ENGINEERING MONOGRAPHS No. 24

United States Department of the Interior
BUREAU OF RECLAMATION

THE STRUCTURAL BEHAVIOR OF HUNGRY HORSE DAM

by Joe T. Richardson

Denver, Colorado
April 1959 85 cents

United States Department of the Interior

FRED A. SEATON, SECRETARY

Bureau of Reclamation

W. A. DEXHEIMER, Commissioner
GRANT BLOODGOOD, Assistant Commissioner and Chief Engineer

Engineering Monographs

No. 24

THE STRUCTURAL BEHAVIOR OF HUNGRY HORSE DAM

by Joe T. Richardson
Engineer, Dams Branch
Division of Design
Commissioner's Office, Denver, Colorado

Technical Information Branch
Denver Federal Center
Denver, Colorado

ENGINEERING MONOGRAPHS are published in limited editions for the technical staff of the Bureau of Reclamation and interested technical circles in government and private agencies. Their purpose is to record developments, innovations, and progress in the engineering and scientific techniques and practices that are employed in the planning, design, construction, and operation of Reclamation structures and equipment. Copies may be obtained from the Bureau of Reclamation, Denver Federal Center, Denver, Colorado, and Washington, D. C.

CONTENTS

LIST OF FIGURES

INTRODUCTION

During the past 25 years, the Bureau of Reclamation has studied the structural behavior of its major concrete dams. The performance of dams under operating conditions has been measured by instruments embedded in the mass concrete of the structures. The measurements had their beginning at Hoover Dam where embedded instruments were installed in the mass concrete during the period 1933-34 while the dam was under construction. Since that time instruments have been installed at Grand Coulee, Shasta, Canyon Ferry, Hungry Horse, and Monticello Dams. Similar installations are to be made in new high dams under construction by the Bureau of Reclamation.

The measurements and analyses of the structural behavior of these major dams were conducted to obtain a better understanding of the stress variation within the mass concrete and to determine the effects contributed to stress by progressive construction, loading of the dam during reservoir filling, contraction joint grouting, and temperature variation. This monograph presents the results of such a stress study undertaken for Hungry Horse Dam. The study analyzes stress and its development near the foundation of the dam, in several cantilever elements, and at several locations within one complete arch element. The results of this study are shown graphically in the accompanying drawings which illustrate normal vertical stresses, normal horizontal arch stresses, shear stresses on transverse planes, and the principal stresses in the planes of the cantilevers and in the planes of the arches.

This analysis indicates that compressive stresses are present at all times, except for slight tensile stresses that developed during the early construction of the dam. At no time does unit stress at points of observation approach the allowable design stress. The effect of the longitudinal contraction joint is reflected in the distribution of stress in both vertical and horizontal planes, and indicates that portions of the blocks divided by the joint have their own stress distribution.

The structural behavior measurements in Hungry Horse Dam thus have twofold importance. Of primary importance is information by which the actual structural safety of the structure can be gaged. The results obtained also serve as criteria [1]*, which when integrated with similar data from other arch dams [2],[3],[4], aid designers by increasing their understanding of in-service stress conditions of existing structures. This should lead to improved and more economical designs of future arch dams.

*Numbers refer to publications in List of References.

FIGURE 1 - Photograph of Hungry Horse Dam and Powerplant.

GENERAL

Hungry Horse Dam

Hungry Horse Dam is the principal feature of the Bureau of Reclamation's Hungry Horse Project. The dam, shown in Figure 1, is a variable thickness concrete arch structure on the South Fork of the Flathead River, about 9 miles southeast of Columbia Falls, Montana. Ranking with the highest and largest dams in the United States, the dam has a height of 564 feet above foundation, is 2,115 feet long at the crest, has a base thickness of 330 feet on the plane of centers, and contains 2,935,000 cubic yards of concrete.

The dam foundation is Pre-Cambrian blue-grey limestone containing appreciable quantities of clay, silica, and magnesia. Bedding planes of the rock are regular and range in thickness from a few inches to several feet. All planes dip in the same direction, following closely the left abutment of the dam. In the bottom and on the right abutment the foundation line cuts the planes.

Several vertical faults on the left abutment and in the river channel were effectively sealed by cutoff shafts at the upstream and downstream toes of the dam. The foundation was grouted to prevent seepage of water through seams and cracks in the foundation rock. Low-pressure shallow grouting over most of the foundation was performed prior to the placement of any concrete in the dam. Final high-pressure grouting of the foundation to depths of from 50 to 200 feet followed the placing of sufficient concrete to prevent any displacement of the foundation rock.

The dam has a vertical upstream face and a downstream face which slopes 0.6:1 on the plane of centers. The downstream face slopes become flatter toward the abutments.

The spillway for the dam consists of a glory-hole shaft and tunnel near the right abutment. Water flowing over the spillway drops about 480 feet and discharges through an outlet structure 550 feet downstream from the dam. The spillway discharge tunnel is concrete lined for its entire length and has a capacity of 50,000 cubic feet per second.

Hungry Horse Powerplant at the toe of the dam houses four 75,000-kilowatt generators, each unit driven by a 105,000-horsepower turbine.

Concrete Placement

Hungry Horse Dam was constructed in four annual stages, concrete placement being discontinued during the cold weather months from about November until about March of each year. The first concrete was placed from September to November 1949; 60,000 cubic yards were placed during this 3-month period. Construction resumed in March 1950 and 909,000 cubic yards of concrete were placed by November. In 1951, 1,330,000 cubic yards were placed between April and November. During 1952, the final year of construction, concrete placing began on April 7 and the final bucket of concrete was placed in the dam on October 4. The volume of concrete placed during the final year was 636,000 cubic yards. The construction progress for the dam and the water surface elevation record for the reservoir are shown in Figure 2.

The interior mass concrete of Hungry Horse Dam averaged approximately 192 pounds of portland cement, 83 pounds of pozzolan (fly ash), 158 pounds of water, 847 pounds of sand, and 2,802 pounds of coarse aggregate per cubic yard. The exterior, or face, concrete averaged approximately 288 pounds of portland cement, 86 pounds of fly ash, 174 pounds of water, 818 pounds of sand, and 2,724 pounds of coarse aggregate per cubic yard. The interior and face concrete contained 3.7 percent and 3.3 percent of entrained air, respectively.

The average grading of the coarse aggregate for the mass concrete was as follows:

Size	Percent retained on screen	
	Interior concrete	Face concrete
6 inch to 3 inch	22	21
3 inch to 1-1/2 inch	27	27
1-1/2 inch to 3/4 inch	25	24
3/4 inch to No. 4	26	28

The interior and face concrete both averaged 23 percent sand by weight of the total aggregate.

Water-cement plus pozzolan ratios averaged 0.58 for the interior mix and 0.46 for the face mix. Average yearly strengths of interior concrete test cylinders showed a 28-day strength of between 2,580 and 2,830 pounds per square inch, and a 4-year average strength of 2,740 pounds per square inch. The average 28-day strength for the face concrete for the 4-year construction period was 3,890 pounds per square inch. Tests of 1-year cylinders for the first 3 years of the 4-year period showed strengths of 5,270 pounds per square inch for the interior concrete and 6,080 pounds per square inch for the face concrete. Laboratory tests on 8-year-old concrete cores removed from the interior concrete indicated a compressive strength of 5,460 pounds per square inch.

Temperature Control of Concrete

The dissipation of heat generated by the setting of the cement in the concrete of the dam was controlled by an artificial cooling system of embedded pipes through which river water was circulated. The concrete was cooled to the desired minimum temperature of 38° F, at which temperature the contraction joints of the dam were opened to their maximum width. The cooling system was used during the early age of the concrete to control cracking by lowering the temperature rise of the concrete. This expedient, in turn, permitted the wider spacing of contraction joints than would otherwise have been permissible.

The dam was divided by a series of vertical radial joints into blocks 80 feet wide at the upstream face. The joints were continuous from the upstream to the downstream face of the dam. As the base thickness of the dam is 330 feet at the maximum section, a single longitudinal joint was provided in the design for the larger blocks to prevent development of continuous wide cracks across the blocks. The longitudinal joint was offset in adjacent blocks and so placed that the longest block would be 186 feet in length. The largest blocks were thus 80 by 186 feet; they contained 2,700 cubic yards of concrete in each 5-foot lift.

The water used in cooling the concrete was circulated through 1-inch-diameter thin-wall steel tubing spaced 3 to 5-1/2 feet horizontally and placed on the foundation rock and top of each 5-foot lift of concrete. The total length of tubing embedded in the dam is 4,435,140 feet.

Grouting

To assure monolithic action of the dam structure and provide assurance of arch action, the contraction joints were filled

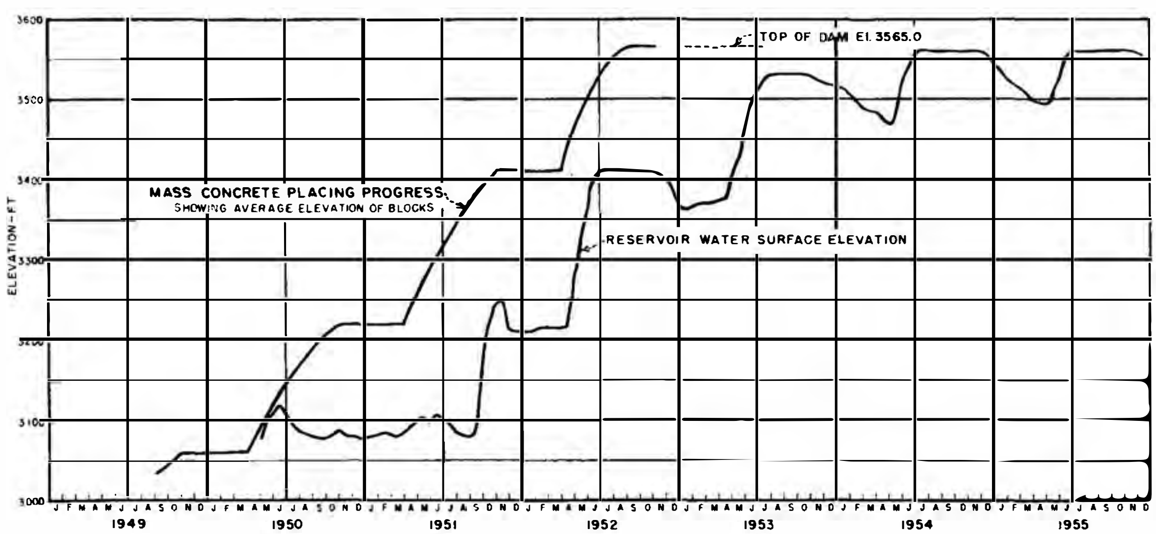


FIGURE 2 - Construction progress and reservoir surface elevation record.

with cement grout under pressure through a system of grout pipes after the concrete was cooled to the predetermined temperature of 38° F. Grouting of joints followed progressive construction and concrete cooling, the joints being grouted as soon as possible after initial cooling had been completed.

INVESTIGATION PROGRAM

Instrument Layouts in Dam

Six types of instruments, totaling 681 units, were embedded in the concrete of Hungry Horse Dam to measure the behavior of the structure. These instruments comprise 508 strain meters, 12 stress meters, 22 reinforcement meters, 80 joint meters, 6 hydrostatic pressure gages, and 53 resistance thermometers.

Most of these instruments, which were strain meters installed in groups, were provided to determine the stress distribution on gage lines at seven sections in the dam, as shown in Figure 3. The locations selected for gage lines of strain meter groups are representative of areas of maximum stress. Five of the gage lines are in the plane of one arch and thus provide for investigation of representative stress distributions in this arch. The additional two locations provide gage lines at the bases of cantilevers, the gage lines of the cantilever bases being directly beneath gage lines in the arch. Included with each gage line of strain meters is a pair of strain meters for the determination of volume change within the dam concrete. These pairs of "no-stress" strain meters were placed near the top of the lift containing strain meter groups. A 40-inch-diameter metal cover plate supported a few inches above the free concrete surface covers the meter pairs. Concrete placed over these installations is not in contact with the free surface of concrete beneath the cover, and thus there result these stress-free areas in the mass concrete of the structure. By this expedient, the vertical strain meter measures only changes in volume due to temperature variation, moisture variation, and autogenous growth. The horizontal strain meter measures changes resulting from horizontal stress that could contribute, through Poisson's ratio effect, error to the vertical strain meter measurements.

Near the bases of the cantilevers where the gage lines of strain meter groups cross the longitudinal contraction joint, strain meter groups were provided on either side of the joint to determine stress conditions near the edge of each portion of the block that resulted from the effects of the joint [5]. At three locations near the faces of the dam strain meter groups were provided to study the variation of stress near these free surfaces.

Two stress meters were included with each of the six groups of strain meters forming the gage line at the base of the maximum section. These meters provided a check of stress in the vertical direction.

Strain meters were installed at two elevations on the shell of one of the four penstocks and reinforcement meters were installed at companion locations in the reinforcing steel that surrounds the penstock. The strain meters were located at four points about the circumference of the penstock, and the reinforcement meters were similarly located in the surrounding rings of reinforcing steel. Other reinforcement meters were installed in reinforcement steel above and below a gallery and in the vertical reinforcement steel of a trashrack structure. These installations provided determinations of stress variation in each particular construction detail.

Joint meters were installed on both radial and longitudinal joints bounding or crossing blocks containing gage lines of strain meter groups and were used to measure the beginning and extent of joint opening. On the longitudinal contraction joint the meters were located at several elevations to measure joint opening as the dam was loaded.

Hydrostatic pressure gages were located on a radial line at a selected location near the base of the dam at varying distances from the upstream face to trace the development of pore pressure within the concrete of the dam.

