STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS

REPORTS OF THE
DIVISION OF WATER RESOURCES
EDWARD HYATT, State Engineer

REPORTS OF

CONSULTING BOARD
ON
LAKE HODGES DAM

SAN DIEGO COUNTY, CALIFORNIA

MEMBERS OF BOARD
B. A. ETCHEVERRY F. C. HERRMANN A. KEMPKEY
HENRY D. DEWELL

1929

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C O N S U L T I N G   B O A R D

On

L A K E   H O D G E S   D A M

SAN DIEGO COUNTY, CALIFORNIA

<u>Members of Board</u>

B. A. Etcheverry,   F. C. Herrmann,   A. Kempkey

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1929

(SEAL)

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HISTORICAL SUMMARY AND FOREWORD

The construction of Lake Hodges Dam, a multiple arch structure of the Eastwood Type, was completed on the San Dieguito River in 1918. This dam has a height of 115 feet from streambed to spillway crest level and a storage capacity of 37,700 acre feet.

Approval of the plans, together with inspection during construction, was exercised by the late W. F. McClure, at that time State Engineer of California.

During April and May, 1928, an investigation of all dams included in the impounding system of the City of San Diego was made by a Board consisting of A. J. Wiley, L. C. Hill, C. R. Olberg, Consulting Engineers, and C. F. Tolman, Geologist. This investigation was made in accordance with a resolution of the Common Council dated April 16, 1928 and was limited to a determination of safety of these various dams. From page 49 of their report dated May 21, 1928, "It has been impossible to check the design of the dam (Lake Hodges Dam) in the short time available as a long and highly involved series of computations is required. The conditions found, however, make it imperative that someone especially qualified in the design of multiple arch dams make an analysis of all stresses in Hodges dam and make a complete examination of the entire structure. This should be done without delay." The recommendations of this Board relative to Lake Hodges Dam are outlined on page 50 of the report and are as follows:

"1. That a complete examination of Hodges Dam be made immediately with especial reference to the cracks in the buttresses.

2. That a complete analysis of all stresses in the dam be made immediately by someone especially qualified.

3. That immediately subsequent to the next large flood the dam and spillway be protected completely and permanently against erosion and that the spillway approach channel be improved."

A report on Hodges Dam was later prepared under the direction of the hydraulic engineer of the San Diego Water Department by Mr. J. H. A. Brahtz of that department. This "Report and Recommendation Lake Hodges Dam" submitted July 24, 1938 recommended as follows:

"1. Provide additional lateral interbracing in area of cracks of Buttress Wall No. 10 to 21 inclusive.

2. Determine safe load on rock either from available logs or by new test borings, and examine especially the sub-soil under and adjacent to Buttress No. 19.

3. If the determined safe load is less than 30 tons per square foot, provide such additional foundation area as will be needed for reduction of rock pressure at points of maximum, and design these with due regard to present width of crack and height of buttress, so that the original plane of arch groins is reestablished.

4. Consolidate concrete along cracks in arches over passage-way on lower upstream strut.

5. Establish a systematic periodical inspection of the dam for the purpose of watching development of present and new cracks. Reset gauge points in walls so as to read horizontal and vertical components of relative displacement at crack.

6. Keep careful records of behavior of present cracks along with temperature readings at upstream and downstream edge of buttress and in addition establish weekly readings of temperature, of atmosphere and of water at surface and at bottom of reservoir."

Question having been raised as to the safety of such an important structure a Board consisting of Prof. B. A. Etcheverry, F.C.Herrmann and A. Kempkey, Consulting Engineers, was retained by State Engineer Edward Hyatt to examine the Hodges Dam and report on the technical considerations of the structure. To assist in their investigation the Board availed themselves of the services of Mr. H. D. Dewell, Consulting Engineer of San Francisco. These consultants exhaustively and comprehensively reviewed the safety features of this structure and their report in full follows.

Report to State Engineer

on Lake Hodges Dam

by

F. C. Herrmann, Consulting Engineer.

B. A. Etcheverry, Consulting Engineer.

A. Kempkey, Consulting Engineer.

-:-:-:-

March 11, 1929.

-:-:-:-

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F. C. HERRMANN
Consulting Civil Engineer

San Francisco, March 11, 1929.

Mr. Edw. Hyatt, State Engineer,

Forum Building,

Sacramento, California.

Dear Sir:

At your request we have made an examination and investigation of the Lake Hodges Dam in San Diego County, and submit the following report.

Field examination of the structure was made on December 6th and 8th, 1928, during which time all component parts of the structure were closely examined.

We were accompanied by the engineers of the City of San Diego and of the other interests concerned with Lake Hodges reservoir, and were given every opportunity by these engineers to examine all data and reports in connection therewith, and we wish to express our appreciation for their hearty cooperation.

In addition the full records of the State Engineer were made available, among which were plans and specifications of the dam, and monthly progress diagrams of construction and numerous reports of inspection during the progress of the work.

The special reports available are the following:

1. Report and Recommendations, Lake Hodges Dam, made to H.N.Savage by H.A.Brahtz, dated July 24th, 1928.

2. Report on the Examination of the Dams of the Water Supply System of the City of San Diego to the Mayor and Common Council of the City of San Diego, by Louis C. Hill, C.R.Ohlberg, A.J.Wiley and C.F.Tolman, dated May, 1928.

3. Report on Lake Hodges Dam to J.B.Lippincott, by Charles Derleth, Jr., dated July 18th, 1922.

Records in the office of the Hydraulic Department of the City of San Diego which include:

1. Measurements of changes in the widths of cracks in the buttresses from October, 1922 to September, 1928.
2. Hydrographic data relating to floods.
3. Storage curve and spillway discharge curve of Lake Hodges reservoir.

In addition to the above, other data were obtained from the San Francisco office of the United States Geological Survey, and from the Engineer of the San Diego County Water Company.

Mr.W.A.Perkins, Associate Engineer of the State Engineering Department, at our request made studies of stresses in the various elements of the structure under assumptions as to external loadings.

Mr.H.D.Dewell, Consulting Structural Engineer of San Francisco, at our request has carefully reviewed the structural phases of all the available reports and computations, and has made independent analyses of various stresses in the structure with certain assumptions as to behavior under external loads. Mr.Dewell is a member of the Committee of the American Society of Civil Engineers which, after investigations over the past four or five years, recently reported on the effects of earthquakes on engineering structures. He has given consideration to possible effects of earthquakes on Lake Hodges Dam, appreciating that the science of determining stresses due to earthquakes is yet in its infancy.

Description of Lake Hodges Dam (Plate 1)

Lake Hodges Dam is a concrete multiple arch dam of the Eastwood type. It is located on the San Dieguito River about thirty miles from San Diego. The dam consists of the main non-overflow portion, with a crest length of 390 feet, the ogee rollway portion 175 feet in length, and the broad crest spillway extension cut in the side hill 185 feet in length. The maximum height of the dam is about 130 feet above the streambed. The top of the dam is 15 feet above the crest of the spillway. The buttresses are unreinforced, except where arches, strut beams and foot walks are tied in. They are spaced 24 feet on centers. Their thickness is 18 inches for the top 47 feet, and increases to 4 feet at the base for the highest buttress. The northerly seven buttresses are covered with a reinforced concrete deck rollway forming the ogee portion of the spillway. Reinforced concrete struts placed at intervals of about 35 feet both vertically and horizontally give the buttresses some lateral bracing. The arches are reinforced and have a minimum thickness of 12 inches for the first 50 feet below the crest. Below the 50 foot point the thickness increases to 2 feet and 7-1/2 inches at the bottom of the dam. The slope of the crown of the arches is 45°.

The dam was completed in 1918. On account of the small run-off for the 1918-1919 season, less than 3000 acre feet was held in storage in 1919. In 1920 the maximum amount held in storage was 15,500 acre feet, with the water level in the reservoir about 23 feet below the crest of the spillway. Water remained at about this level in the reservoir until about December 1st, 1921, when it began to rise, and reached the crest of the spillway on December 26th of that year. Surplus water flowed through the spillway from December, 1921, to June, 1922, and from January, 1923, to May, 1923. The reservoir has been fairly well filled since the beginning of the year 1922.

Assumed Loadings

In all the computations of stresses in the various loadings it has been assumed that the reservoir was full to the crest of the dam.

