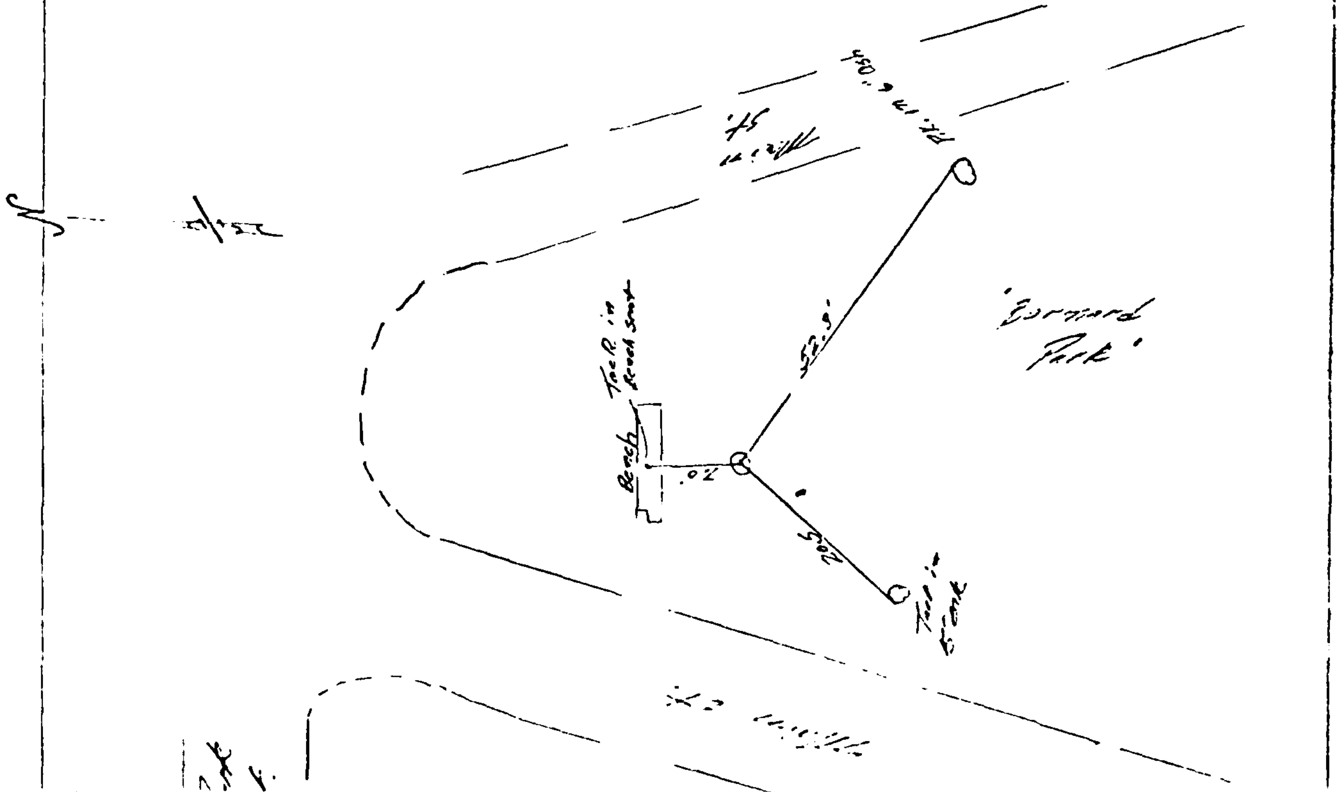
12/18/77 DATE

_______ NOTES FIGURED

_______ NOTES CHECKED

2 / 3



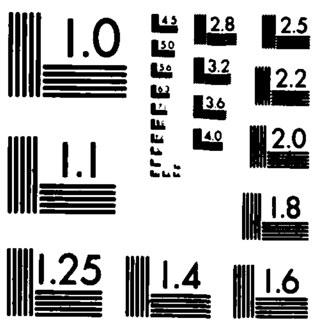


AD-A125 752 PARK RIVER LOCAL PROTECTION AUXILIARY CONDUIT TUNNEL AS-BUILT FOUNDATION..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV DEC 82 4/4

UNCLASSIFIED F/G 13/2 NL

END
DATE FILMED
4 83
DTIC

M-2



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Roger J. Au & Son, Inc.
Cleveland Sewer & Water Construction Co., Inc
P.O. Box 1488
Mansfield, Ohio 44901

SUBJECT Reference & location
B.M. set near outlet
shaft.
Hartford Conn.