Resistance thermometers were installed on a 50- by 50-foot grid pattern in a maximum section of the dam. These instruments determine the distribution of temperature. The determination of temperature is of great importance, as volume

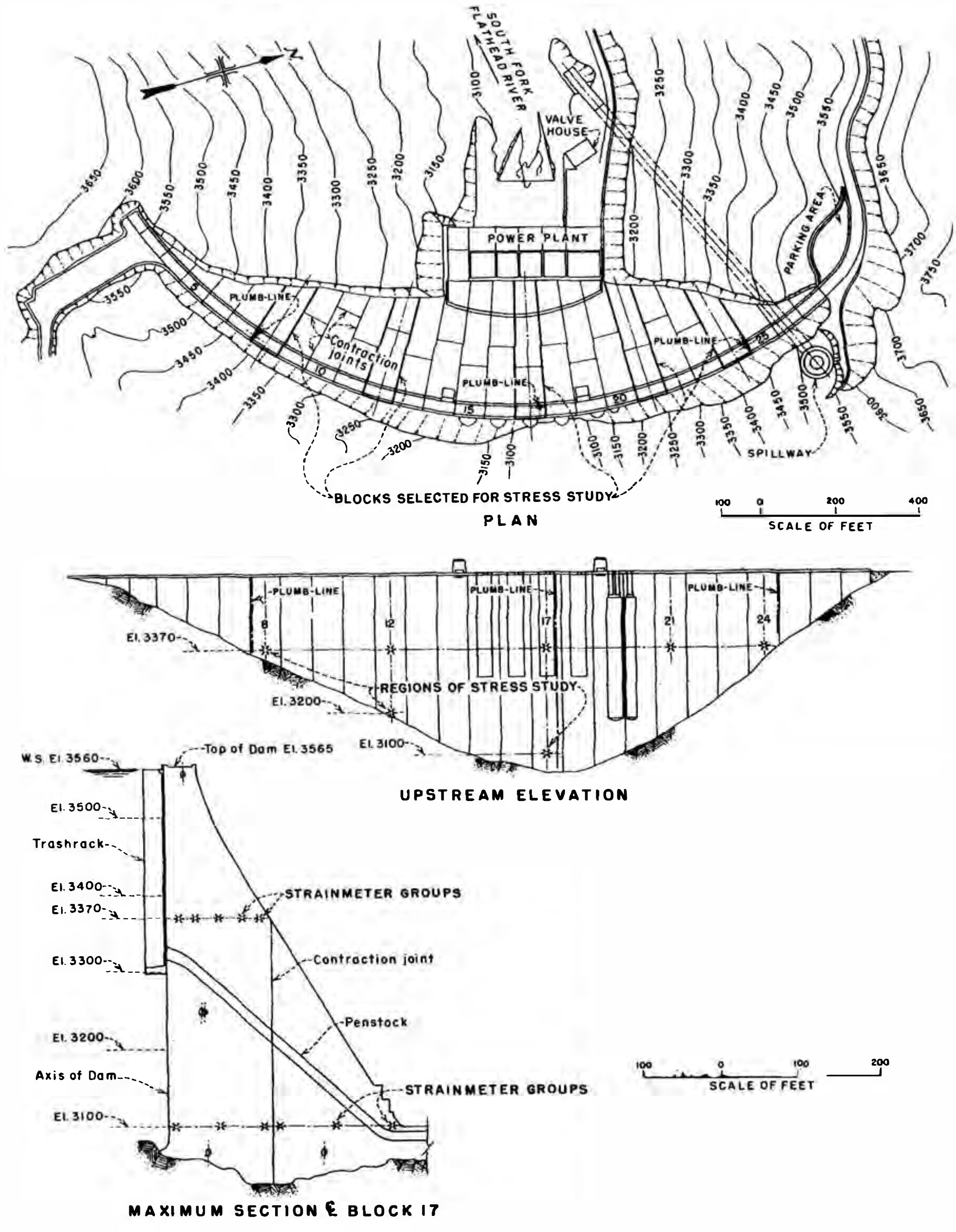


FIGURE 3 - Locations of strain meter groups.

change of the concrete caused by temperature change is one of the major factors contributing to stress in a concrete arch dam.

Other Systems of Measurement

In addition to the measurements made by embedded instruments, plumb lines were installed in specially provided wells at three locations in the dam. Near the center of the dam, radial and tangential deflections were measured with respect to the top of the dam at four elevations. At the quarter points of the dam, radial and tangential deflections were measured only at the lower ends of the plumb lines. Figure 4 shows a plumb-line observation station, the micrometer carriage, and microscope apparatus being used to obtain the plumb-line position with respect to a fixed reference mark.

A system of piping consisting of five series of individual lines of piping between 35 points located on the foundation and terminating in the galleries of the dam provides for observing the under-pressures at the contact between the foundation and the mass concrete of the dam. Each of the five series of piping provides for six or seven points of uplift pressure measurement at uniformly spaced intervals on radial lines through sections of the dam.

A seismograph station located near the dam site gages the seismic activity of the area and determines if any such activity might be originating on the Swan or Flathead faults between which the dam is located. The station was installed prior to the dam construction to determine whether the mass of the dam and impounded reservoir might later contribute to possible seismic disturbances in the area. To date, seismic activity in the area has been limited to shocks

FIGURE 4 - Plumb line observation station showing micrometer carriage and microscope.

FIGURE 5 - Strain meter group at base of maximum section.

of minor nature occurring at points of considerable distance from the dam.

Strain Meter Groups

The groups of strain meters were embedded in the dam in two types of arrays. Near the base of the maximum cantilever section, the gage line of strain meter groups is about 50 feet above the foundation rock. The installation consists of 6 groups of 10 meters each. The strain meters in each group are oriented in five directions. Each direction is provided with duplicate meters. In addition, each group includes two stress meters placed to measure stress in the vertical direction. At the base of the dam, the maximum cantilever is divided by the longitudinal joint into two parts about equal in length. Three strain meter groups were placed in each part. These groups are at the center of the blocks and 5 feet from the upstream and downstream edges of the blocks. In each of these groups the strain meters were placed individually in fresh concrete as shown in Figure 5. The strain meters were alined using a template, level, and protractor, the groups being located from surveyed reference points. A companion pair of stress meters is shown in Figure 6.

All strain meter groups on the gage lines in the other 6 sections of the dam comprised 12 meters each. In these groups, 11 meters were assembled to a spider or holding fixture as shown in Figure 7. The whole array was placed on a leveled base of fresh concrete, oriented for direction, and covered with concrete. The twelfth meter was embedded vertically at the side of the cluster. Figure 8 shows a pair of "no-stress" strain meters being installed in the concrete, and Figure 9 shows the steel cover plate placed over the pair of strain meters to form the stress-free surface. Readings from all instruments began immediately preceding embedment and continued on a time scale of four readings the first day, two on the second, and daily for more than two weeks; readings were then taken on a gradually increasing interval to one week which continued during the construction period. Following completion of construction, readings were reduced to a semimonthly basis and finally to a monthly basis, on which schedule the readings are continuing.

Joint Meters on Longitudinal Joint

The joint meters installed at several elevations on the longitudinal joint in the maximum section of the dam were provided to measure the joint opening between the upstream and downstream portions of blocks in the central part of the dam as loading on the dam increased. These joint meters also serve to detect the effectiveness of the grouting of this joint. Figure 10 shows a typical joint meter installation. Two vertical rows of joint meters are installed on the staggered

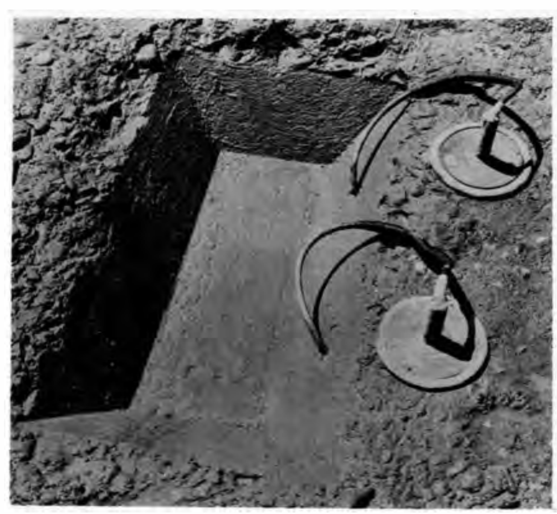


FIGURE 6 - Stress meters prior to embedment.

longitudinal joint in blocks that adjoin each other. Block 17, which contains major strain meter groups, contains one vertical row of joint meters, and Block 18, immediately to the right, contains the other vertical row. In Block 17, joint meters are located at elevations 3050, 3065, 3100, 3150, 3250, and 3350 near the top of the joint. In Block 18, joint meters were installed at identical elevations with the exception of the top and bottom meters. The bottom meter was placed at elevation 3055, and the top meter of this row was placed at elevation 3250 near the top of the joint. The reading schedule for all joint meters is the same as that for strain meters.

Creep Properties of Concrete

Simultaneously with the embedment of instruments in the dam, a series of fifteen 6-inch by 12-inch-long cylinders of identical concrete, each containing an embedded strain meter, was fabricated in the concrete laboratory of the Bureau's engineering laboratories in Denver. These cylinders were loaded at various ages. The purpose of these tests [6] was to determine separately the creep function for the dam concrete. This information is required in the method of determining the stresses in the dam. The cylinders were completely encased in neoprene rubber jackets to avoid loss of moisture, and thus, as nearly as practicable, approximate conditions of curing similar to those of the curing of the mass concrete in the dam.

From the laboratory tests a logarithmic function was found that would completely define creep in the dam concrete. This function is expressed as:

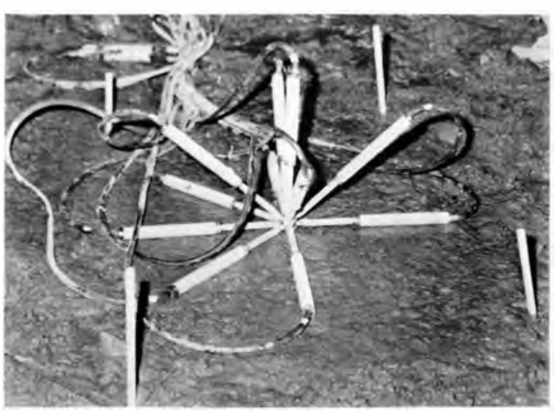


FIGURE 7 - Strain meter group showing instruments assembled to holding spider.

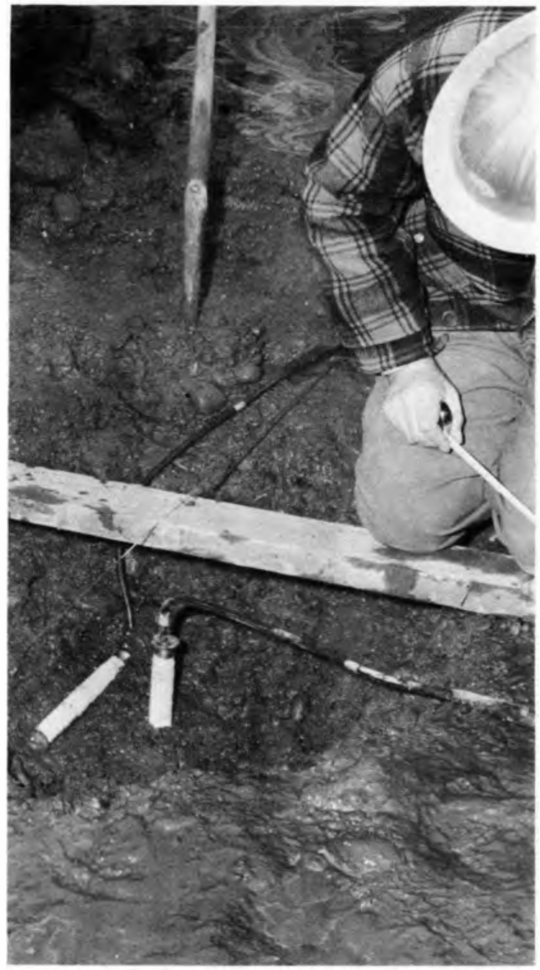


FIGURE 8 - Pair of strain meters for determining volume change in mass concrete.

$$\epsilon = \frac{1}{E'} + F(K) \ln (t + 1)$$

where

- ϵ is elastic strain + creep strain, in inches per inch per pound per square inch
- E' is the instantaneous modulus of elasticity, varying with age, in pounds per square inch
- $F(K)$ is creep function, a constant in inches per inch per pound per square inch for each particular age of loading
- $\ln (t + 1)$ is the natural logarithm of the time in days after loading plus 1 day

FIGURE 9 - Steel cover plate placed over pair of embedded strain meters.

The following table shows the parameters of the creep function for the various ages of cylinder loading:

Loading age in days	$\frac{1}{E'}$	F(K)
2	0.360	0.0550
7	0.240	0.0478
28	0.220	0.0275
105	0.194	0.0201
365	0.175	0.0174
5 years	-	-

A coefficient of thermal expansion of 6.00 millionths of an inch per inch per degree F was determined by separate tests. Other tests determined Poisson's ratio for this concrete to be 0.17.

Stress Computations

The records from the embedded strain meters extend over a period beginning immediately prior to embedment and continue through to date. However, the results of this study were terminated at the end of 1953, the cutoff date selected for the analysis. At some future date extension of the results is anticipated to include an additional period of 5 to 7 years of in-service operation of the structure. During the period of study, temperature and concrete properties have varied considerably. Successive corrections were required for temperature effect in the concrete and for the Poisson's ratio effect. Dilatation corrections were required to obtain elastic compatibility in all combinations of three mutually perpendicular strain meters in the strain meter groups. These corrections are described elsewhere.[7], [8]

The designers of Hungry Horse Dam had expressed the desire to have available at progressive intervals during construction the distribution of stress at the base of the maximum cantilever as concrete placing operations progressed and as the reservoir water load increased on the dam during reservoir filling. The designers also requested to have available the complete stress history for cantilevers and the complete arch at about the time reservoir water surface had reached normal operation elevation. However, computations of stress from strain using the method of superposition and the creep of concrete [9] as performed using hand methods in earlier completed studies [8] were tedious and time consuming. Results were not available until several years after completion of a dam. Thus, several successive improvements over earlier procedures were required in computation methods to accelerate production of stress results.

FIGURE 10 - Typical joint meter installation.

A method was devised by which the computations of stress were considerably facilitated [10], using punched cards and high-speed electronic digital computers. Initially, the machine process reproduced mechanically the routine formerly performed by hand but at a more rapid rate. This improvement over hand processing was found to require more time than desired and a further refinement of the method [11] was subsequently made.

The initial computation may be represented as:

$$\Delta\sigma = \frac{\Delta\epsilon}{\frac{1}{E'} + F(K) \ln (t + 1)}$$

where

$\Delta\sigma$ is the change in stress during an increment of time in pounds per square inch

$\Delta\epsilon$ is the total measured strain at the end of the increment of time, minus the elastic strain plus the creep strain at the beginning of the increment of time in inches per inch per pound per square inch

$\frac{1}{E'} + F(K) \ln (t + 1)$ is the function of elastic and creep strain obtained from the laboratory tests in inches per inch per pound per square inch

E' is designated as instantaneous modulus of elasticity in pounds per square inch

In this computation, cards were punched for intervals of time for the creep function and for the observed total strain. The machine computers then integrated for the individual strain meters, and for each interval of time, the elastic strain plus the creep strain from all previous increments of stress, subtracted the total from all previous increments of stress, subtracted the total from the observed strain, and computed the next increment of stress. The punched cards were reshuffled and the process repeated for the next increment of stress, and so on to the end of available data.

In the further refined process, a stress computation method was devised using an average logarithmic concept for the creep function that reduced considerably the amount of processing and also provided a readily usable method adaptable to hand computation. Through the latter modification, the summation of total strain could be carried forward in successive computations, making unnecessary the computation of individual increments of creep for each interval of time. Using the advanced method of computation, stress at the base of the maximum cantilever became available within about a day or two after meter records were received from the field, thus providing the designers of the dam information on the stress state as the construction progressed. The method also expedited the processing of the field and laboratory data for all times of stress investigation and made possible the stress results presented in this monograph within a reasonably expected length of time.