Such condition will occur only in time of maximum flood of 90,000 second feet, which we estimate may occur once in 500 to 1000 years, and in such instance will last but a few hours. The maximum height of water in the reservoir since it was completed was about 3-1/2 feet below crest of dam. This was in the flood of 1927 when the water stood within four feet of the crest for about four hours.

Had the large flood of 1916 occurred since the dam was completed, the water would have stood at a maximum height of 1 foot from the crest of the dam, and would have stood within 4 feet of the crest for about twelve hours. With the spillway improved, as discussed later in this report, the maximum height of water for a 1916 flood would have been 2 feet below the crest, and would have stood within 4 feet of the crest for about twelve hours.

Except in times of large flood, the water stands at least 15 feet below the crest. It is therefore seen that the maximum stresses mentioned in this report may occur but rarely, and if they occur are sustained but for a short time. During all but these short periods of times, the stresses are considerably less than those discussed in this report.

Foundations

The geology of the damsite, as described by Dr. Tolman, geologist of Stanford University, is as follows:

"The foundation consists of a variety of volcanic rock including layers of rhyolite, basalt and of andesitic agglomerate. These formations belong to the group classified as 'pre-granitic volcanics'. At about 100 yards above and to the right of the structure, the lava foundation rocks are in contact with the younger intrusive granite. A zone of intense shearing and probably of minor faulting cuts through the spillway to the right of the dam. This zone of weakness, however, probably does not seriously affect the dam structure.

The foundation rock is cut by very prominent zones of shear, and the surface rock is blocky and subject to ravelling by action of overflow waters. Spillway protection is therefore essential".

In general the foundation rock may be considered satisfactory.

The map of the Seismological Society of America shows a few geological faults in this part of California, though none of them are in close proximity to the dam.

The records of the State Engineer show clearly that very careful scrutiny was given the foundation for each buttress during the progress of the work. In the bottom of the stream foundations were acrried from 4 to 8 feet into bedrock and on the sidewalls of the canyon excavation was carried to depths of from 8 to 21 feet in order to secure good foundations.

Foundation Pressures

The maximum foundation pressure is approximately 20 tons per square foot, and the maximum shear along the base is estimated at approximately 145 pounds per square inch. These are not excessive. Should the cracks in the buttresses referred to later, extend so as to pass through the arch rings and the base, a different distribution of the foundation pressure would result, with a maximum pressure of about 28 tons per square foot, which the foundation would safely stand.

Inspection of the footings of the buttresses after ten years of service, and of the exposed rock at the damsite, show that the foundation is amply strong to support the structure.

Buttresses

Stresses in the Buttresses

The buttress walls are high and slender, unreinforced and tied together by widely separated concrete struts. Without proper bracing the walls might be subject to excessive deflection. That the struts have been effective in preventing excessive deflection is indicated by the fact that the structure has served successfully for about ten years, during which time it was once subjected to maximum loading.

Analyses of the buttresses when considered as walls show unit stresses which are not excessive.

If considered as columns the determination of additional bending stresses is subject to assumption and conjecture. They would, however, tend to increase the direct stresses. Modern practice would require that the buttresses be adequately reinforced and braced. We believe, however, that the buttresses will continue to carry the loads, provided the structure is not subjected to the shock incident to an earthquake of major intensity or that resulting from overtopping.

Cracks in the Buttresses

Cracks in the buttresses are reported to have occurred prior to storage of water in the reservoir, and frequent micrometer measurements of the changes in the widths of the cracks have been taken since October, 1922, up to the time of our examination of the dam.

The cracks have been mapped and are shown on diagrams included in Mr. Brahtz's report. A typical crack is shown on Plate 2 of our report. The cracks generally extend upward from the roof of the gallery archway toward the arch rings in a direction about at right angles to them, and downward from the floor of the gallery foot walk about vertically toward the foundation. The cracks have a maximum width at the gallery. They gradually decrease in width in either direction from the gallery, and end before reaching the arch rings at the upper end and the foundation at the lower end, with the exception of one crack the upper end of which extends for a short distance into the arch ring.

We have plotted on separate graphs the micrometer measurements of the changes in the horizontal and vertical components of the width of the cracks, (Plates 3 to 7), and have studied the factors which might have a tendency to effect the changes, including the records of temperatures taken in the buttresses and the variations in water level in the reservoir.

From the separate graphs of the cracks we have prepared a composite generalized graph for the horizontal and vertical components, showing the general behavior of the cracks during the period of measurements. (Plates 8 and 9).

The graphs show that the cracks obtain their maximum widths during the winter and their minimum widths during the late summer. There is a gradual small increase in the width of the cracks from one season as compared to that of the preceding season. The average annual permanent increase in the vertical and horizontal components of the width of cracks is each approximately one- one-hundredth of an inch.

We find that the variations in water pressures on the dam have no relation to the variations in the width of the cracks, and the study of stresses in the buttresses shows no loading stresses which could have produced the cracks. We condlude that the cracks themselves as well as the variations in their widths are due to the expansion and contraction resulting from changes in temperature and moisture conditions together with shrinkage occurring during the period of hardening of the concrete. The permanent increase in width is due to the long time continued shrinkage of concrete in the process of hardening, and the water pressure transmitted to the portion of the buttress lying above the crack as this pressure favors the widening of the crack during the period of contraction in the concrete and resists narrowing of the crack during the subsequent period of expansion of the concrete. In addition closure of the cracks may be mechanically restricted by the lodgment of dust or small particles of foreign matter.

The presence of the cracks in the buttresses does not affect their stability. Should these cracks in the future be so enlarged as to extend upward and completely through the arch rings and downward to the foundations so as to completely separate the upper portion from the lower portion of the dam, we would have the condition assumed by Mr. Brahtz in his very thorough analysis.

Possible Effect of Cracks on Buttress Stresses

Mr. Brahtz has made a comprehensive study of the stresses which would result on several horizontal sections of the buttresses at different elevations for two possible conditions which may result from the cracks. In one case he assumes that the cracks extend upward clear through the arch barrel and downward to the foundation. In the other case he assumes that the arch barrel at the extension of the cracks would resist a tension of approximately 50 pounds per square inch. He obtains the following results for the properties and stresses on the horizontal section at Elevation 200, at the base of the tallest buttress:

Properties and stresses for a horizontal section at the base of tallest buttress

1. Assuming that the cracks extend through the arch barrels and to the foundation:

   (a) For the portion of the dam west of the crack:

      Safety factor against overturning = 1.485

      Coefficient of sliding = .582

      Maximum foundation pressure at heel = 27.6 tons per sq. ft.

   (b) For the portion of the dam east of the crack:

      Safety factor against overturning = 3.22

      Coefficient of sliding = .822

      Maximum foundation pressure at crack = 18.7 tons per sq. ft.

2. Assuming that the concrete of the arch barrel at the extension of the cracks resists 50 pounds per square inch tension:

(a) For the portion of the dam west of the crack:

Safety factor against overturning = 1.542

Coefficient of sliding = .522

Maximum foundation pressure at heel = 24 tons per sq. ft.

(b) For the portion of the dam east of the crack:

Safety factor against overturning = 2.71

Coefficient of sliding = .863

Maximum foundation pressure at crack = 21 tons per sq. ft.

The above values may be compared with the following values obtained for the buttress acting as a whole with no cracks:

Safety factor against overturning = 3.22

Coefficient of sliding = .710

Maximum foundation pressure at heel = 19.3 tons per sq. ft.

This analysis, together with the studies of Mr. Dewell, show that while the distribution of the foundation pressures would be changed, the maximum pressures resulting therefrom would not be excessive.

Arches

Mr. Brahtz has also made a careful detailed analysis of the stresses in the arch rings at several elevations. Mr. Dewell finds his results correct, except that Mr. Brahtz obtained a maximum total compressive stress in the concrete for the arch ring at Elevation 205 of 1203 pounds per square inch, apparently based on the assumption that the steel reinforcing could not take the tensile stresses. Mr. Dewell finds that the steel is adequate and that, including its effect, the maximum compression in the concrete is 910 pounds per square inch and the tensile stress in the steel is less than 14000 pounds per square inch.

The above stresses incoude the additional stress due to a fall in temperature of 15°. Without this the maximum compressive stress in the concrete is approximately 600 pounds per square inch.

The arch ring section at Elevation 205 is about 5 feet above the base of the arch at the lowest point of the dam. The additional five foot depth of water would give slightly larger stresses for the lowest arch ring, if the cantilever action resulting from the connection of the base of the arches with the foundation is neglected. The reduction in arch stresses due to this cantilever action warrants the conclusion that the maximum stresses are those obtained for the arch ring at Elevation 205.