PARTY
C.M. IN CHARGE
R.t. INSTRUMENT
D.t. ROD
_____ ROD

12/13/77 DATE

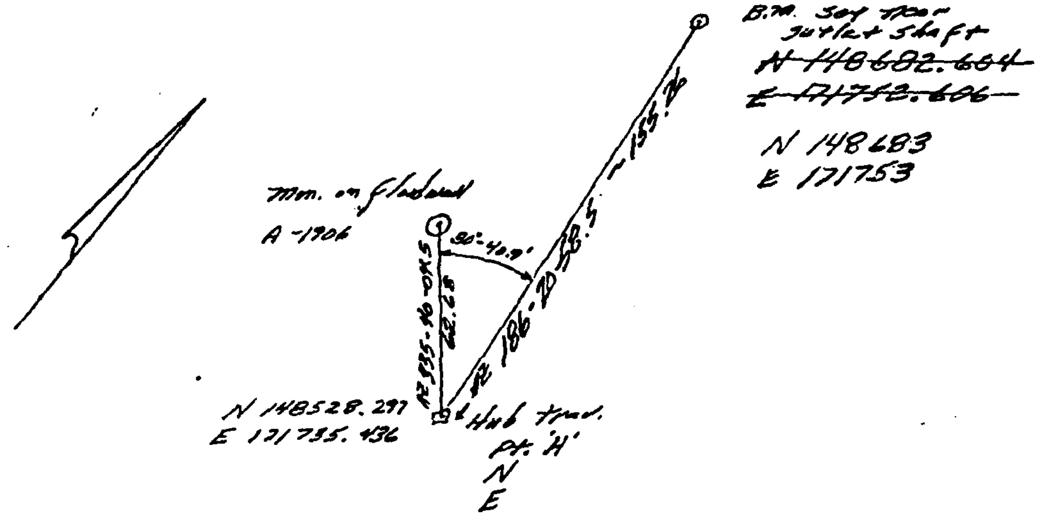
_____ NOTES FIGURED
_____ NOTES CHECKED

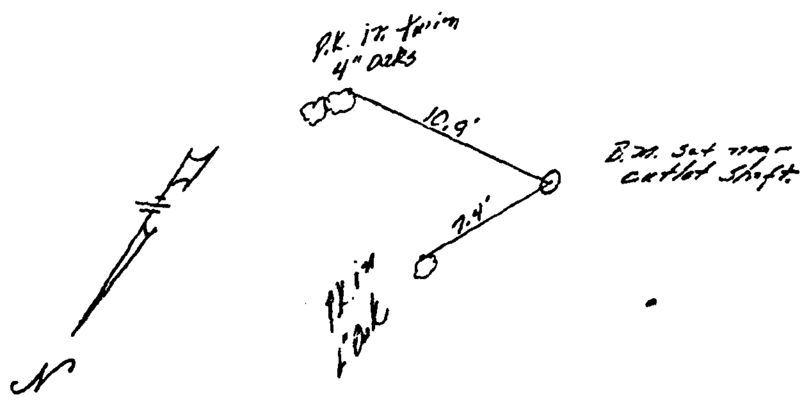
3
3

6.29
4.71
138
10°-41'
10°-41'

305' Rt. Sta
10+06.5

138.261





Roger J. Au & Son, Inc.
Richlands Sewer & Water Construction Co. Inc
P.O. Box 1488
Mansfield, Ohio 44901

SUBJECT: ELEVATIONS OF PERMANENT BENCH MARKS

C.M. IN CHARGE
R.T. INSTRUMENT
D.T. ROD
ROD

12/14/77

NOTES F.B. REF.
NOTES CHECKED

STATION	+	HI	-	ELEVATION	DESCRIPTION
T.B.M. #4	4.088	23.446		19.358	TOP OF NUT ON SOUTH ANCHOR BOLT @ N.W. END OF CONC [illegible], N.W. OF OUTLET CHAMBER
PERMANENT B.M.	8.000	23.149	8.297	15.149	PERMANENT B.M. SET TO [illegible] OF [illegible], STA. [illegible], [illegible] RT. (TO BLEBROOK)
T.B.M. #4			3.792	19.357	(19.358)
T.B.M. #12	3.920	68.537		64.617	N.E. BOLT F. HYD @ MAIN AND [illegible] @ N. END OF BERNARD PARK
PERMANENT B.M.	[illegible]	65.215	5.043	63.494	PERMANENT B.M. SET TO [illegible] OF [illegible], STA. 44+29.8, [illegible] LT.
T.B.M. #12			3.601	64.614	([illegible])
T.B.M. #25	2.577	[illegible]		80.132	N.W. COR. BASE LIGHT POLE # [illegible] @ S.E. COR. PARK ST. & PARKTERRACE
PERMANENT B.M.	[illegible]	82.406	[illegible]	[illegible]	PERMANENT B.M. SET TO [illegible] OF [illegible], STA. 75+31.6, 59.8' LT.
[illegible]			[illegible]	80.13[illegible]	([illegible])