INVESTIGATION RESULTS

Stresses, Temperatures, and Deflections

The results of this investigation show the manner in which stress, temperature, and dam deflections are developed progressively with time for the sections of the dam that have been investigated. Several important factors become evident from the study.

The stress distribution curves for sections of cantilevers and sections of arches of Hungry Horse Dam indicate no excessively high compressive stresses at any of the points of stress investigation. Indicated tensile stresses are generally low and are limited to locations near the faces of the structure. The exceptions to low tensile stress occurred during early construction of the dam, when magnitudes greater than 100 pounds per square inch were indicated in the vertical direction near the downstream face of Block 12, elevation 3200, and in the horizontal direction near the upstream face of the arch at elevation 3370, Block 21. These tensile stresses disappeared within a few months after additional construction raised the height of concrete on the sections.

Shear stresses on the transverse planes of arch and cantilever intersections are incicated to be low, the maximum being approximately 100 pounds per square inch and appearing at the base of the downstream portion of Block 17.

Temperatures within the mass of the dam were effectively decreased by cooling, and by the time of the final conditions of stress shown for the study, the temperatures were within limits of approximately 40° to 46° F throughout the dam, with the exception of temperatures at measurement points near the downstream face of the dam. Temperature at these downstream face points fluctuated with the seasonal temperature cycle.

The principal stresses in the planes of the cantilevers show a direction of major principal stress approximately parallel to the faces of the upstream base portions of the blocks divided by the longitudinal contraction joint and tipped in the upstream direction in the base portions of the blocks downstream from the longitudinal contraction joint. In the planes of the arches investigated, the principal stresses show directions of the major stress to be parallel to the line of arch thrust at the upper elevation investigated except near the faces of the structure at the abutments. At the abutments the principal stresses near the face of the sections are rotated toward each other with respect to the crown of the arch. At the lower elevation at the abutment, the principal stresses are oriented approximately normal to the abutment foundation rock. Principal stresses were not determined in the plane of the arch at the base of the maximum cantilever.

The radial deflections of the dam from plumb-line measurements are indicated to be downstream at the center and quarter points of the dam. The tangential deflections at the quarter points indicate very slight deflection toward the plane of centers of the dam, and at the center of the dam the tangential deflection is indicated to be practically zero; only a very slight deflection trend is indicated toward the left.

Figure 11 shows the normal stresses from the strain meters and the average unit stress on the various sections for the final conditions of stress. For this loading, the concrete placement in all parts of the dam had been completed for approximately 9 months; contraction joint grouting was completed to the top elevation of the dam; and the reservoir water surface had reached **approximately 90 percent of normal operating level, being at elevation 3515.0, or 45 feet below the normal operating level. Temperatures at the points of stress measurement are between the approximate limits of 42° to 45° F.**

Details of Stress

The detailed results of the measurement program are shown in the series of

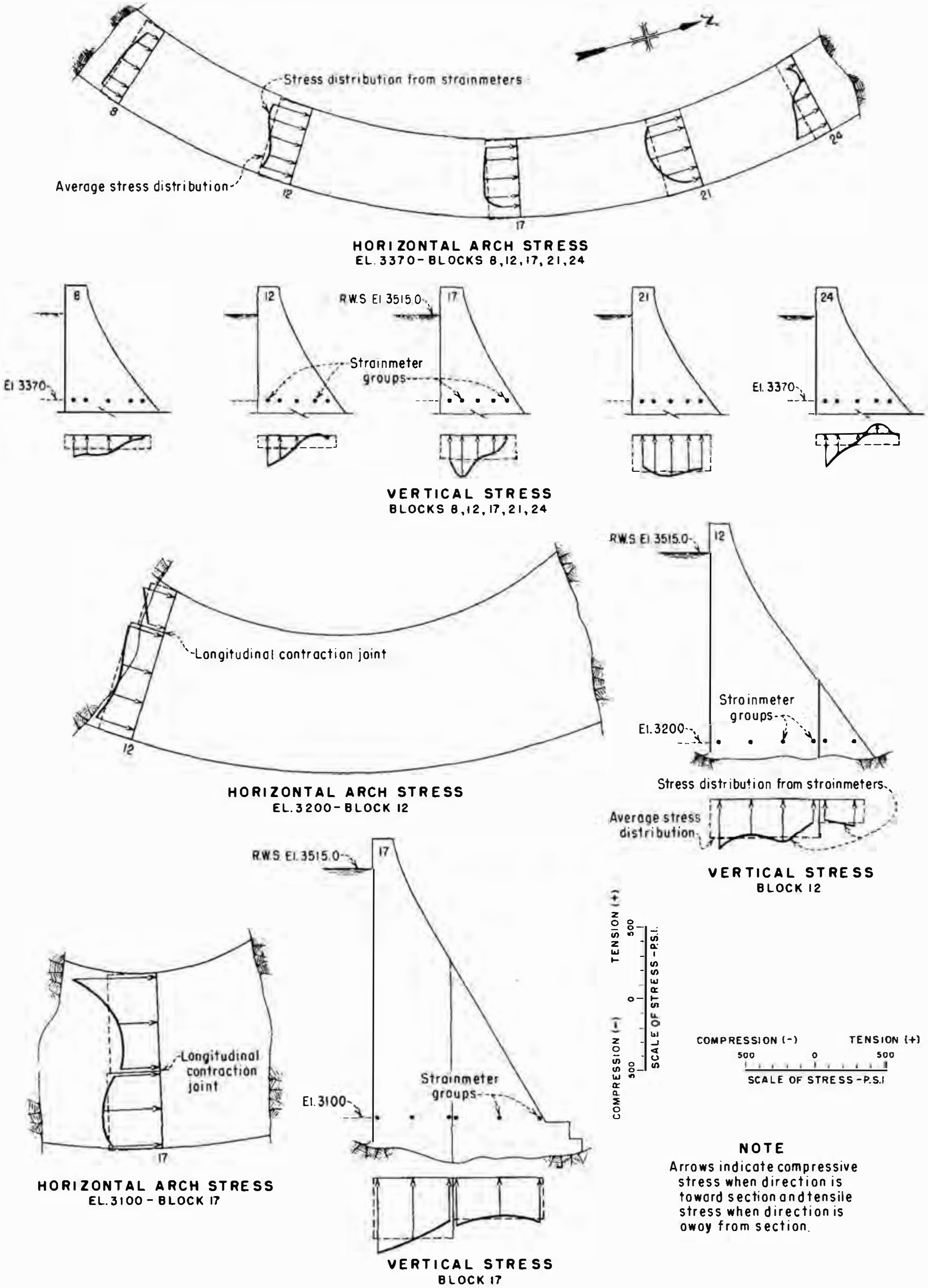


FIGURE 11 - Normal stresses, July 1, 1953--Reservoir water surface, el. 3515.

drawings, Figures 12 through 24. In general, dates have been selected at about 4-month intervals. For each date there are shown the plots of stress distribution and temperature distribution on the sections of the cantilevers and the section of the arches where the measurements were made. The major dead and live loads on the structure are shown on the sections above the stress plots for cantilevers. On these sections are shown the elevation to which concrete had been placed, corresponding reservoir elevation, and conditions of completed grouting. No reservoir water is shown during 1950 and 1951, as the river was held back by a cofferdam at that time and the water had not at that time come in contact with the lower arch elevations.

Cantilevers (Vertical Stresses)

Figures 12 through 16 show the distribution of stress on the gage lines through the cantilevers. Vertical stress, shear stress, and temperature have been plotted for selected dates every 4 months beginning about 3 months after the groups of strain meters were placed, and continue for more than 2 years. After 2 years stresses and temperature are plotted twice each year.

El. 3100, Block 17 (Base of Maximum Cantilever)

Figure 12 shows the vertical stress distribution at the base of the maximum cantilever. The initial stress conditions indicate that each block on either side of the longitudinal joint acted separately and developed its own vertical stress pattern. As time elapsed and with the appearance of the smoother curve for stress, the progressive stress plots indicate the dam to be acting more as a unit than as individual blocks. Examination of the curves of vertical (and horizontal) shear stress in the transverse plane indicates discontinuity induced by the longitudinal joint; this discontinuity became more evident as concrete and water loads increased on the dam. The stress plots indicate that most of the shear stress is resisted in the downstream portion of the block. The included plots of temperature indicate variation through the section and greatest variation at the downstream face.

El. 3200, Block 12 (Base of Quarter Point Cantilever)

Figure 13 shows the vertical stress distribution at the base of the cantilever located approximately at the left quarter point of the dam. The initial and successive stress conditions at the base of this cantilever indicate, as evident in Block 17, that the blocks on either side of the longitudinal joint acted separately and each block developed its own vertical stress pattern. The progressive plots of stress indicate that at this location the dam acts more as two parts rather than as a single unit. The curves of vertical (and horizontal) shear stress indicate slight discontinuity due to the longitudinal joint. As concrete and water load increase on the dam, the stress plots indicate the shear is resisted about equally by both blocks.

The temperature plots indicate greatest variation at the downstream face of the dam. Increasing face temperature shows the stress curves resulting in increased stress at the downstream face.

El. 3370, Blocks 8, 12, 17, 21, and 24

Figures 14, 15, and 16 show the vertical stress distribution, shear stress, and temperature conditions on the several gage lines of strain meters through the sections, all on the same elevation. Near the abutments, Blocks 8 and 24, the gage lines may be considered at the bases of short abutment cantilevers, the remaining gage lines being at approximately the right and left quarter points of the dam, Blocks 12 and 21, and at the crown, Block 17. The elevation of these gage lines is above the upper extremity of the longitudinal joint. Thus, the stress distributions on the sections are not affected by the longitudinal joint, as they are at the lower elevations. Strain meter groups on all five gage lines were placed within a period of approximately 2 months. Comparison of vertical stresses at the five cantilever locations shows a remarkable similarity of stress trend for the same dates, although certain peculiarities in the shapes of the stress curves are present near the downstream face for Blocks 12 and 24.

The general trend of vertical stress at each section shows the increase of stress in the upstream portions of the sections as concrete placement progressively increases in elevation. As reservoir water increases above the elevation of the strain meter groups, the stress at the upstream face stabilizes, indicating the effect of stabilized temperature; at the downstream face stress varies in accordance with fluctuating temperature at the downstream face. Shear resistance is generally indicated to

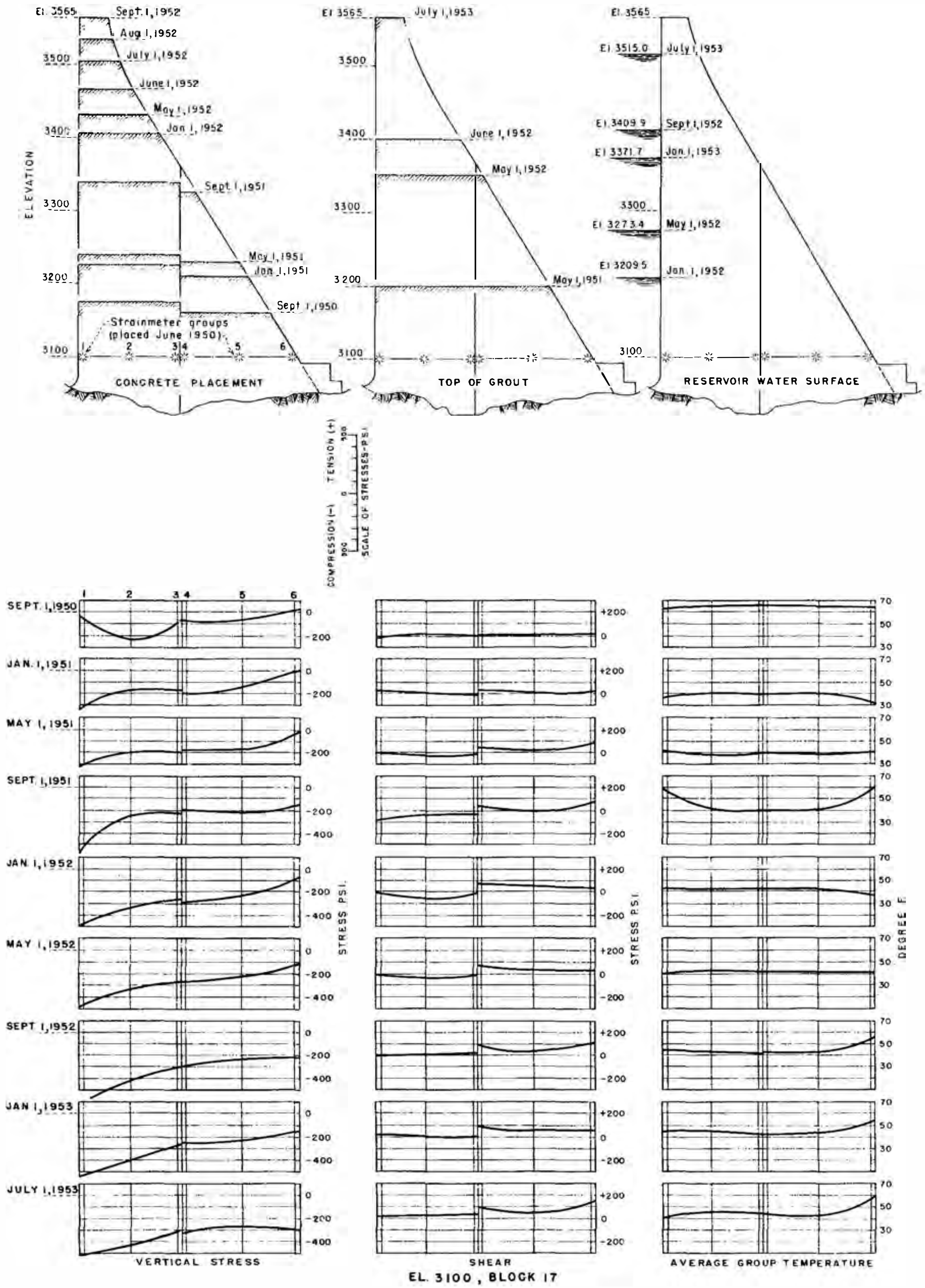


FIGURE 12 - Vertical stresses and temperatures--el. 3100, Block 17.