In the computation of these stresses no allowance was made for the effect of the increase of moisture content in the concrete, which would tend to offset the stresses due to a fall in temperature. Lack of data regarding the distribution in moisture content and its expansive action do not justify detailed computation of stresses resulting therefrom, but it is known that they are of considerable magnitude and probably more than offset the stresses due to fall in temperature.

A consideration of all factors producing the stresses leads us to the conclusion that the arches are safe.

Effect of Possible Earthquake on Dam

The structure was subject to an earthquake shock of moderate intensity in 1918 soon after its completion, and was not damaged thereby.

Mr. H. D. Dewell, after considering the possible effects of a major earthquake, states as follows:

"Computations of actual stresses likely to be induced in the various structural units comprising the Lake Hodges Dam should be regarded as qualitative rather than as possessing any quantitative accuracy.

An earthquake shock of destructive intensity, with a direction up and down stream, might crack the arch barrels, but would probably not destroy them. The principal danger would be in the additional stresses induced in the high unreinforced buttress walls. These would undoubtedly be further cracked; they might be expected to fail, due to lack of reinforcing and proper bracing.

A shock of major intensity, with a direction perpendicular to the line of the stream bed, or parallel to the face of the dam would, in my opinion, be likely to cause more damage. Such a shock would cause the buttress to vibrate, and tear out the light and widely spaced bracing. These buttresses are of varying heights; consequently they will vibrate with different periods, and differential motion must result, unless the bracing is of such strength as to cause them to act together. The existing bracing has no such strength, and in my opinion would probably be torn out."

He recommends that to resist an earthquake of a major intensity, the buttresses should be braced crosswise of the stream by ties made capable of resisting bending stresses. The findings and recommendations of Mr. Dewell apply to major earthquakes.

The records show that San Diego was severely shaken in 1812 and again in 1857. There is reason to believe that the Lake Hodges Dam will doubtless be subjected at some time in the future to earthquakes of major intensities. Therefore it would be prudent to reinforce the dam against such a contingency by thorough bracing of the buttresses in a manner similar to that suggested by Mr. Dewell.

Flood Flows, Spillway Capacity, and Spillway Improvements

Maximum Flood Since Completion of Dam in 1917

The maximum flood since 1917 is that of February, 1927. This was a major flood and was over three times the 1922 flood, the next largest since 1917.

The hydrograph of the 1927 flood prepared by F. E. Green gives the discharge of the San Dieguito River, at the Lake Hodges Spillway, at intervals during the period from February 14th to 17th, 1927. The maximum crest discharge through the spillway was 36,000 second feet.

To compare this flood with that of January, 1916, which is the maximum on record, it is necessary to compute the February, 1927, flood flow as it would have been without the regulating effect on the crest flow produced by storage. This was done by computing the inflow into the reservoir from the outflow at the spillway, using the storage capacity curve of the reservoir and the spillway discharge curve, prepared by I.F. Judy, from F.E. Green data. (Plate 10). The results are given in Table 1. This table shows a crest flow, unregulated by storage of 50,000 second feet.

Table No. 1

Flood Flow Discharge of San Dieguito River at Lake Hodges in February 1927 without and with effect of storage

Date	Time	Outflow Discharge in sec. ft.	Reserve Gauge Height	Outflow mean discharge in sec.ft. for mean time of period	Computed inflow discharge without storage for mean time of period
Feb.15	noon	5,200	117.55		
				5,250	6,500
	2 p.m.	5,300	117.60		
				5,700	8,100
	3	6,100	117.85		
				6,800	20,000
	4	7,500	118.35		
				8,250	16,700
	5	9,000	118.90		
				9,500	19,000
	6	10,000	119.20		
				12,200	35,000
	7	14,400	120.60		
				14,600	17,000
	8	14,800	120.70		
				15,000	17,400
	9	15,200	120.80		
				15,400	16,500
	10	15,600	120.90		
				15,750	35,000
	10:45	15,900	121.00		
				16,100	26,500
	11	16,300	121.10		
				17,400	19,800
	12	18,500	121.65		
				19,800	35,500
	1 a.m.	21,100	122.35		
				21,800	17,000
	1:30	22,500	122.75		
				22,750	36,500
	3	23,000	122.85		
				23,750	33,500
	4	24,500	133.20		
				25,650	33,000
	5	26,800	123.80		
				27,800	41,200
	6	28,800	124.30		
				29,700	37,000
	7	30,600	124.75		
				31,800	47,500
	8	33,000	125.30		
				33,850	41,000
	9	34,700	125.70		
				33,350	50,000
	9:30	36,000	126.00		
				35,500	32,500
	11	35,000	125.80		
				35,500	40,000
	12:30	36,000	126.00		

Maximum Flood on Record

The maximum recorded flood flow is that of January, 1916. Information and data for the flood on this and other streams in Southern California are given in Water Supply Paper 426 of the United States Geological Survey on Southern California Floods of January, 1916.

This 1916 Flood was most severe in San Diego County. The maximum crest discharge on the San Dieguito River was 72,100 second feet on January 27th between 1:00 and 3:00 p.m. It was the result of extraordinary precipitation on the entire 300 square miles of watershed during January 25th, 26th and 27th, following a period of very large precipitation from January 14th to the 20th, which itself caused a crest discharge on January 18th of 37,600 second feet.

While the maximum crest discharge on January 27th is given as 72,100 second feet, this was no doubt somewhat larger than would have occurred had it not been for the flood wave created by the release of flood waters held back by the approach embankments of the Bernardo bridge, through the washing away of this bridge.

The discharge at stated times during the flood with the mean flow for the period between these times as obtained from data in the United States Geological Survey office are given in Columns 2 and 3 of Table 2.

Table No. 2

Flood Flow Discharge of San Dieguito River at Lake Hodges, January, 1916.

Date	Time	Discharge unregulated by storage, in second feet. For time given	Discharge unregulated by storage, in second feet. Mean for period	Discharge if regulated by storage: With present spillway Gauge Ht.	With present spillway Discharge	With improved spillway Gauge Ht.	With improved spillway Discharge
Jan.27	0:00 a.m.	2,700		116.8	2,700	116.8	2,700
			4,350				
	6:00 a.m.	6,000		117.0	3,300	117.0	3,300
			29,200				
	12:00 a.m.	52,400		122.1	20,000	122.1	22,000
			61,450				
	1:00 p.m.	70,500		124.1	28,000	123.9	31,200
			70,500				
	3:00 p.m.	70,500		127.1	41,000	126.55	45,800
			68,000				
	5:00 p.m.	65,500		128.9	50,000	128.0	54,000
			50,600				
	12:00 p.m.	35,700		128.95	50,400	127.4	50,400
			26,700				
Jan.28	6:00 a.m.	17,700		125.4	33,600	124.2	32,000

Comparison of 1916 and 1927 Floods

Although the 1916 flood produced a crest flow very much in excess of that of the 1927 flood, the daily discharge for the day of maximum flow as well as the aggregate flood flow for both floods were nearly the same, as indicated by the following data:

Daily Discharge of San Dieguito River at Hodges in Acre Feet

February 15, 1927,	18,486	January 26, 1916,	5,850
February 16, 1927,	73,529	January 27, 1916,	73,800
February 17, 1927,	11,706	January 28, 1916,	29,770
February 18, 1917,	2,777	January 29, 1916,	9,420
Total for 4 days	106,498		118,840

The following crest flow data on the two main tributaries, and the flow of the main stream at Hodges show that a very large rate of run-off was obtained from all parts of the watershed, both in 1916 and 1927.

January 27, 1916

	Santa Ysabel near Ramona	Santa Maria Creek	Santa Ysabel and Santa Maria	San Dieguito at Hodges
Crest flow in second feet	28,400	7,140	35,540	72,100
Area of Watershed in square miles	110	57.3	167.3	300
Crest flow in second feet per square mile	258	124	212	240

February 16, 1927

	Santa Ysabel near Ramona	Santa Maria Creek	Santa Ysabel and Santa Maria	San Dieguito at Hodges
Crest flow in second feet	19,401	6,318	25,719	50,000
Area of Watershed in square miles	110	57.3	167.3	300
Crest flow in second feet per square mile	176	110	149	167

Water Supply Paper No. 426 of United States Geological Survey, on Page 12, gives the daily rainfall for a large number of stations in the San Dieguito basin, during the storm which produced the 1916 flood. These data show that the extraordinary precipitation was quite uniformly distributed over the entire 300 square miles of the watershed, and explain, in part at least, the large rate of run-off from all parts of the watershed.