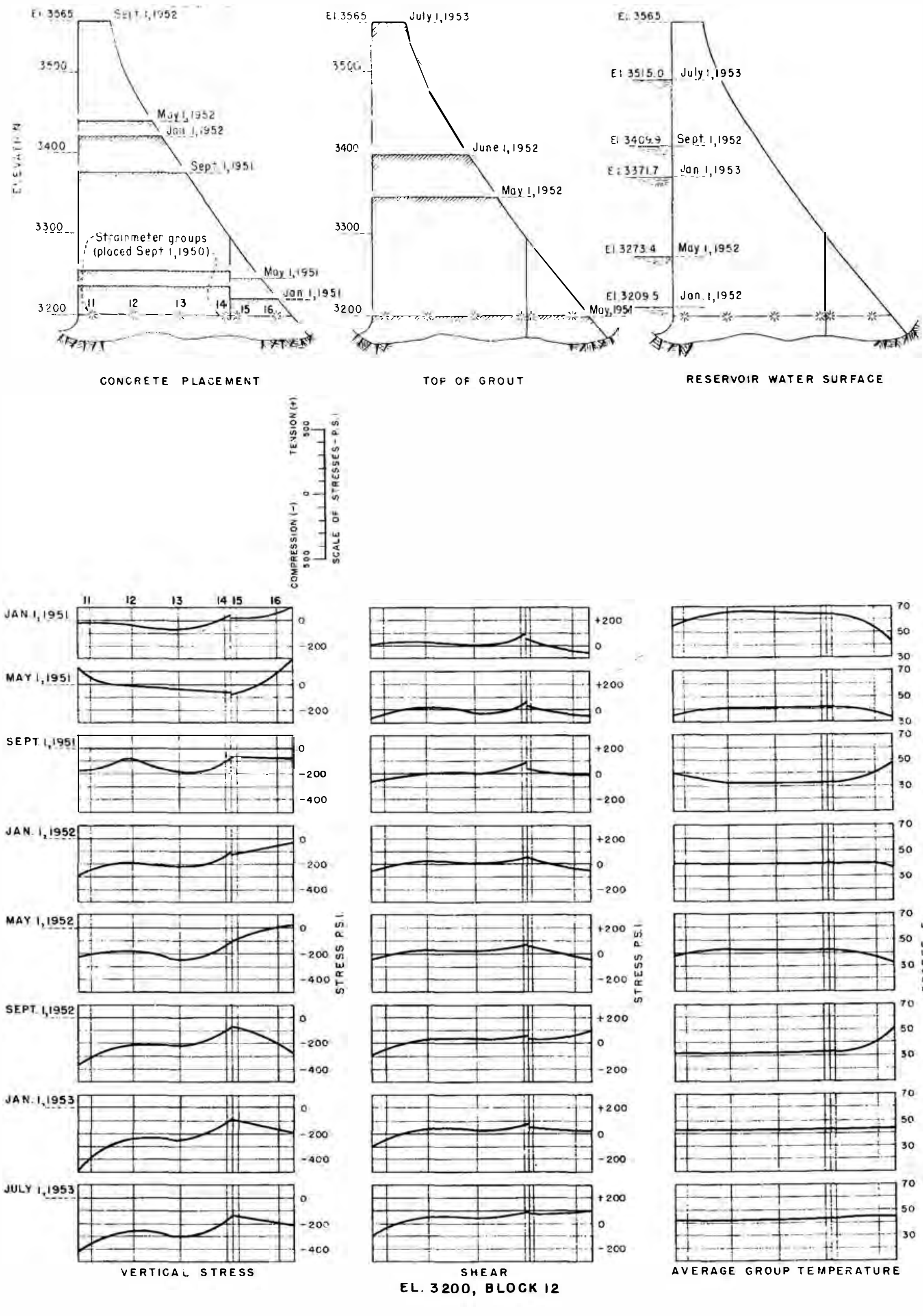


FIGURE 13 - Vertical stresses and temperatures--el. 3200, Block 12.

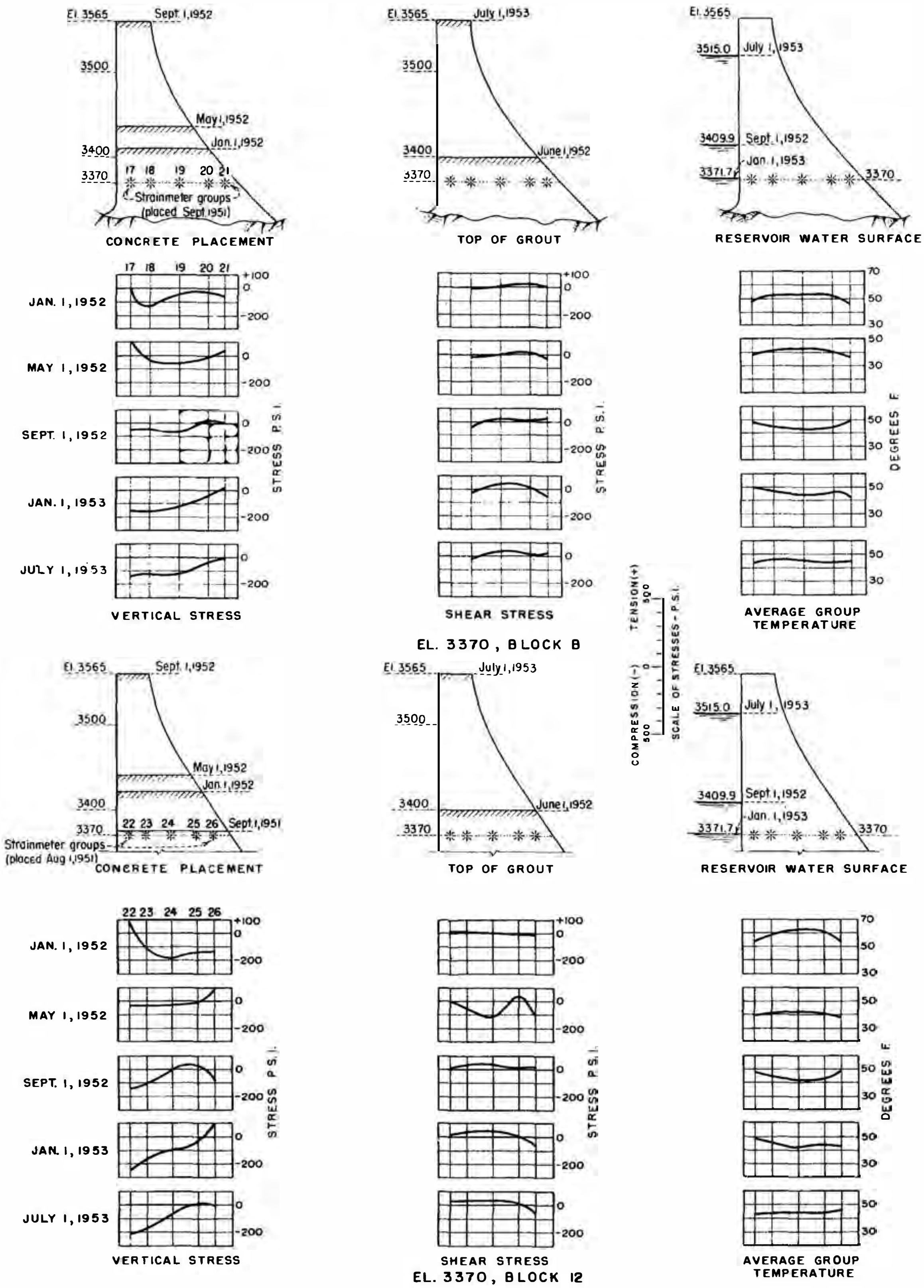


FIGURE 14 - Vertical stresses and temperatures--el. 3370, Blocks 8 and 12.

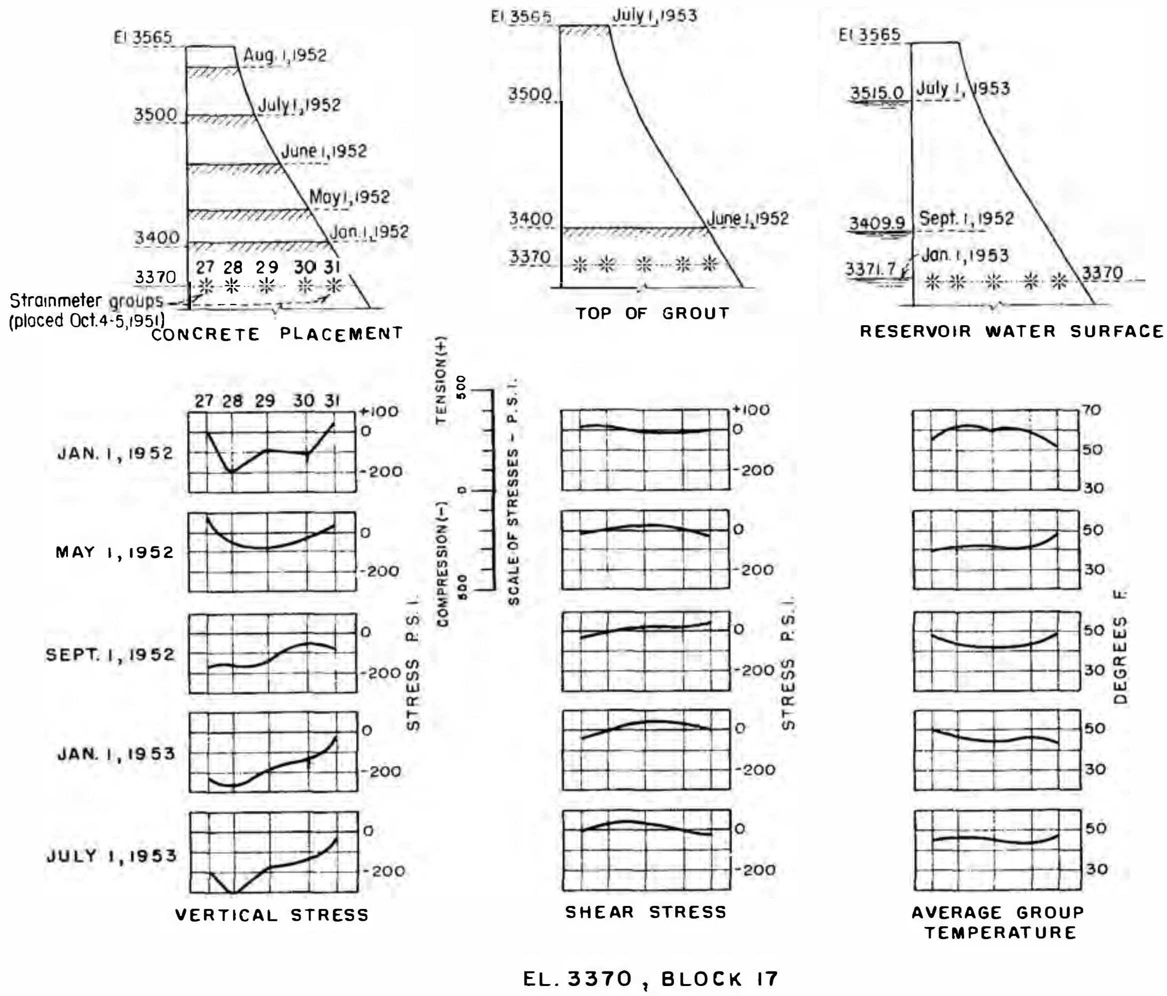


FIGURE 15 - Vertical stresses and temperatures--el. 3370, Block 17.

be greater in the downstream portion of the sections.

Principal Stresses (Plane of Cantilever)

Figures 17, 18, and 19 show the principal stresses and their orientation in the vertical planes. The directions of the major principal stresses are shown to be nearly vertical in the upstream portion of the blocks divided by the longitudinal joint and nearly parallel to the downstream face of the dam in the downstream portion of the blocks. At the higher elevation, above the longitudinal joint, the major principal stresses are oriented nearly parallel to the vertical in the upstream portion of the sections that have the greatest amount of covering concrete. The directions of the stresses are parallel to the face in the downstream portion of the sections that have a sloping face.

It can be seen that the longitudinal joint apparently has considerable effect in disrupting the smooth variation of stress from upstream to downstream face. Marked discrepancies of stress on either side of the joint may be caused by joint grouting, resulting in a "locked-in" stress condition that may remain long after the dam is completed, loaded, and in service.

Arches (Horizontal Stresses)

Figures 20, 21, and 22 show the development of horizontal arch stress at the base of the maximum cantilever, at the left abutment of the arch between the quarter points of the dam, and at five locations in a complete arch. Temperature conditions are shown for each condition of stress distribution.

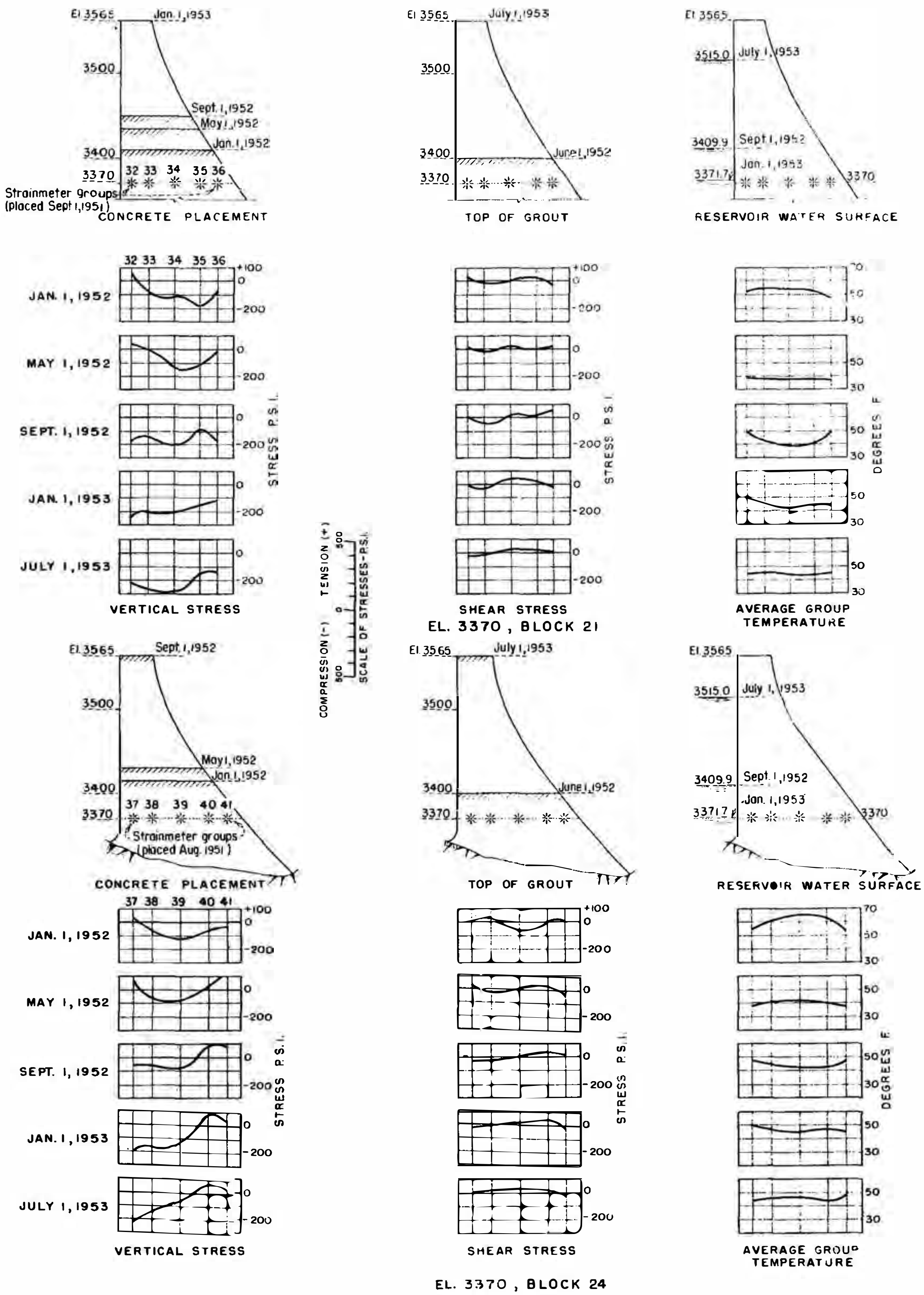


FIGURE 16 - Vertical stresses and temperatures--el. 3370, Blocks 21 and 24.