Other Major Floods

Water Supply Paper No. 426, on Page 35, states: "To determine whether the flood of 1916 was more or less severe than previous floods in Southern California, a search was made of the early records and many old residents of the County were interviewed." The results of this investigation are given on Pages 35 to 40 of the Paper, and while they are necessarily based on data and statements, which are not and cannot be substantiated by measurements of flow, they are the best obtainable. From this information major floods occurred on the San Dieguito River, probably in the following order of magnitude: 1916, 1862, 1825, 1833, 1884, 1891, 1927 and 1895.

A flood with a crest flow of that of 1927 or about 50,000 second feet, has a frequency of about once in fifteen years; and a flood with a crest flow of that of 1916 or about 70,000 second feet, has a frequency of about once in one hundred years.

From a consideration of the above data, of flood frequency curves derived in connection with the Investigation of Water Resources of California, by the State Department of Public Works, Division of Engineering and Irrigation, and of flood flow data and equations, we believe that floods in excess of that of 1916 will occur at rare intervals, and that a flood having a crest of approximately 90,000 second feet may occur with a frequency of once in five hundred to one thousand years.

To insure a proper degree of security against the dam being overtopped, the spillway should be improved to discharge the outflow from a flood of this magnitude.

Capacity of Spillway

The best information on the present capacity of the spillway is the Spillway Rating Curve drawn by I.F. Judy, which is reported by J.W.Williams of the Bureau of Water Supply of the City of San Diego, to be based upon observations and computations by F.E. Green. From this curve the rates of discharge for different gauge heights, and the corresponding values of C in the weir formula $Q = CLH^{3/2}$, for a value of $L = 350$, are as follows:

Gauge Elev. of Water Surface	Height Head on Spillway Crest	Discharge in cubic feet per second	Corresponding Value of C in formula $Q = CLH^{3/2}$
116	1	1,280	3.66
117.	2	3,640	3.68
118	3	6,530	3.62
119	4	9,061	3.28
120	5	12,500	3.20
121	6	16,000	3.11
122	7	19,880	3.06
123	8	23,560	2.99
124	9	27,620	2.93
125	10	31,845	2.88
126	11	36,150	2.84
127	12	40,490	2.78
128	13	45,200	2.76
129	14	51,000	2.76
130	15	56,000	2.75

The above results indicate that for gauge heights in excess of 3 feet on the crest of the spillway, the conditions for maximum discharge become progressively unfavorable with an increase in gauge height. This is due largely to the very poor condition of the channel of approach to the spillway. It is our opinion that the spillway can be improved so as to give a coefficient of C in the weir formula equal to 3.30.

Crest Flow at Spillway for 1916 Flood

The outflow or flow at the spillway has been computed for the spillway in its present condition, and also for the spillway improved to give a coefficient of C in the weir formula equal to 3.30. The inflow and the outflow for both conditions of spillway are given in Table 2. The results show the following:

1. For the present spillway the crest flow is reduced by storage to a maximum outflow of 50,400 second feet, with a gauge height of 128.95 feet, or a water level 1.05 feet below the top of the dam.
2. For the improved spillway the crest flow is reduced to 54,000 second feet, with a gauge height of 128.0 feet of a water level 2.0 feet below the top of the dam.

Crest Flow of Spillway for a Crest Inflow of 90,000 Second Feet

Computations were made for a flood of the same characteristics as that of 1916, but with the quantities for the periods of time during the flood increased in the proportion of 90,000 to 70,500. The results for the improved spillway are given in Table 3, and show that the inflow crest is reduced from 90,000 second feet to 67,500 second feet, with a corresponding gauge height of 130.05 feet, which brings the maximum water level practically to the top of the dam.

Table No.3

Flood Flow Discharge of San Dieguito River at Lake Hodges for a flood crest of 90,000 second feet with same characteristics as that of the 1916 flood

From computations based on Spillway Discharge $Q = 3.30\ L\ H^{3/2}$ (L = 350)

Date	Time	Discharge un-regulated by storage in sec.ft. For time given	Mean for period	Discharge regulated by storage, in sec. ft. Gauge Ht.	Discharge
Jan.27	0:00 a.m.	3,400		117.1	3,400
			5,500		
	6:00 a.m.	7,700		117.6	4,800
			37,300		
	12:00 a.m.	67,000		123.8	30,500
			78,500		
	1:00 p.m.	90,000		125.55	39,800
			90,000		
	3:00 p.m.	90,000		128.7	59,000
			86,800		
	5:00 p.m.	83,600		130.05	67,500
			64,600		
	12:00 p.m.	45,600		129.8	66,700
			34,100		
Jan.28	6:00 a.m.	22,600		125.6	52,750

Spillway Improvements

The necessity for spillway improvement has been referred to above.

The spillway crest consists of 175 feet of ogee rollway crest continued with a broad crest about 35 feet wide and 185 feet in length, constructed on a bench excavated into the side hill at the north end of the dam. The approach to the northerly 10 feet of the shore end of the broad crest spillway is essentially obstructed by the end or abutment wall; so that the total effective length of the spillway crest is about 350 feet.

The conditions which are unfavorable to maximum discharge over the spillway are:

1. A rocky spur upstream from the north end of the spillway projects out into the reservoir toward the spillway and dam so as to very materially contract the channel approach toward the spillway. (See Plate 11).
2. The flat crest spillway extension is excessively wide and does not have a smooth sloping floor approaching the crest.
3. The spillway channel downstream from the northerly end of the spillway may not allow the water to escape freely.

The only topographic map of the spillway area is shown by Plate 2. It is not sufficiently detailed or comprehensive to specify in detail the work which should be done to form an adequate spillway.

In general the work necessary to increase the capacity of the spillway includes the following:

1. Remove the rocky spur and excavate enough material to form a straight, unobstructed channel of approach of sufficient depth and width, gradually decreasing in cross section from the body of the reservoir to the spillway, with no projecting points.
2. Shape the floor of this upstream channel where it joins the upstream edge of the broad crested weir, to a relatively smooth surface on a slope of about 1 to 1, rounding off the connection between the two.
3. Remove any obstruction in the upper portion of the downstream spillway channel, which may prevent a free getaway of the flood waters discharged over the spillway.

After the above improvement work has been done, the spillway should be rated when the opportunities will make it possible to do so. We believe that it will be feasible to increase its capacity to at least that used in the computations referred to above.

In addition to the work necessary to increase the spillway capacity, it is necessary to protect the downstream channel of the spillway against the displacement and washing away of the loose or fractured rocks forming the bed of the spillway. The 1927 flood demonstrated that relatively large blocks of rocks can be displaced and carried away. Considerable repair and protection work was done following this flood.

This channel is now apparently able to resist the action of relatively large floods. No doubt floods of equal or larger magnitude will cause the washing away of some additional material. This action should be watched, and following such floods, holes and cracks should be filled with concrete until a stable condition is reached.

Effect of Sutherland Reservoir on Flood Flows at Lake Hodges

Because of the uncertainty of the completion of the Sutherland Dam, we have not considered the possible effect of the storage of this reservoir. Should this dam be constructed, it is probable that the storage created thereby would reduce the magnitude of the flood crest at Lake Hodges. The extent of the reduction can only be determined by detailed computations. That it would probably be material is indicated by the fact that the crest discharge of Santa Ysabel Creek at the Sutherland site on January 27, 1916, was 21,000 second feet, or approximately 30 per cent of the total crest discharge at Lake Hodges.

Conclusions

1. Lake Hodges dam is safe to carry the loads, excepting in the case of a major earthquake, provided that the spillway is improved to prevent overtopping.

2. Earthquakes of major intensity occurred in 1812 and again in 1857. There is reason to believe that Lake Hodges dam will at some future time doubtless be subjected to similar earthquakes. To resist such earthquakes the dam must be reinforced by thorough bracing of the buttresses.

3. The spillway capacity is barely sufficient to discharge a flood of the magnitude of that of 1916. While this flood is the maximum since the time stream flow measurements have been made, and floods of this magnitude would probably not recur with a frequency greater than about once in one hundred years, floods of greater intensities will occur.

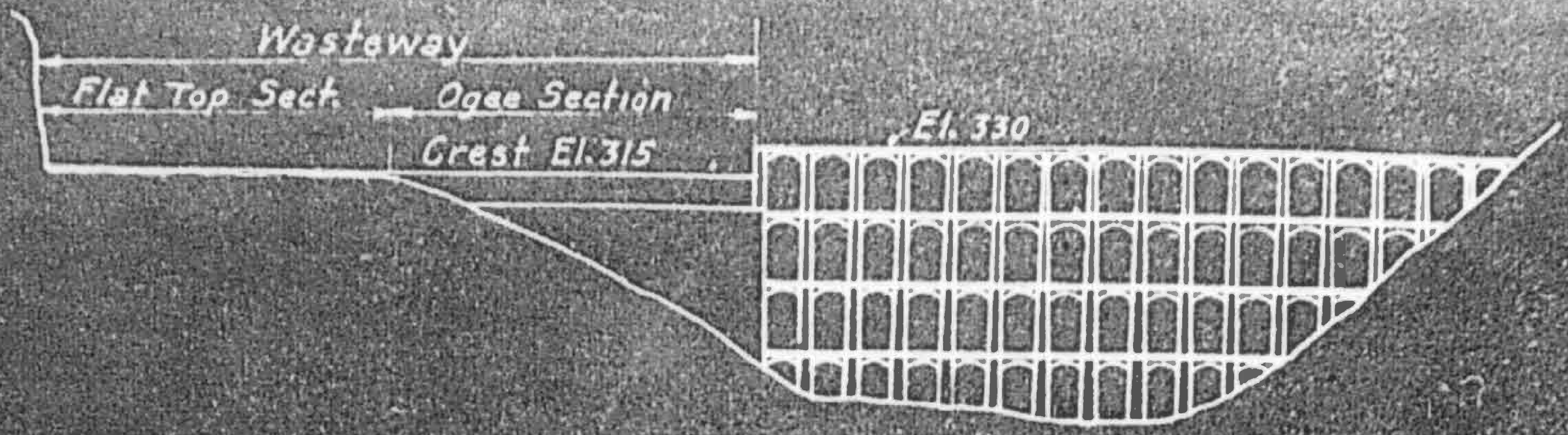
We recommend that in order to adequately and safely discharge flood flows of the magnitude of that of 1916, and to prevent overtopping of the dam by larger floods, the spillway be improved in the general manner suggested by us.

Respectfully submitted,

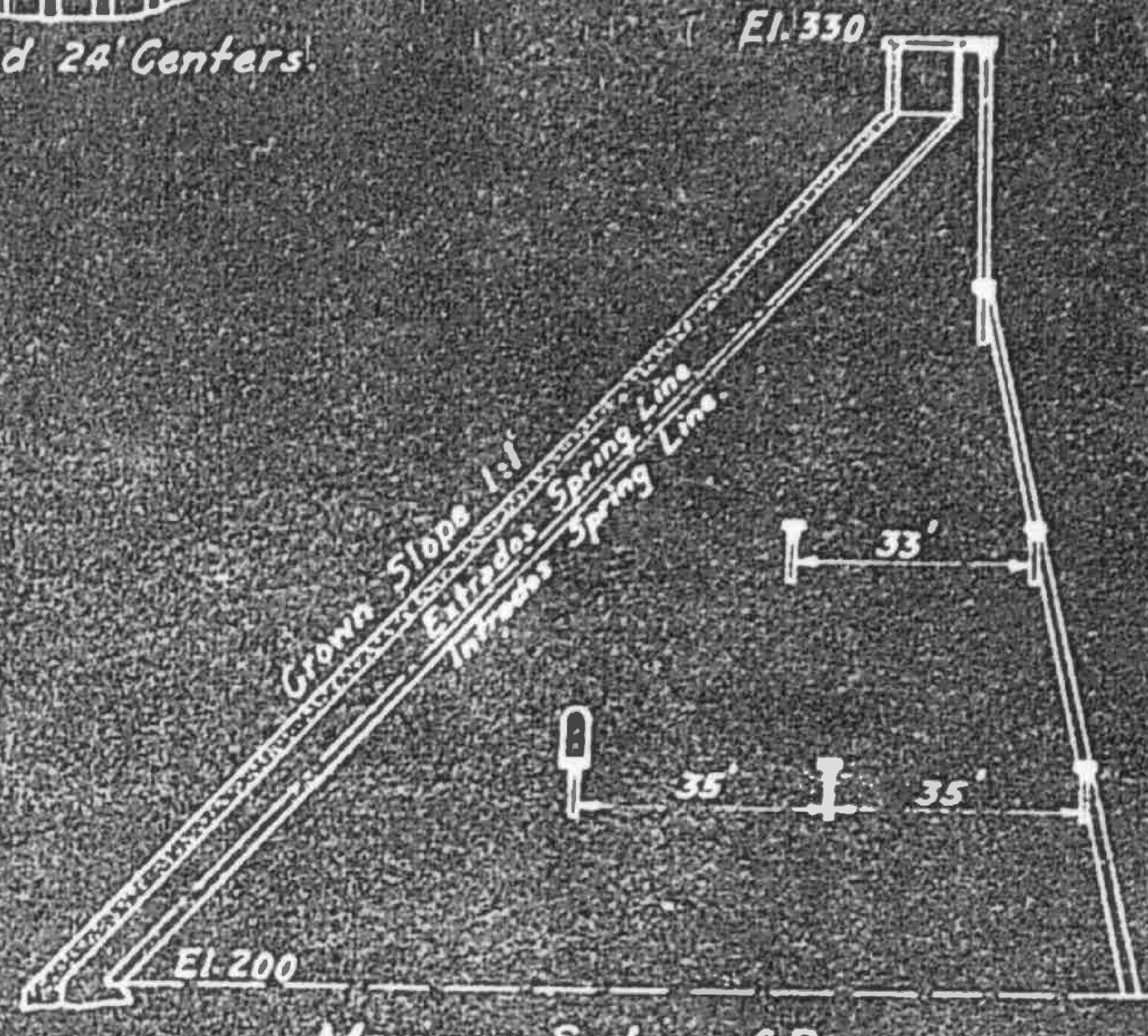
(Signed) F. C. Herrmann

(Signed) B. A. Etcheverry

(Signed) A. Kempkey



Rear Elevation

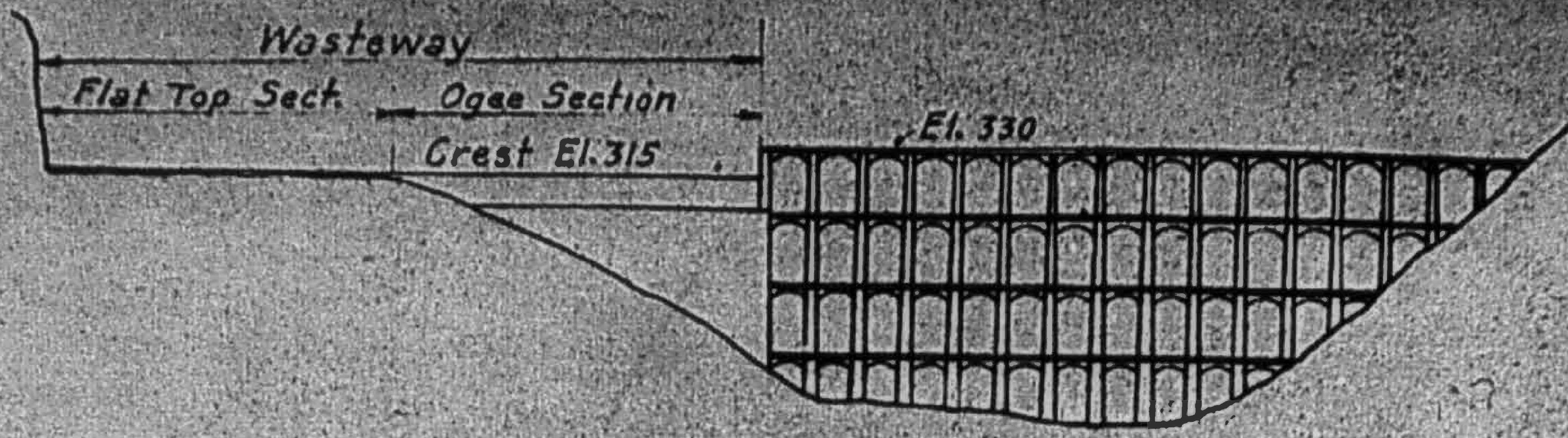


Maximum Section of Dam.