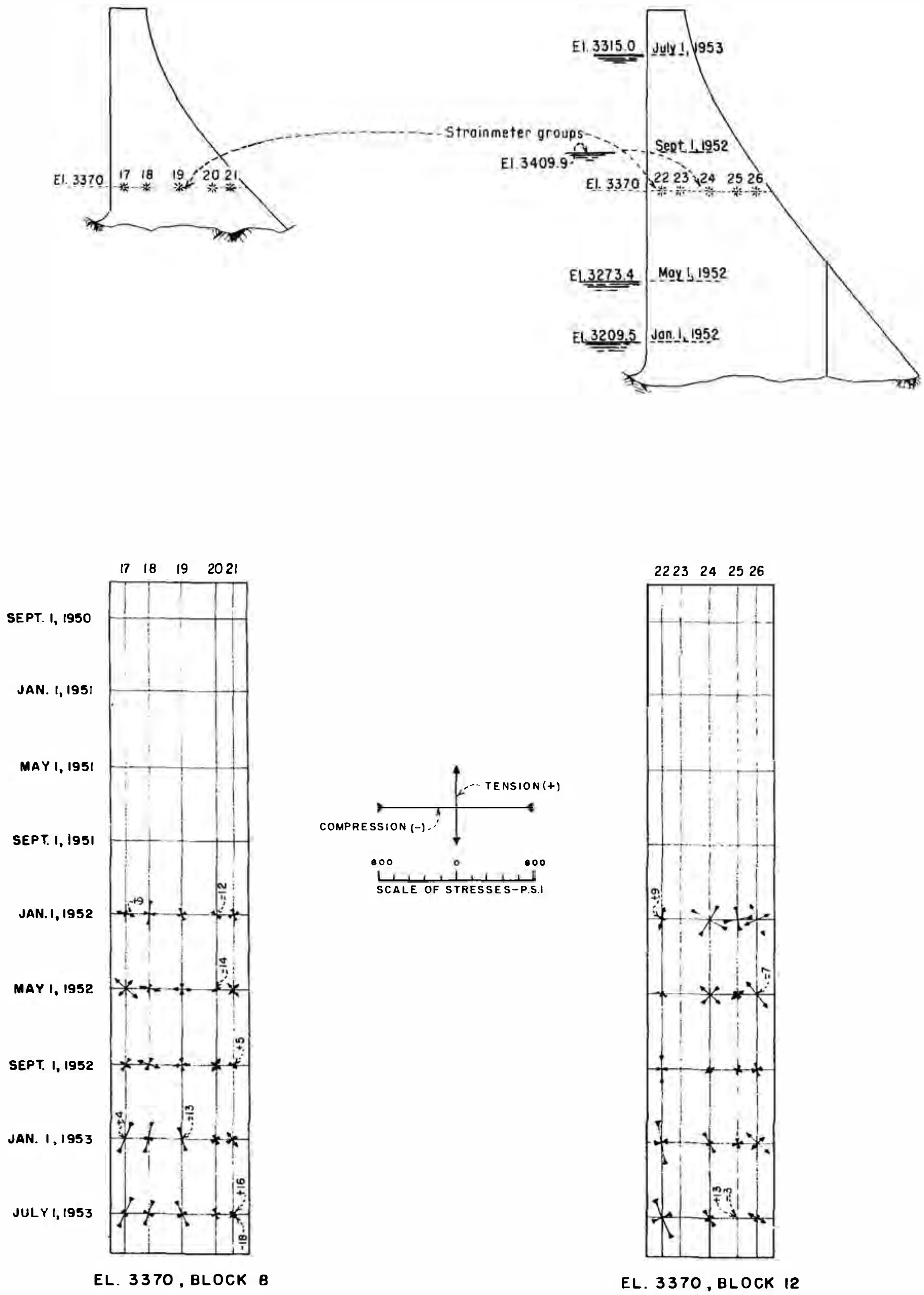


FIGURE 17 - Principal stresses in vertical planes--el. 3370, Blocks 8 and 12.

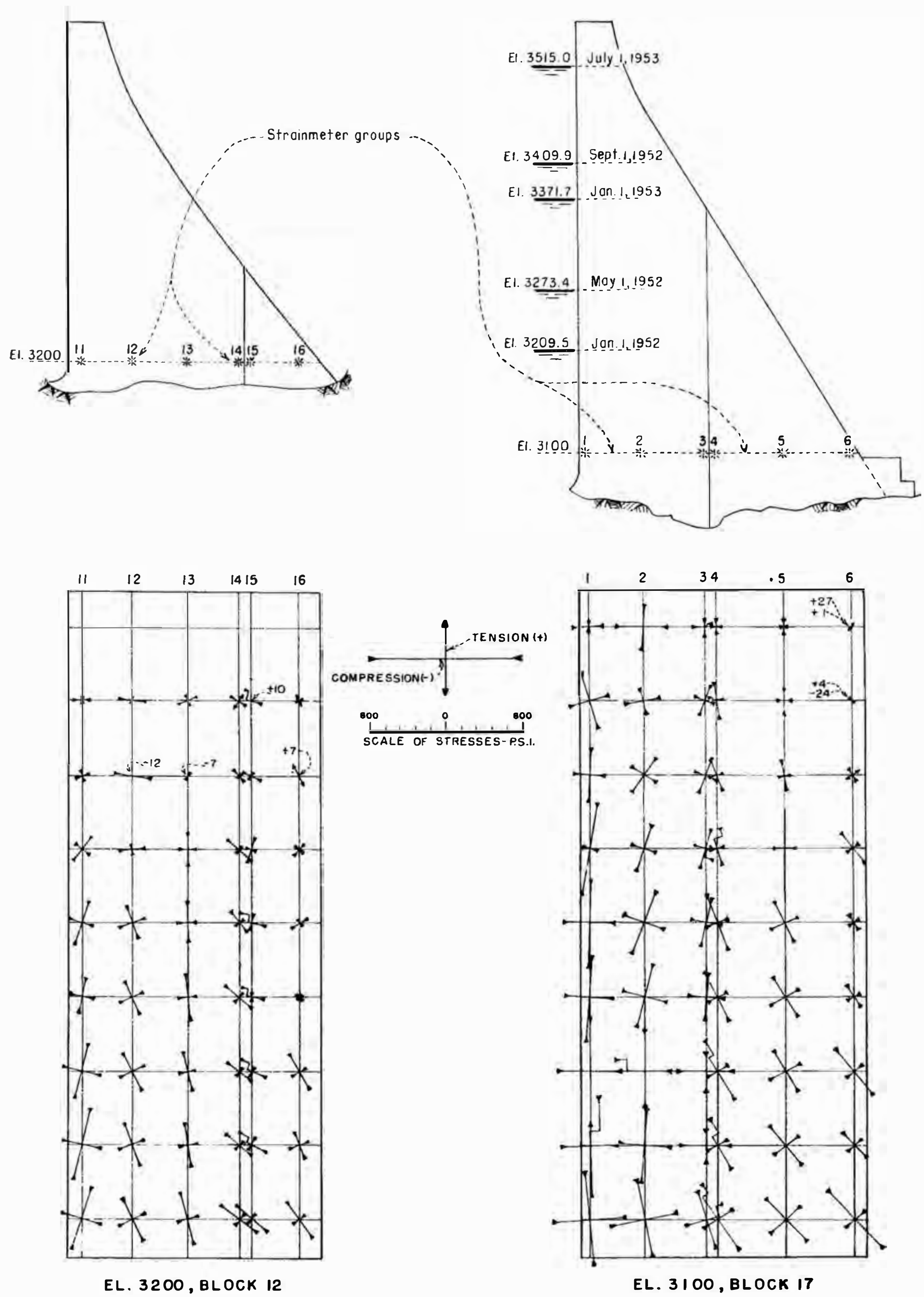


FIGURE 18 - Principal stresses in vertical planes--el. 3200, Block 12, and el. 3100, Block 17.

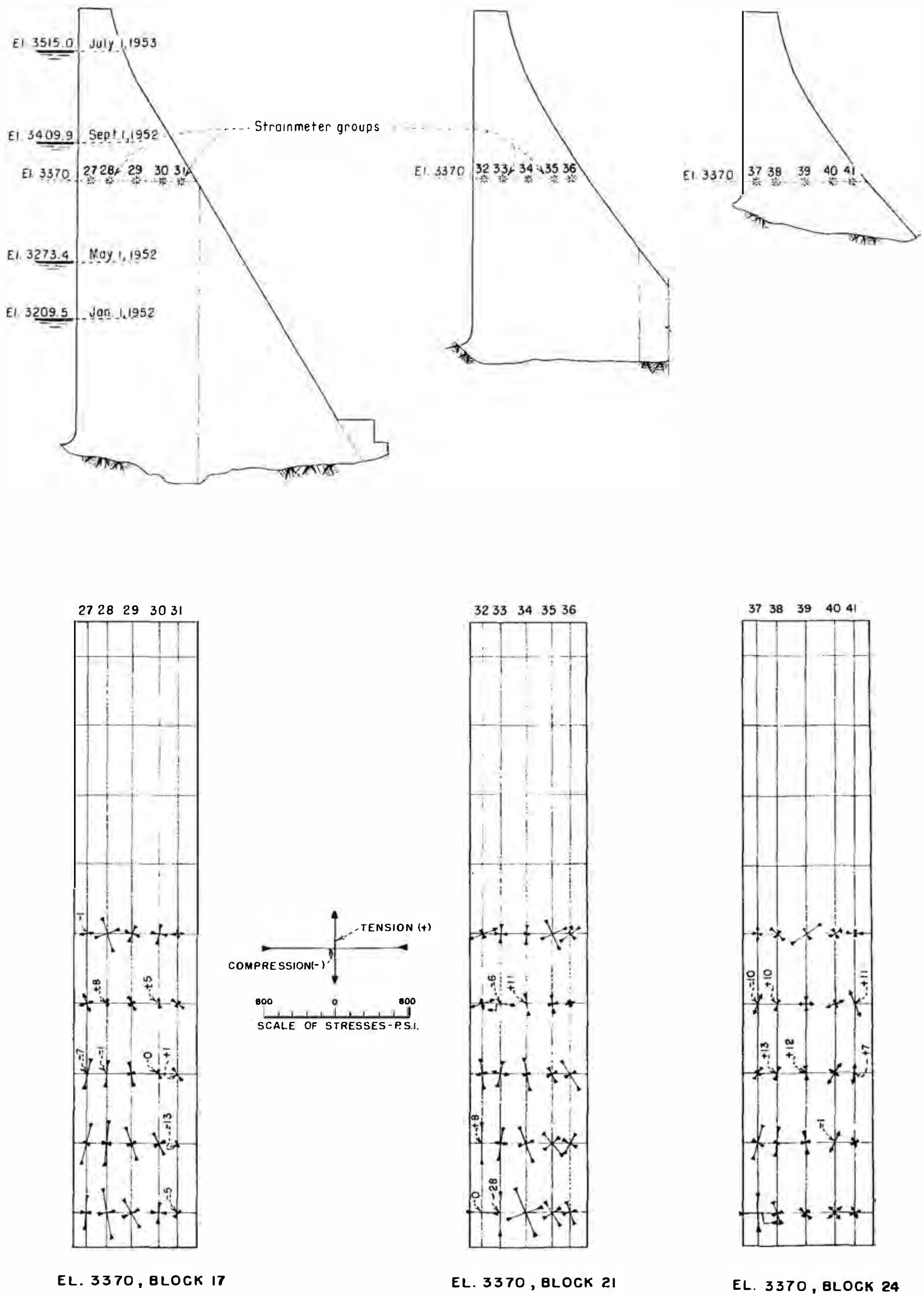


FIGURE 19 - Principal stresses in vertical planes--el. 3370, Blocks 17, 21, and 24.

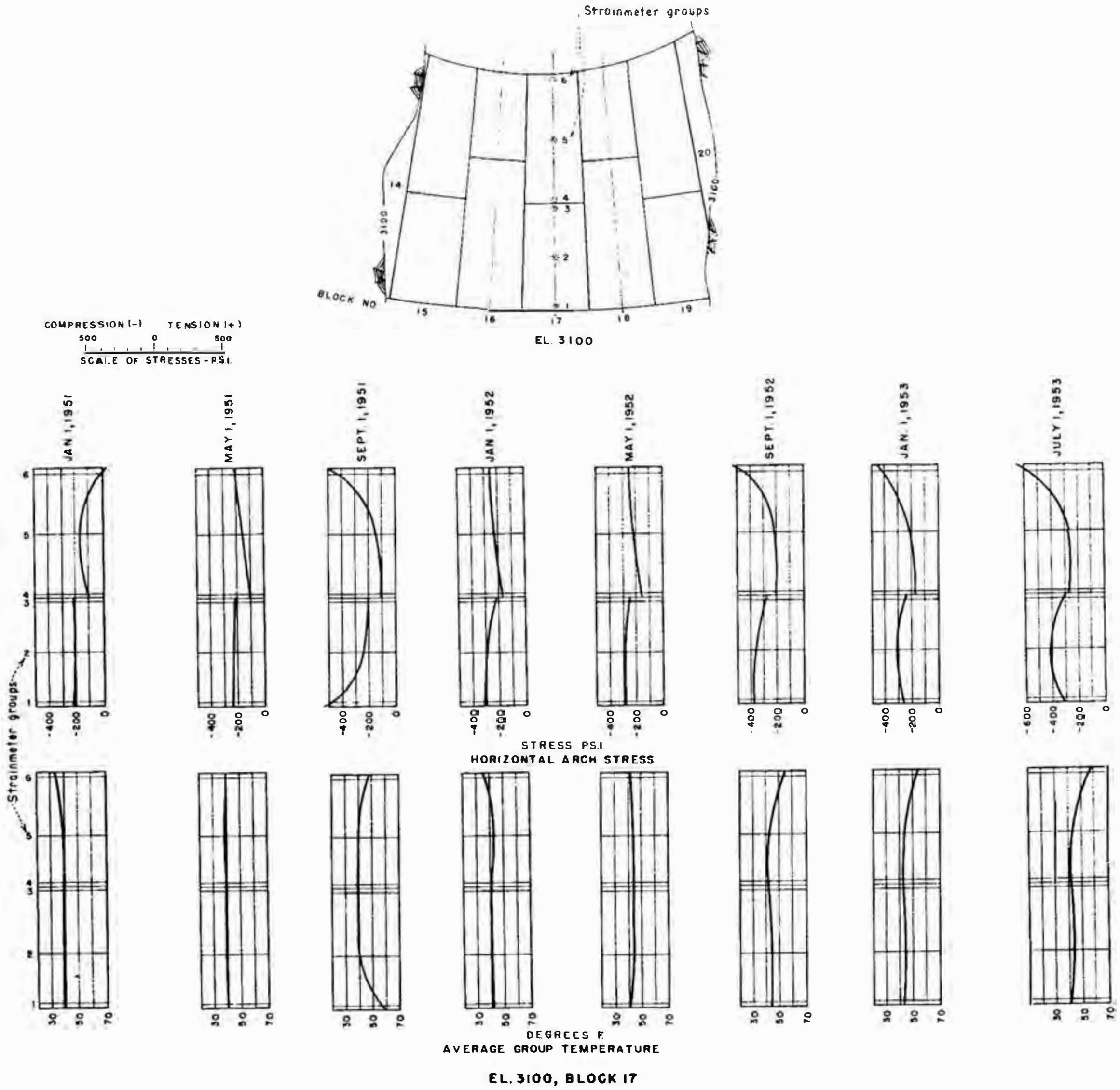


FIGURE 20 - Horizontal stresses and temperatures--el. 3100, Block 17.

El. 3100, Block 17 (Crown and Base of Dam)

Figure 20 shows the horizontal arch stress for the identical dates and loading conditions that are indicated on the cantilever stress figures. Horizontal arch stresses increase in magnitude at this location as the load increases. Effects of the longitudinal joint, as well as the influence of temperature variation, are significant. Increasing temperature at the downstream face induces increased stress near the face. At this base location the structural element appears more as a rectangle with curved faces than as an arch. The stress plots indicate stress in the element to be developed in each portion of the block divided by the longitudinal joint, each portion having its own stress pattern but approaching a smooth, continuous curve at the last plot of stress.

El. 3200, Block 12 (Left Quarter Point of Dam)

Figure 21 shows the horizontal arch stress at this location to have developed in a manner similar to that at the base of the maximum section. The horizontal structural element at this elevation approaches the dimensions of a thick arch, although curvature is rather slight. The stress plots indicate that near the abutment the stress is developed for each portion of the block divided by the longitudinal joint, and for all dates and increasing load conditions the upstream portion of the arch element develops more stress than the downstream portion. Varying temperature at the downstream face causes fluctuating stress near the downstream face. As reservoir water increases above the elevation of the strain meters, the upstream face stresses appear to stabilize and then increase in magnitude with increasing load on the dam.

El. 3370, Blocks 8, 12, 17, 21, and 24 (Complete Arch)

Figure 22 shows the horizontal arch stress as developed near the abutments, quarter points, and crown of one complete arch. Dimensionally this arch approaches a uniform thickness arch, since near the abutments there is only a slight increase in thickness over the crown. Early stress development indicates tensile stress at several locations, particularly near the upstream face. As load comes on the dam and reservoir water rises above the elevation of measurements, stresses at the upstream face appear to stabilize. The final stress devel-

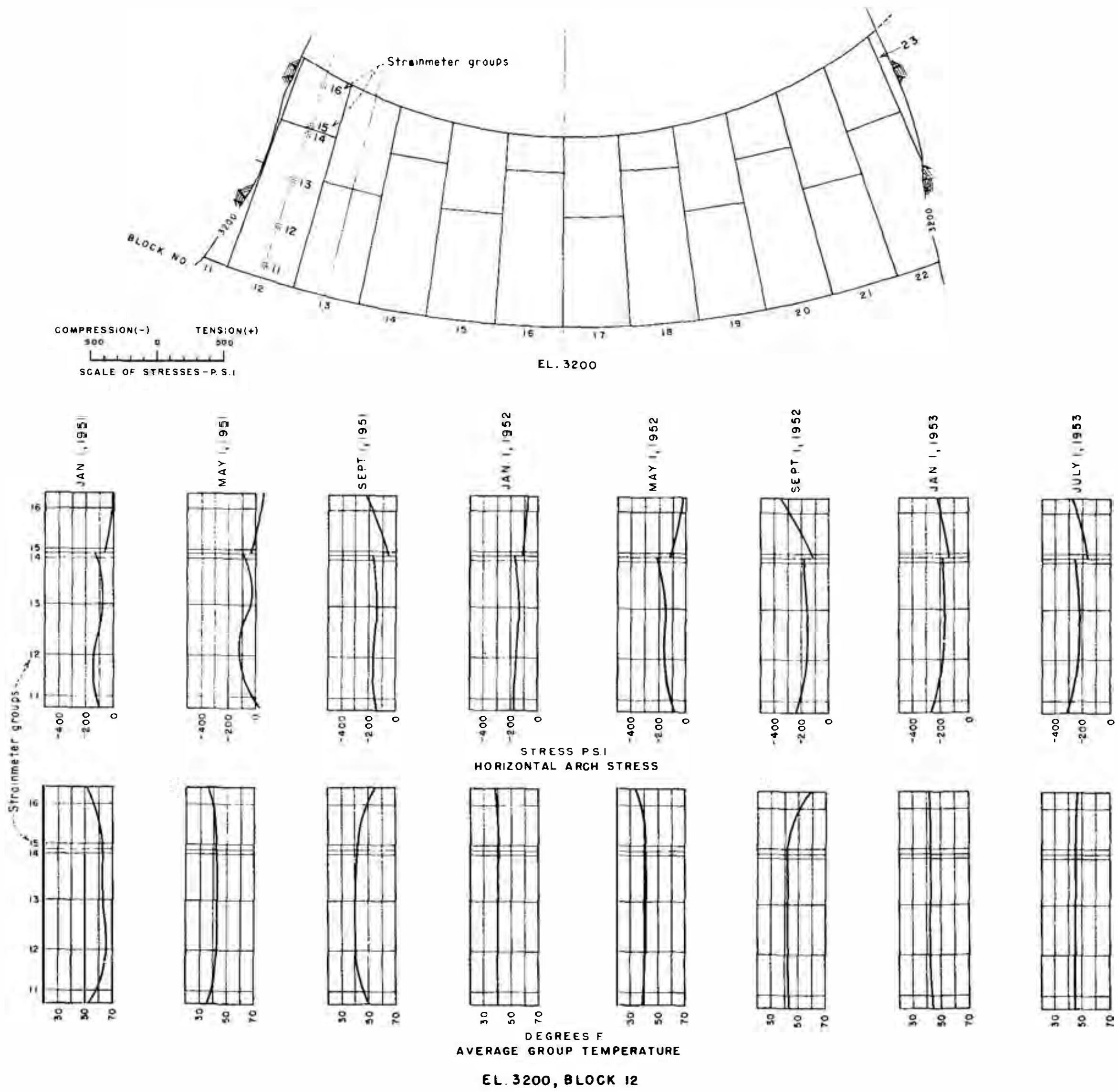


FIGURE 21 - Horizontal stresses and temperatures--el. 3200, Block 12.

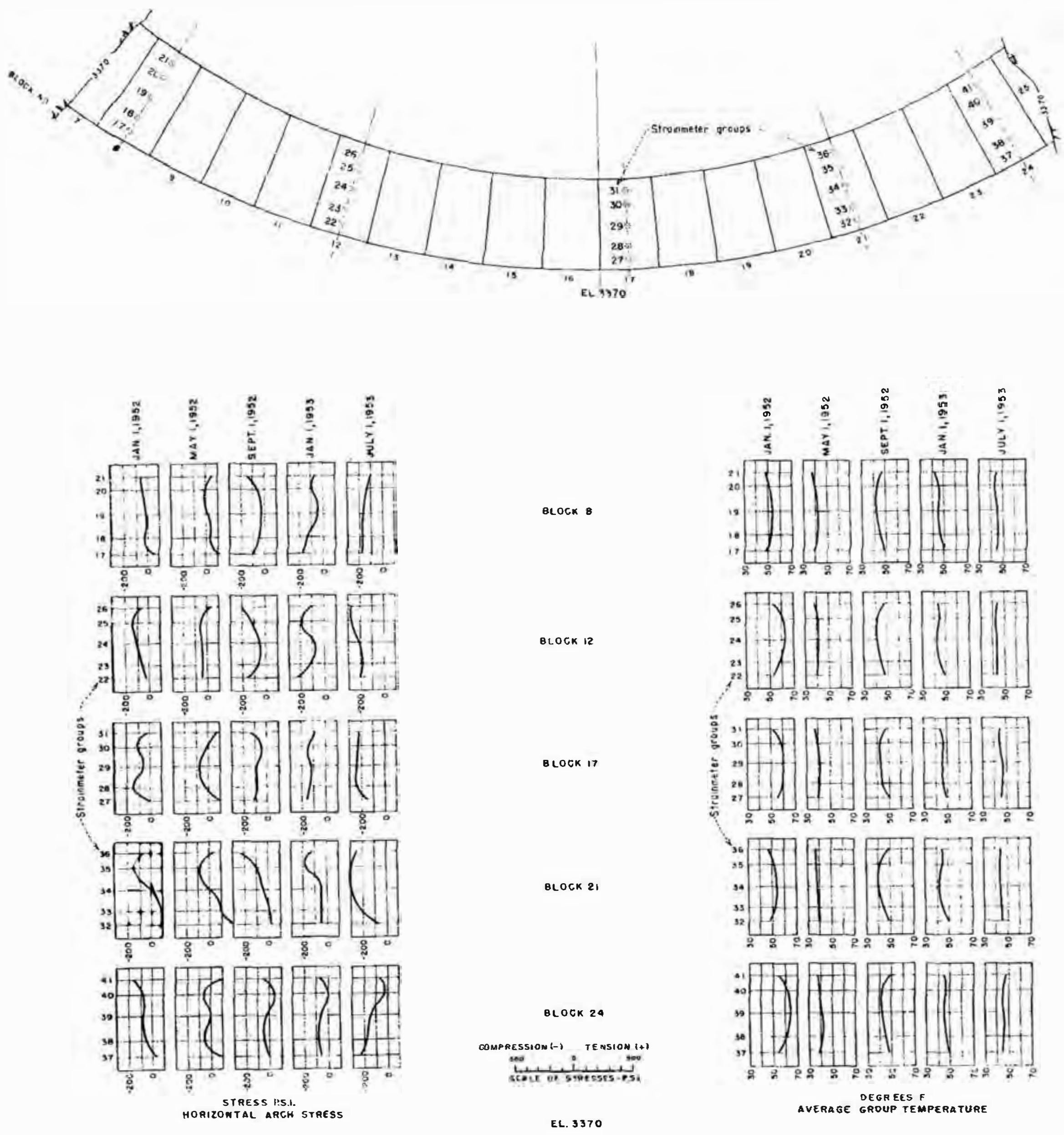


FIGURE 22 - Horizontal stresses and temperatures--el. 3370, Blocks 8, 12, 17, 21, and 24.

oped shows greater compressive stress at the downstream face and center of sections for crown and quarter points. At the abutments stress is greatest at the extrados of the arch and decreases to approximately one-half at the intrados.

The strain meter groups near the downstream face of the dam indicated fluctuating temperatures. These temperatures fluctuate according to seasonal temperature variations and cause a corresponding variation of stress near the downstream face.

Principal Stresses (Plane of Arch)

Figures 23 and 24 show the principal stresses and their orientation at the abutment of the elevation 3200 arch and at the several sections in the plane of the elevation 3370 arch. The major principal stresses are shown to be generally in the direction of expected arch thrust. Near the faces of the dam at the abutments of each arch the stress is rotated more than in the central portion of the arch.

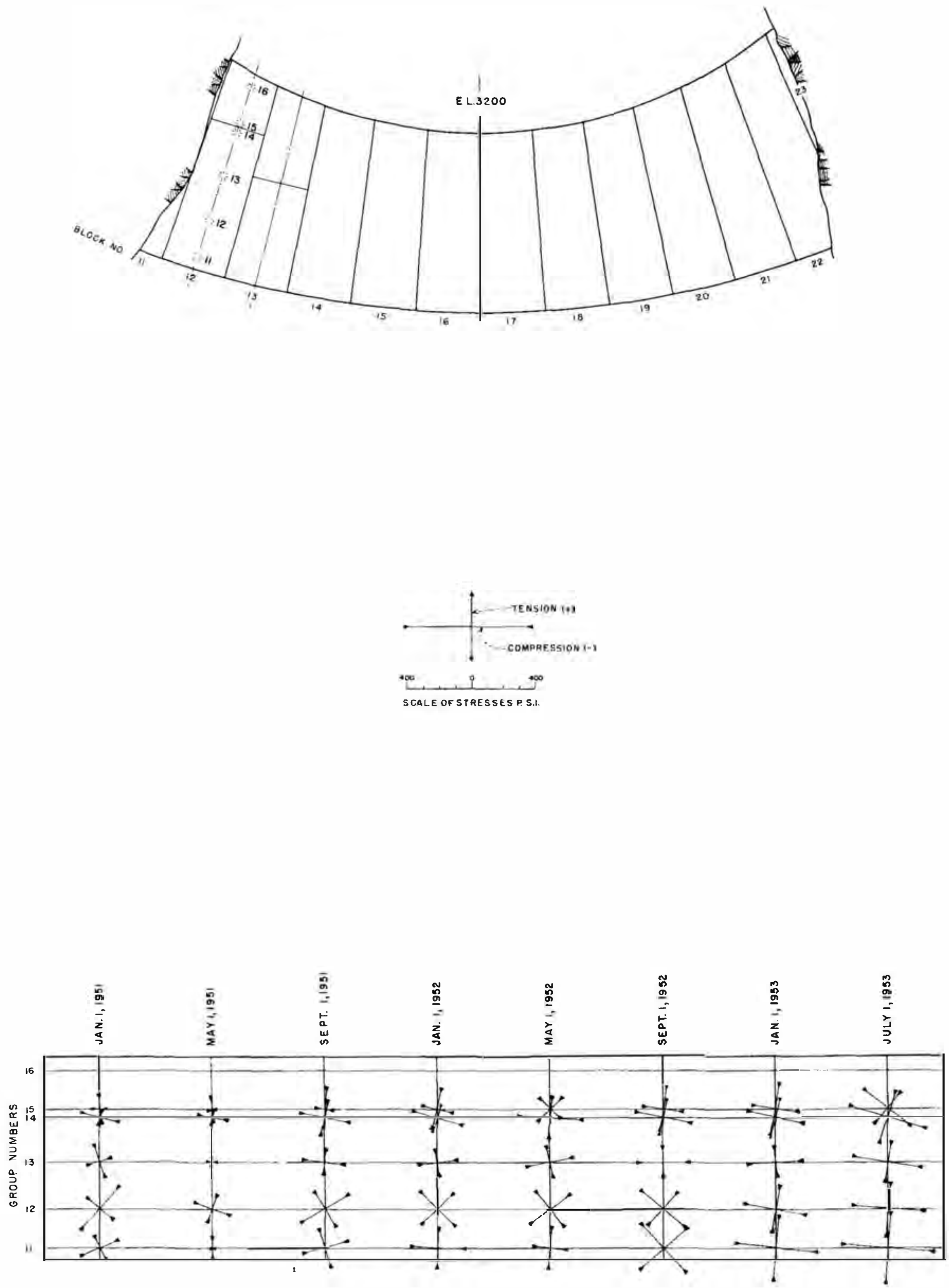


FIGURE 23 - Principal stresses in horizontal plane--el. 3200, Block 12.