LAKE HODGES DAM
SECTION & REAR ELEVATION

To Accompany Report of
F.C. Herrmann
B.A. Etcheverry
A. Kempkey
to State Engineer.
March. 1929.

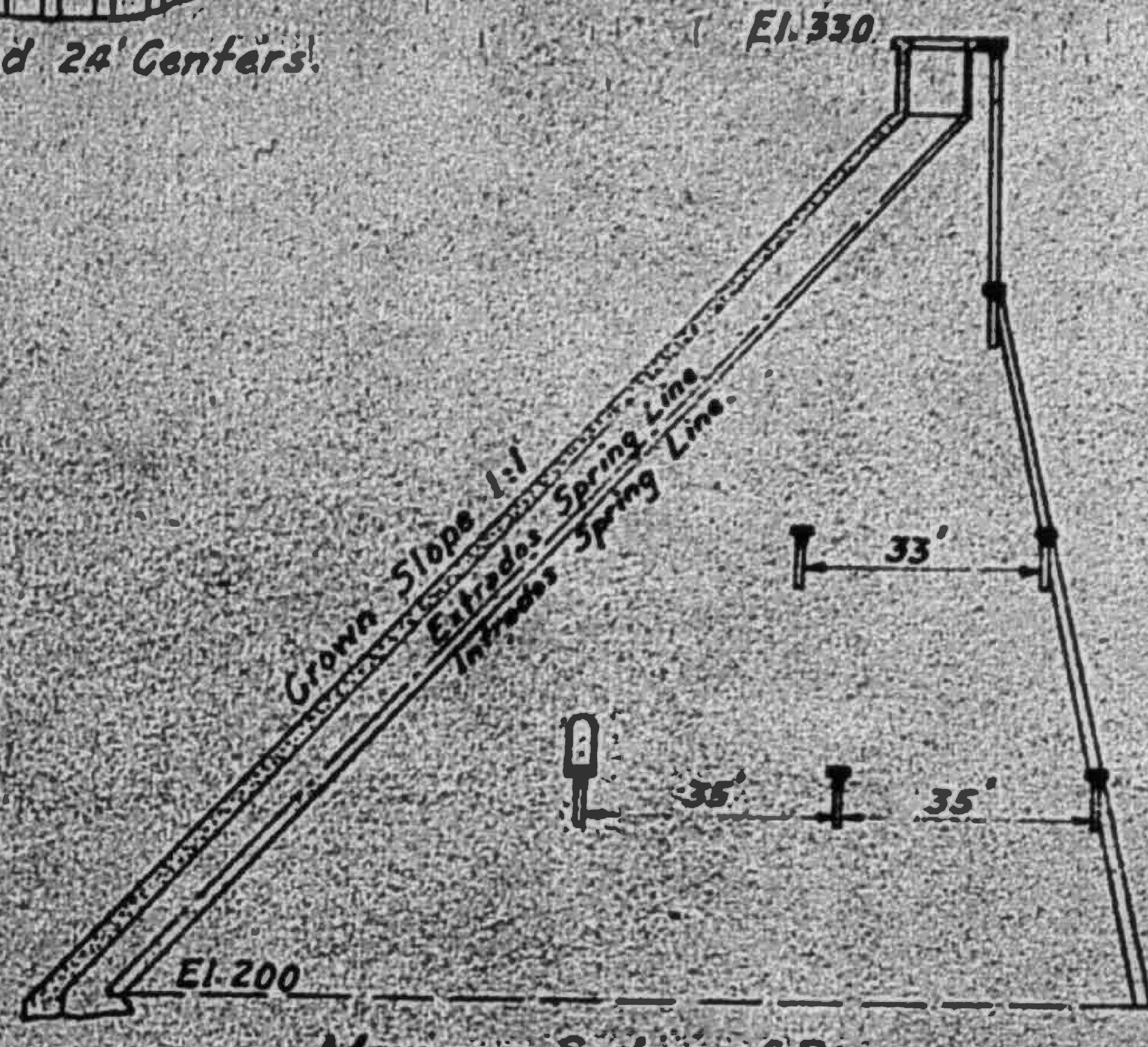
PLATE I



Rear Elevation

LAKE HODGES DAM
SECTION & REAR ELEVATION

To Accompany Report of
F.C. Herrmann
B.A. Etcheverry
A. Kempkey
to State Engineer.
March 1929.



Maximum Section of Dam.