Conditions of Temperature

The temperature of the blocks of the dam was effectively reduced to 38° F which had been determined as a desirable level from a temperature study made prior to construction of the dam. This temperature value was derived from records of annual air temperature, river water temperatures, and effects due to solar heat. Grouting operations were performed on radial and longitudinal joints when concrete temperatures reached the value of 38° F. Thus, after grouting, and with rise in temperature, expansion of the concrete occurred. The effect of the expansion was an increase in stress due to temperature rise. The effect of the temperature rise and increased stress is particularly noticeable at the downstream face of sections for which distributions of stress are shown. At the upstream face of the dam the same effect is noticeable for measurement elevations that are above the water surface of the reservoir, the stress varying with temperature. After the reservoir elevation increased to above the elevation of the measurements the stresses at these locations stabilized.

A point that shows effectively the stress caused by temperature variation is that at

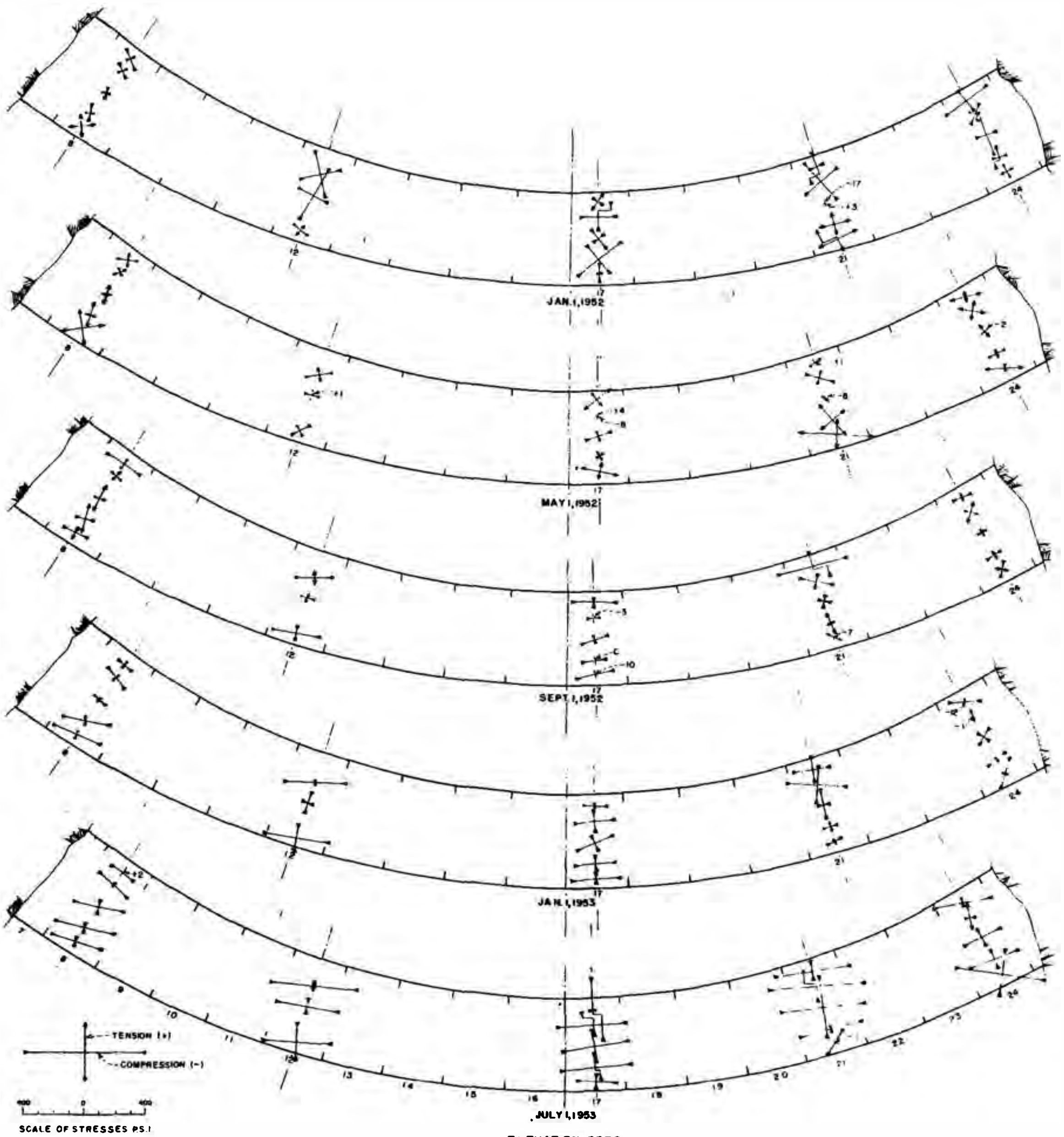


FIGURE 24 - Principal stresses in horizontal plane--el. 3370, Blocks 8, 12, 17, 21, and 24.

the downstream group of strain meters at the base of the crown cantilever, elevation 3100, Block 17. The powerhouse structure is located over this strain meter group location. At this point vertical stress during the early period of records indicated a low magnitude of stress. After powerhouse construction was completed the vertical stress increased at the point of measurement. The temperature gradient for the line of strain meter groups through the section shows increased temperatures at the downstream face stabilizing at about 56° F. The stress gradient for the section shows a marked increase in stress for this point, greater than at a point at the interior. Thus, the effectiveness of warming the face of the dam through room temperature of the portion of the powerhouse that is against the dam is reflected in the stress shown from the strain meter group at that location.

Final temperatures at the interior of the dam obtained by strain meter groups are indicated to have increased from 6° to 8° F above the temperatures at time of grouting, ranging from 44° to 46° F. Near the downstream face of the dam, the rise has been more pronounced in some instances, increasing to 56°, a rise of 18° F. Near the downstream face of the dam, the temperature fluctuates with annual air temperature. Near the upstream face of the dam the temperature is affected by reservoir water temperature. This effect is particularly noticeable when the reservoir is only a few feet above the elevation of measurement. As the reservoir elevation increases, there is a gradual decrease in concrete temperatures which indicates the effect of the reservoir temperatures.

Radiant Heat Effect

The temperature at which a dam is to be grouted is dependent not only on the mean annual air temperature but also upon the added temperature effect due to radiant heat derived from the sun. The increment due to solar effect is dependent on the orientation of the dam with respect to the path of the sun and the angles at which the sun's rays strike the faces of the dam. During construction, when no reservoir is present, the exposure of the upstream face of a dam to the sun must be considered as well as the exposure of the downstream face. At Hungry Horse Dam the temperature effect caused by solar energy was determined to be from 4.6° F to 5.9° F for the downstream face, and for the upstream face, prior to reservoir storage, to be from 4.3° F to 6.1° F.

Movements Indicated by Plumb Lines

Radial deflection of the crown cantilever, as indicated by plumb-line measurements, is shown in Figure 25. This figure shows radial downstream deflection at all points of measurement between the lower extremity of the plumb line and the top of the dam. Near the quarter points of the dam, the plumb lines show radial downstream deflections of lesser magnitude. The tangential movements of the dam indicated by the plumb lines are slightly toward the left abutment at the crown and generally toward the opposite abutment at the quarter points. These tangential movements indicate that along with the downstream radial deflections there is a slight closing together of the quarter points of the dam.

The deflections shown in Figure 25 are made for time intervals that correspond to those selected for stress conditions on the stress plots. As plumb lines were not installed in Hungry Horse Dam until about the end of 1952, only a small amount of deflection data is shown for the period of time included in stress results. The increasing deflections of the dam occur between 1953 and 1956 after the end of the stress investigation.

Behavior of Longitudinal Joint, Blocks 17 and 18

The two series of joint meters, each series comprising five instruments, that are located at several elevations on the longitudinal joint at the center lines of Blocks 17 and 18 between the base of the sections and the top of the joint, include a vertical height of 300 feet in Block 17 and 250 feet in Block 18. The behavior of the joint in these two blocks, at all elevations, is presented in Figures 26 and 27. The figures show that the opening of the longitudinal joint in each block increases with increase in elevation and is about equal in each block at corresponding elevations. Near the base of the section prior to grouting, the record of joint opening progressing with time indicates an opening to 0.062 inch in Block 18 and an opening to 0.060 inch in Block 17. At successive higher elevations, the joint opening is progressively greater than at the lower elevation and is a maximum near the top elevation. The maximum opening prior to grouting is 0.20 inch and occurs in Block 17. At the time of longitudinal joint grouting the joint was indicated to be 0.055 inch at the bottom and 0.210 inch at the top.

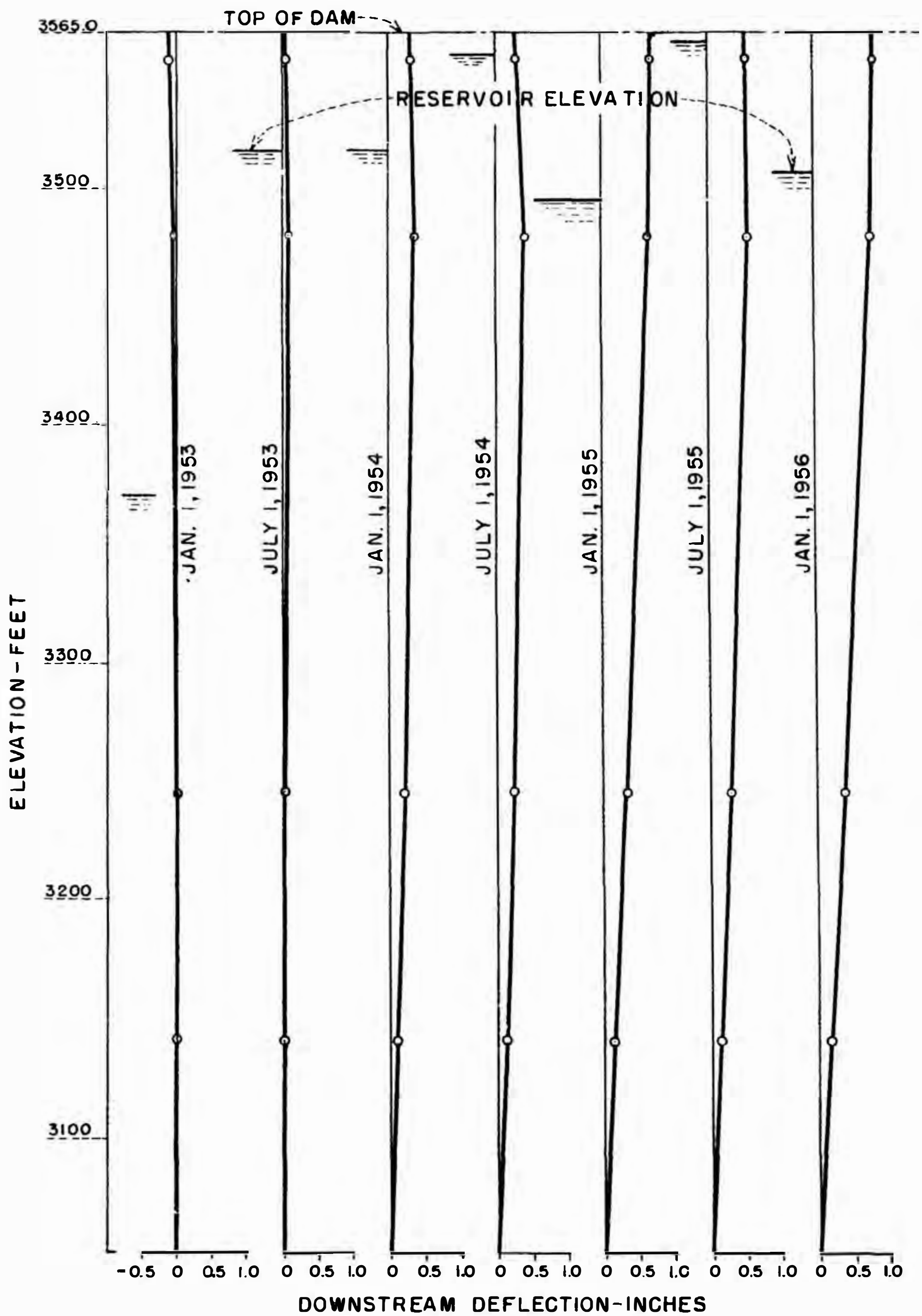
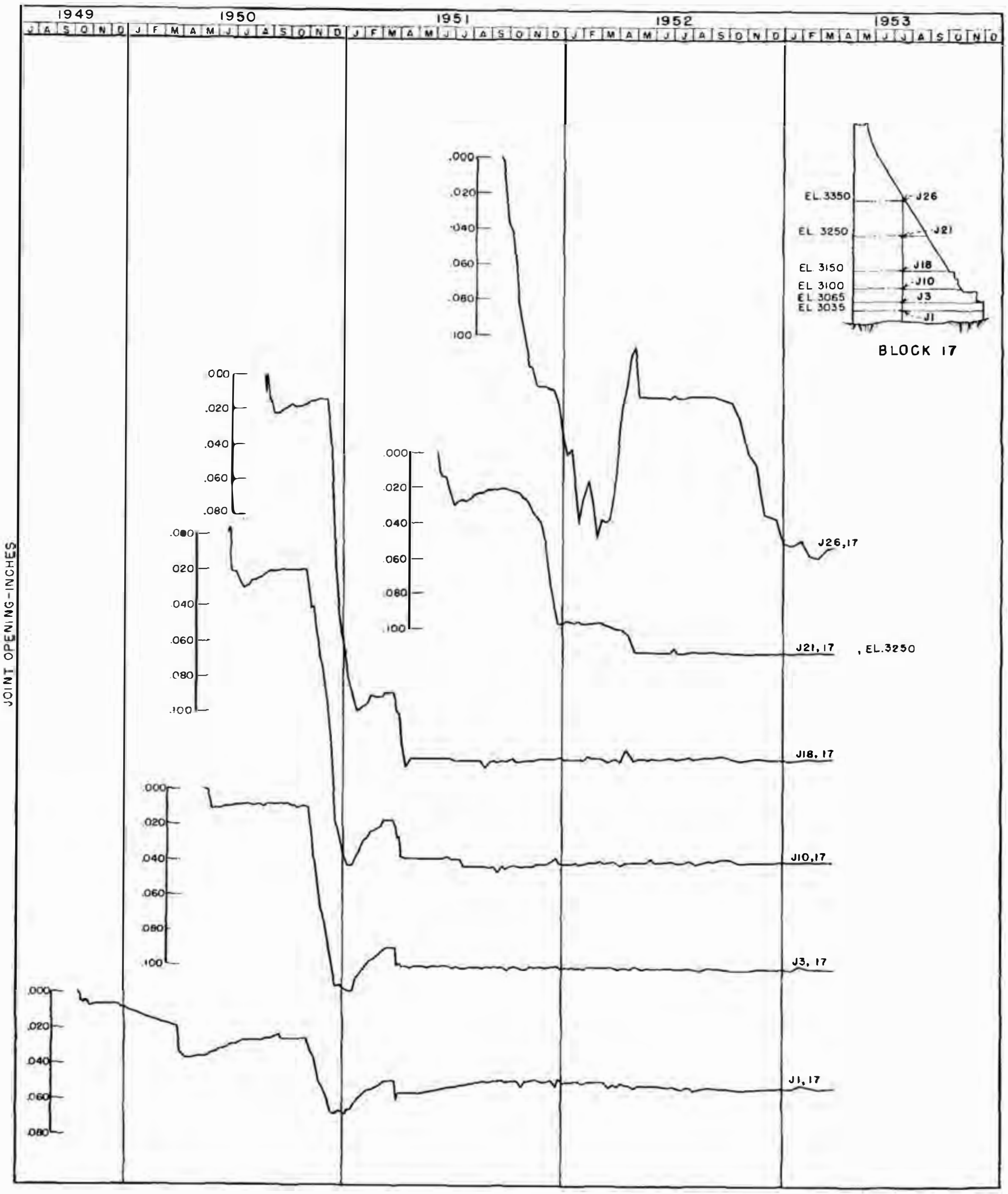