PLATE I

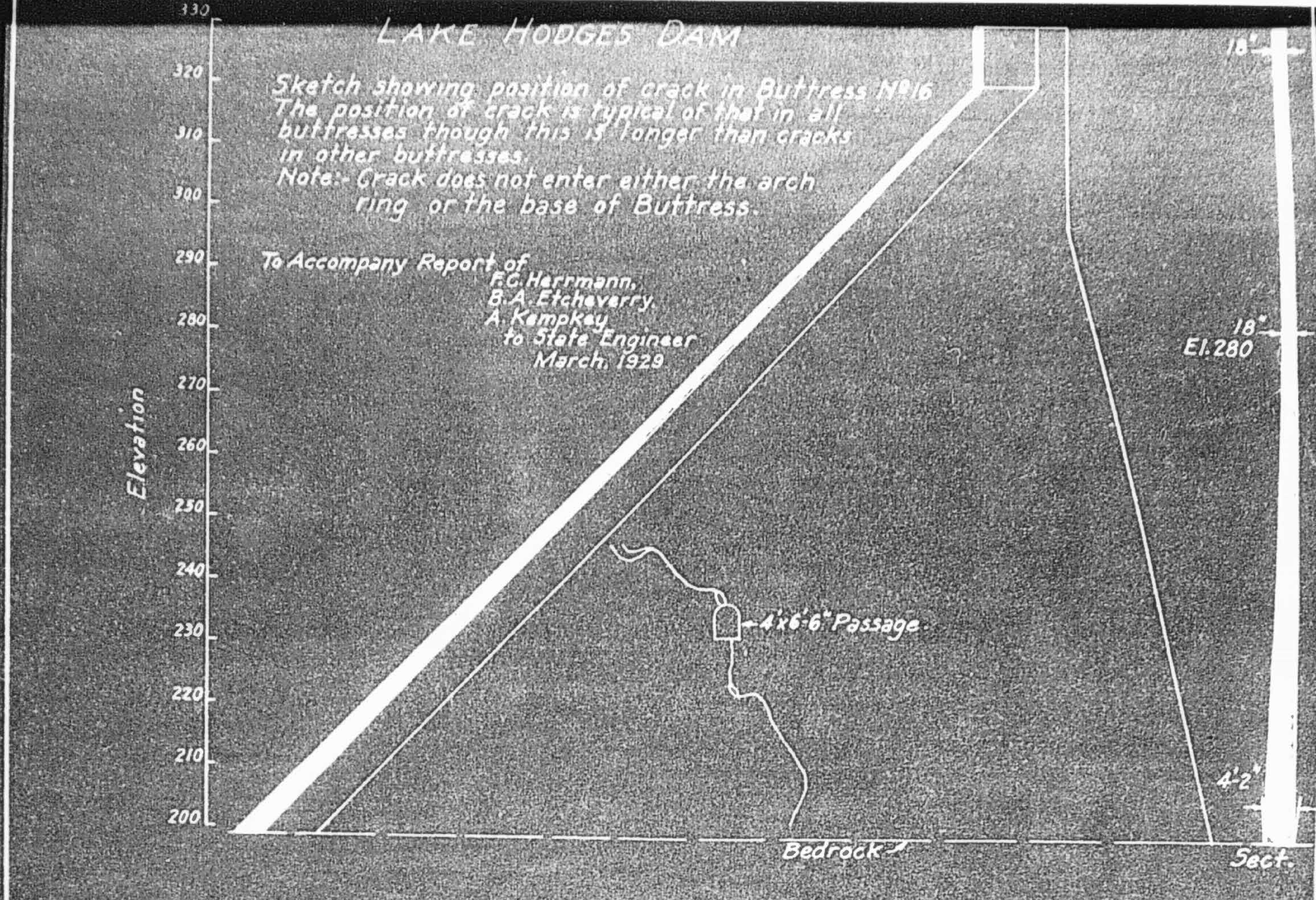
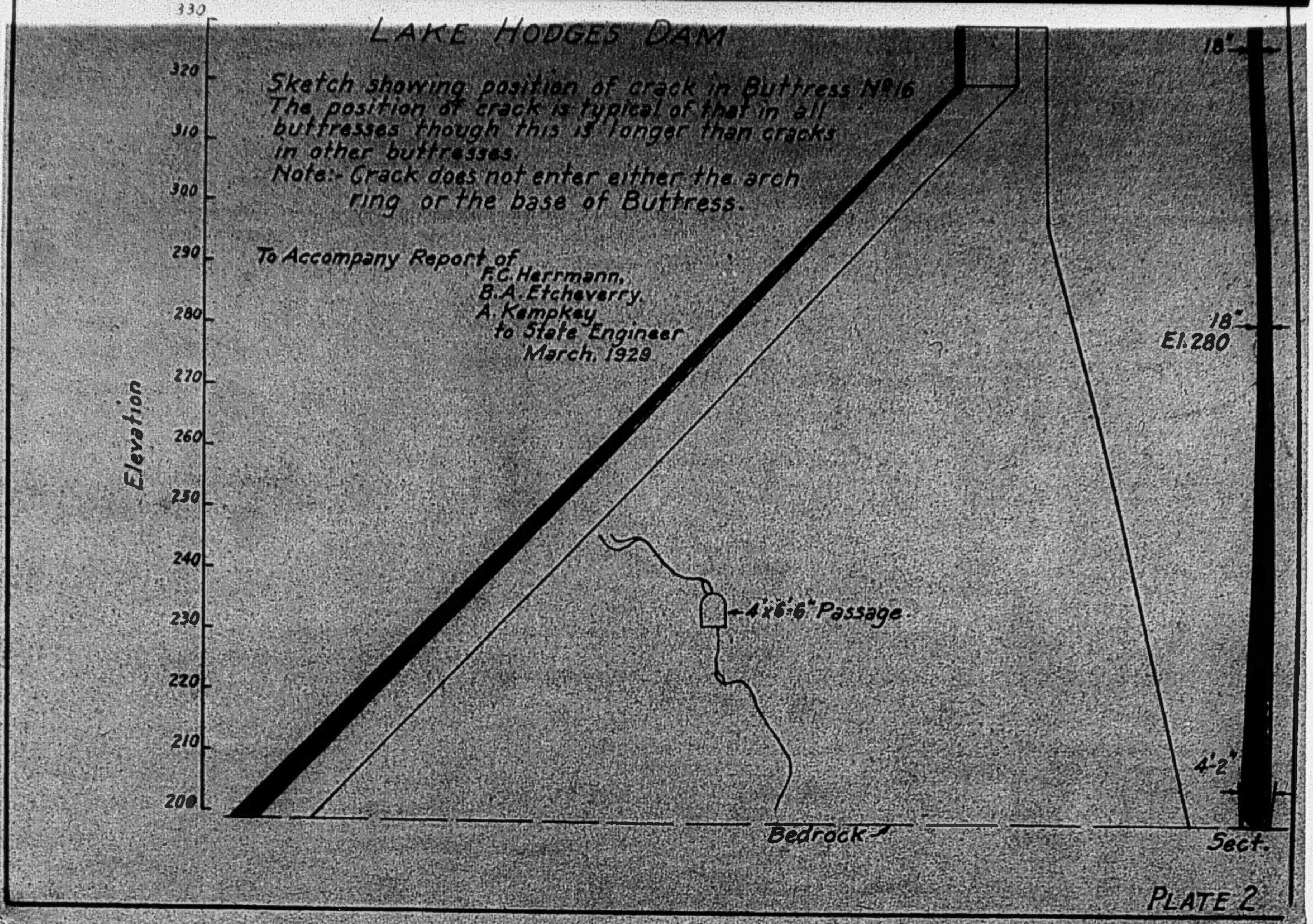

LAKE HODGES DAM
Sketch showing position of crack in Buttress No 16
The position of crack is typical of that in all buttresses though this is longer than cracks in other buttresses.
Note:- Crack does not enter either the arch ring or the base of Buttress.
To Accompany Report of
F.C. Herrmann.
B.A. Etcheverry.
A. Kempkey
to State Engineer
March. 1929
330
320
310
300
290
280
270
260
250
240
230
220
210
200
Elevation
4'x6'-6" Passage.
Bedrock
18"
18"
El. 280
4'-2"
Sect.