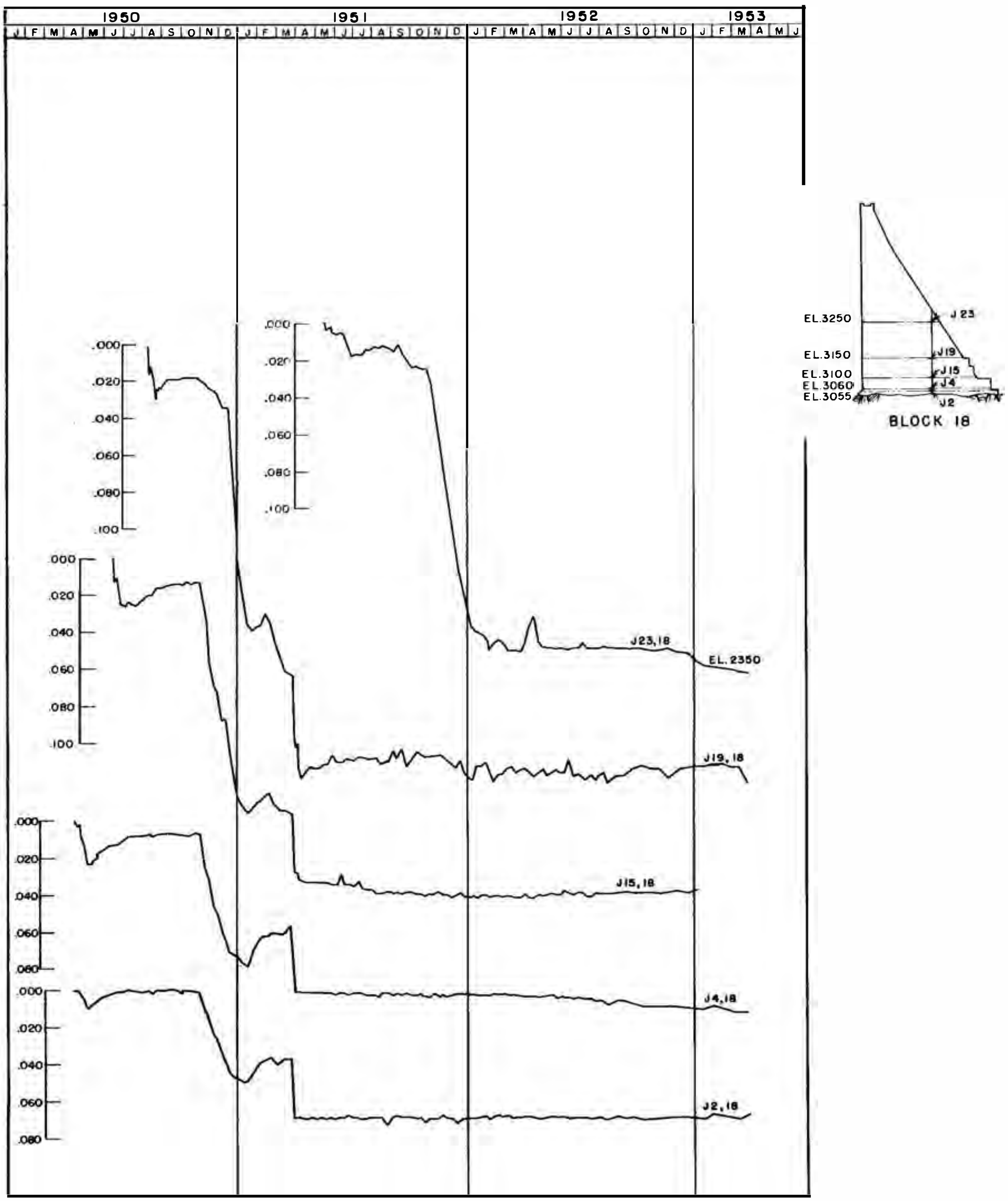
FIGURE 25 - Radial deflection measurements from Block 17 plumb line.



BLOCK 17

FIGURE 26 - Longitudinal joint openings--Block 17.

After grouting, the behavior of the joint in each block is indicated to remain approximately stable. At most points of joint-opening measurement, the magnitude of opening remained within limits of a few thousandths of an inch. However, at the top of the joint in Block 18 there is an indication of a slight increase in joint opening as the magnitude of the opening gradually increased about 0.020 inch from time of joint grouting in 1952 until the end of 1954. The point of indicated increased joint opening was at the upper extremity of the joint in Block 18. Possible movement may be attributed to



BLOCK 18

FIGURE 27 - Longitudinal joint openings--Block 18.

transient temperature effect since the point is in the thinner section of the block where the transverse thickness of the section decreased to nearly zero.

SUMMARY

The stress investigation described in this monograph serves the dual purpose of analyzing the structural behavior of Hungry Horse Dam and providing criteria for use in the design of future dams. The data derived from the study are valuable additions to data obtained from similar studies of other arch dams.

From a stress standpoint, Hungry Horse Dam is considered to be of safe design, as observed stresses are less than maximum allowable stress used in the design. The stresses at points of measurement show that for the maximum water load on the dam during the period of study, elevation 3515.0, the greatest indicated compressive stress is about 600 psi. This stress is about eight-tenths of 750 psi, the allowable design average unit stress.

Only slight tensile stresses were found at any of the points of measurement in the dam. Tensile stresses were indicated in the vertical planes near the downstream face of the dam at the elevation of the complete arch analyzed, and occurred during periods of greatest change in temperature at the points of measurement. At no time was tensile stress indicated in the plane of this arch.

During early construction vertical tensile stresses were indicated at the faces of the sections near the bases of the cantilevers in Blocks 12 and 17. As concrete weight and water load increased on these sections the tensile stresses disappeared. In the planes of the lower arches tensile stresses were present at the sections of measurement only during the earliest record. The low tensile stresses near the downstream face of the arch sections also disappeared as concrete and water load were added to the dam.

The shape of each vertical section of the dam where stress was measured is reflected in the distribution of vertical stress for the sections. The maximum stress is in the upstream portion of the section, that is, at points in the section above which there is the greatest elevation of concrete. This relationship would indicate that the vertical stress distribution could be controlled to some extent by design and by the method of construction.

The influence of the shape of the vertical section on stress is apparent in both arch stresses and cantilever stresses at elevation 3100, Block 17, and at elevation 3200, Block 12. At each of these locations the transverse blocks are divided by the longitudinal contraction joint of the dam, and as a result the design is reflected in the stress distribution for both vertical and horizontal planes. In the plane of the arch at each of these elevations there appear to be a primary and a secondary arch, each having its own horizontal stress distribution and a sharp discontinuity at the division by the longitudinal joint.

In the plane of the cantilever at elevation 3100, Block 17, and elevation 3200, Block 12, the upstream portion of the transverse block and the downstream portion of the transverse block each reflect a stress distribution characteristic of the shape of the block. In the downstream portion of Block 17 at elevation 3100 the weight of the powerhouse and the effect of the temperature within appear to be reflected in the downstream part of the vertical stress distribution curve.

The shear stress curves indicate that load is being carried by shear in the downstream portion of the transverse blocks divided by the longitudinal contraction joint.

This study indicates that the longitudinal joint in a dam has marked effect on the stress distribution on a section of the dam divided by the joint. Considerable discontinuity of stress can result, and the portions of the block on either side of the joint will each have their own individual stress distributions. Grouting of the opened joint may cause increased stress at the base of vertical sections and a redistribution of stress on horizontal arch sections caused by increased moment as a result of grouting. This stress distribution may be due to a "locked-in" stress condition that remains long after the dam is completed and in service. The stress distributions for both arch and cantilever indicate that a longitudinal joint may cause a primary and secondary arch, each having a separate and distinct stress distribution. The vertical stress distribution on sections separated by the longitudinal joint reflects the general shape of the sections, the stress largely being determined by the mass of the block. Thus, should the discontinuity in the stress distribution be considered undesirable in future dams, the criterion indicates that it would be advisable to obtain a continuous distribution of stress across a longitudinal joint.

The deflections of a dam as obtained by plumb-line measurements indicate the movement of upper portions of the structure with respect to the foundation. These deflections are useful to establish indicated trends of stress on sections of the structure. However, determination of any deformation of a structure with respect to the surrounding foundation and abutment structure requires additional measurements between dam and foundation or between dam and abutment structure. Thus, a system of points on the downstream face of a dam on which precise triangulation measurements may be made from points at some distance from the dam would be useful to determine deformation of the structure with respect to the foundation as well as deflection of the cantilever and arch elements of the structure. This method of measuring movement of dams by precise surveying has been used successfully at other dams.[4]

The results of this measurement study show the indicated stress at the internal points of measurement as reflected by the conditions of loading, temperature, and deflection that exist in the dam at the specific times indicated. In contrast, the results of an analytical study show the conditions of stress at the faces of the structure for the specific assumptions of the study and the selected conditions of loading and temperature. The results of the analytical study are the stress conditions for the one, and only one, condition of loading and show the stresses that result for only the conditions stipulated by the analysis.

A design study is usually made for the maximum loading on a dam, including added effects such as ice load, earthquake, and maximum reservoir water elevation, and will not serve for comparison with results of a measurement study. These added effects for maximum conditions contribute to stress within the structure and cause too great a difference in stress at the points of study to make a realistic comparison with stresses obtained by measurement.

Comparisons of stress results from measurement studies and stress results from analytical studies should be made only on a basis of average stress on a section or on a basis of stress trend. Measured stress is in terms of unit stress derived from strain, the strain being determined in masses of concrete which creeps under load. Computed stress from an analytical study is in terms of average unit stress at the faces of the structure. This is because stresses derived in the analytical study of a section are in terms of totals of forces and moments.

Structural behavior investigations thus indicate the need for analytical studies based on conditions that are comparable to conditions of measurement studies. Although analytical studies of maximum conditions of structural loading fulfill the requirements for the design of a structure, they do not supply stress results that may be considered comparable to stresses that will exist in the structure during conditions of normal loading. Similarly, these studies do not arrive at stresses which may be due to a partial reservoir or a full reservoir loading combined with temperature effects in the structure during early and normal operating service conditions. The additional analytical studies are thus needed so that comparisons on an equal basis may be made between results of the two types of studies. Comparisons between results of analytical studies and results of measurement studies for similar loading conditions of a structure will permit similarities and differences in stress to be noted. Conclusions may be drawn which will lead to economies in construction and realistic stress distributions in structural design.

ACKNOWLEDGEMENTS

The results of this investigation of stress by strain meter, temperature, and deflection measurements reflect the accumulative efforts of the many present and former members of the Bureau of Reclamation who have pioneered the techniques of installation, developed the theory used, and devised the methods for computation that have made possible this type of investigation. The efforts of J. M. Raphael, formerly with the Bureau, are particularly acknowledged for his contributions to the techniques, theory, and method, and for supervising the initiation of this investigation. The efforts of Bureau Engineers William T. Lockman, Keith Jones, James A. Stubbs, and Paul Rhoads are acknowledged for performing the major portion of the mathematical computations and data reduction represented in this monograph. The studies of structural behavior of concrete dams are under the supervision of A. W. Simonds, Head, Foundations and Structural Behavior Section, Dams Branch.

Credit is due former and present Hungry Horse Project engineers and technicians who installed and diligently made systematic readings on the embedded instruments. Special credit is due former Bureau of Reclamation Engineer A. A. Armstrong under whose supervision the major portion of the embedded instruments was installed in Hungry Horse Dam and who scheduled the program of readings that continues to date.

LIST OF REFERENCES

1. Hammond, J. J., "Basic Design Criteria for Concrete Gravity and Arch Dams," Journal, American Concrete Institute, Vol. 25, No. 8, pp 657-668, April 1954

2. "Structural Behavior of Ross Dam," Report to the City of Seattle, Bureau of Reclamation, Denver, Colorado, February 1955

3. Simonds, A. W., "The Determination of Stress in an Arch Dam from Observed Strains," (Ross Dam) RILEM Paper 4, U.S.A., Themes 1b et 3a, Lisbon, October 1955

4. Richardson, J. T., "Structural Behavior of Hoover Dam," Bureau of Reclamation Report, Denver, Colorado, November 1957

5. Raphael, Jerome M., "Effect of Longitudinal Joint on the Stresses at the Base of an Arch Dam," (Hungry Horse Dam) RILEM Paper 33, U.S.A., Theme 3A, Lisbon, October 1955

6. Hanson, J. A., "A Ten Year Study of Creep Properties of Concrete," Concrete Laboratory Report No. SP-38, Bureau of Reclamation, Denver, Colorado, 1953

7. Raphael, Jerome M., "Determination of Stress from Measurements in Concrete Dams," Paper No. R-53, Third Congress on Large Dams, Stockholm, 1948

8. Raphael, Jerome M., "The Development of Stress in Shasta Dam," Transactions, American Society of Civil Engineers, Vol. 118, page 289, 1953

9. McHenry, Douglas, "A New Aspect of Creep in Concrete and Its Application to Design," Proceedings, American Society for Testing Materials, Vol. 43, page 1069, 1943

10. Raphael, Jerome M. and Bruggeman, John R., "Analysis of Strain Measurements in Dams by Use of Punched Card Machines," presented at Convention of American Society of Civil Engineers, Denver, Colorado, June 1952

11. Jones, Keith, "Calculation of Stress from Strain in Concrete," Technical Memorandum No. 653, Bureau of Reclamation, Denver, Colorado, February 1955

GPO 830398