PLATE 2


LAKE HODGES DAM
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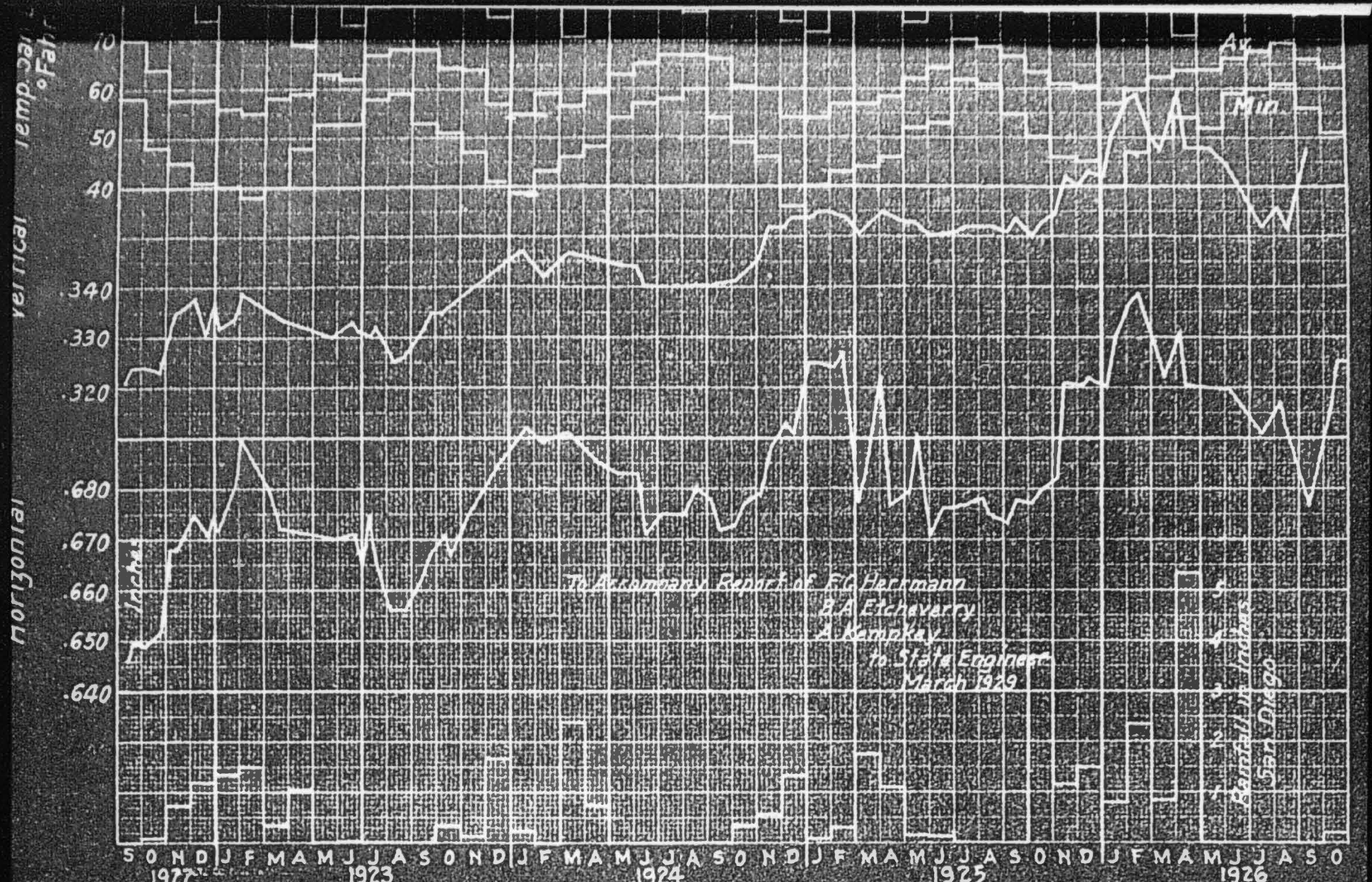
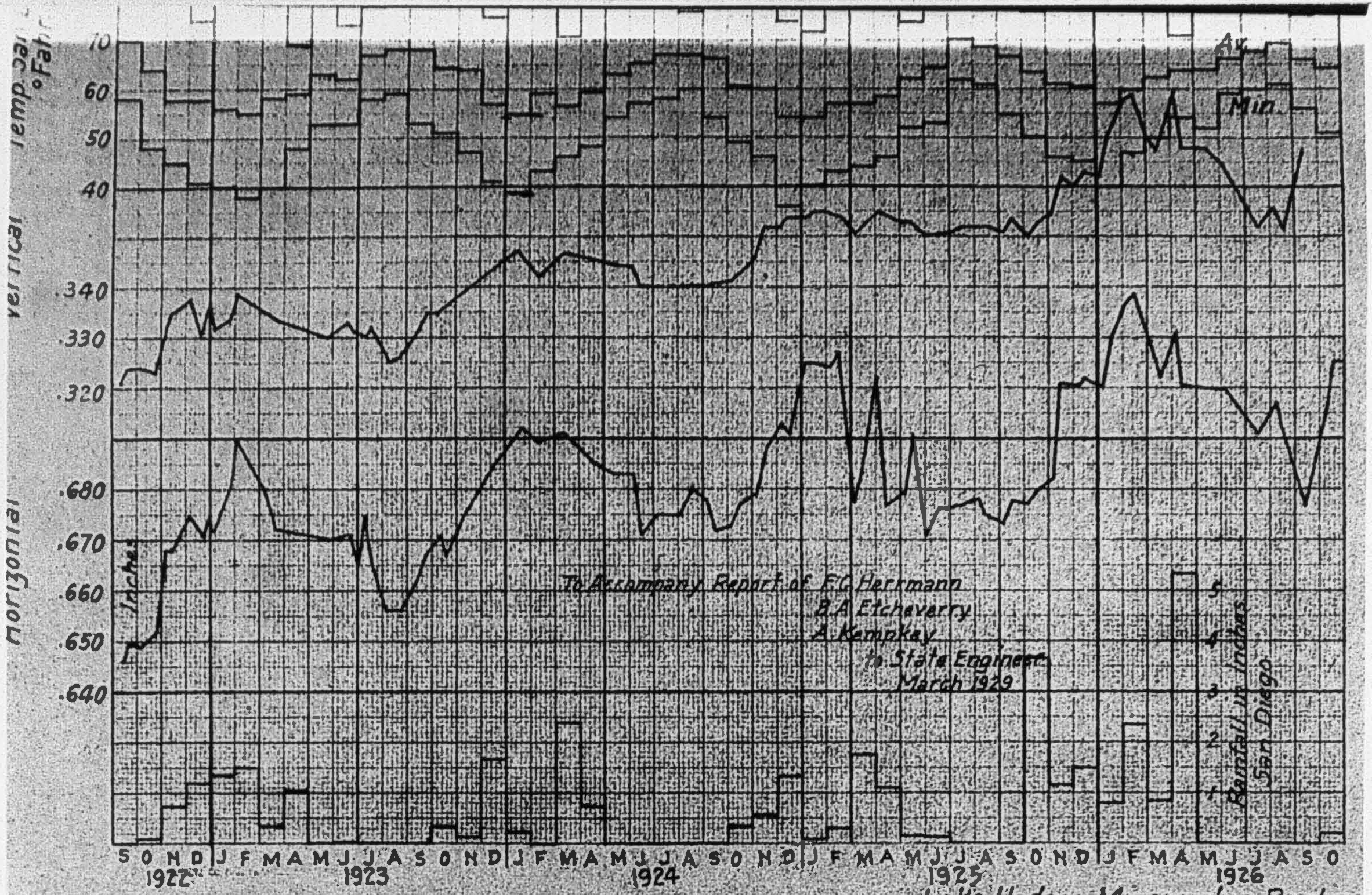
The position of crack is typical of that in all buttresses though this is longer than cracks in other buttresses.
Note:- Crack does not enter either the arch ring or the base of Buttress.
To Accompany Report of
F.C. Herrmann,
B.A. Etcheverry,
A. Kempkey
to State Engineer
March, 1929
Elevation
330
320
310
300
290
280
270
260
250
240
230
220
210
200
4'x6'6" Passage
Bedrock
18"
18"
El. 280
4'2"
Sect.
PLATE 2



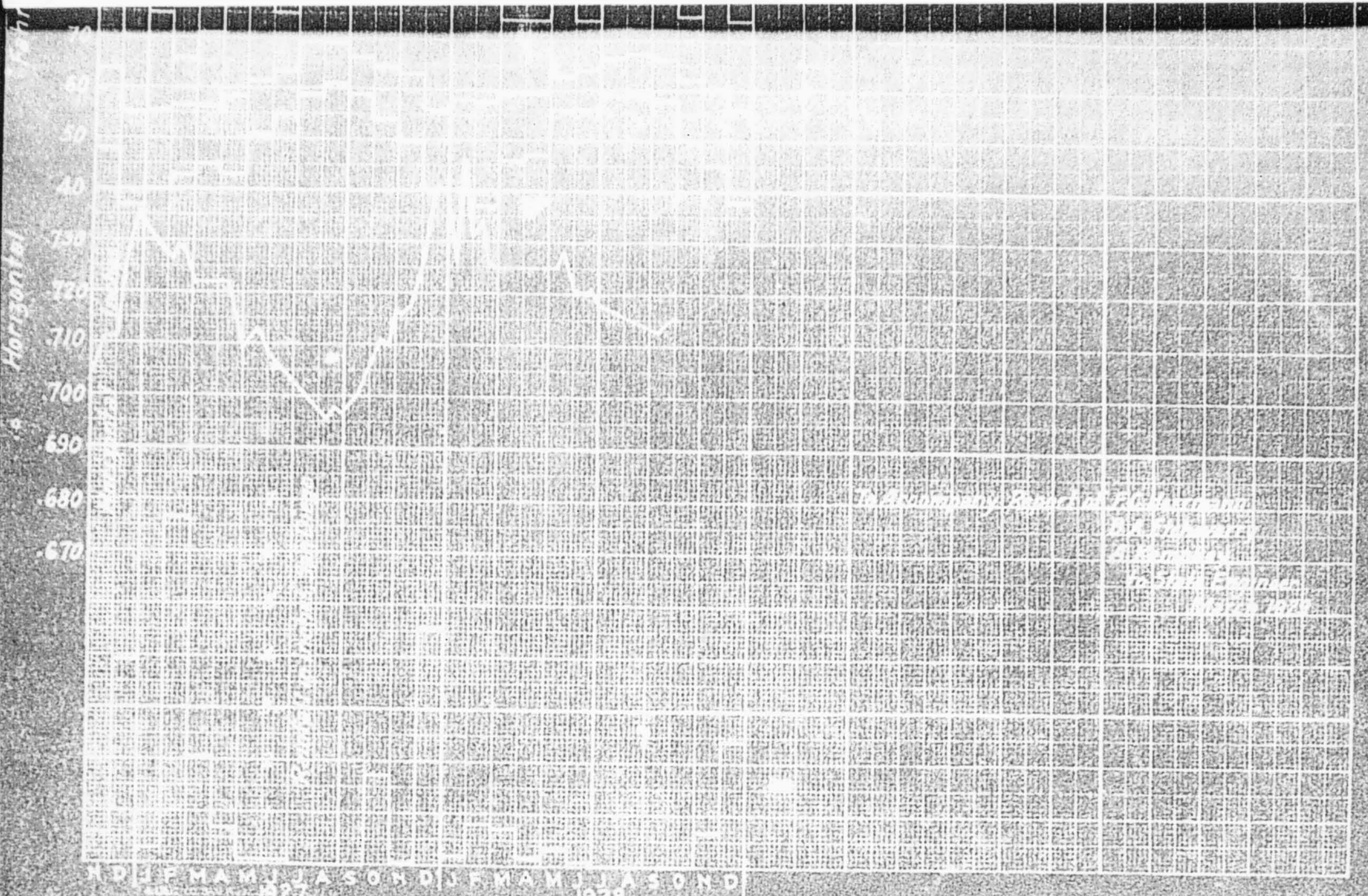
Lake Hodges Micrometer Reading
Buttress No. 15 South Side.

PLATE 3



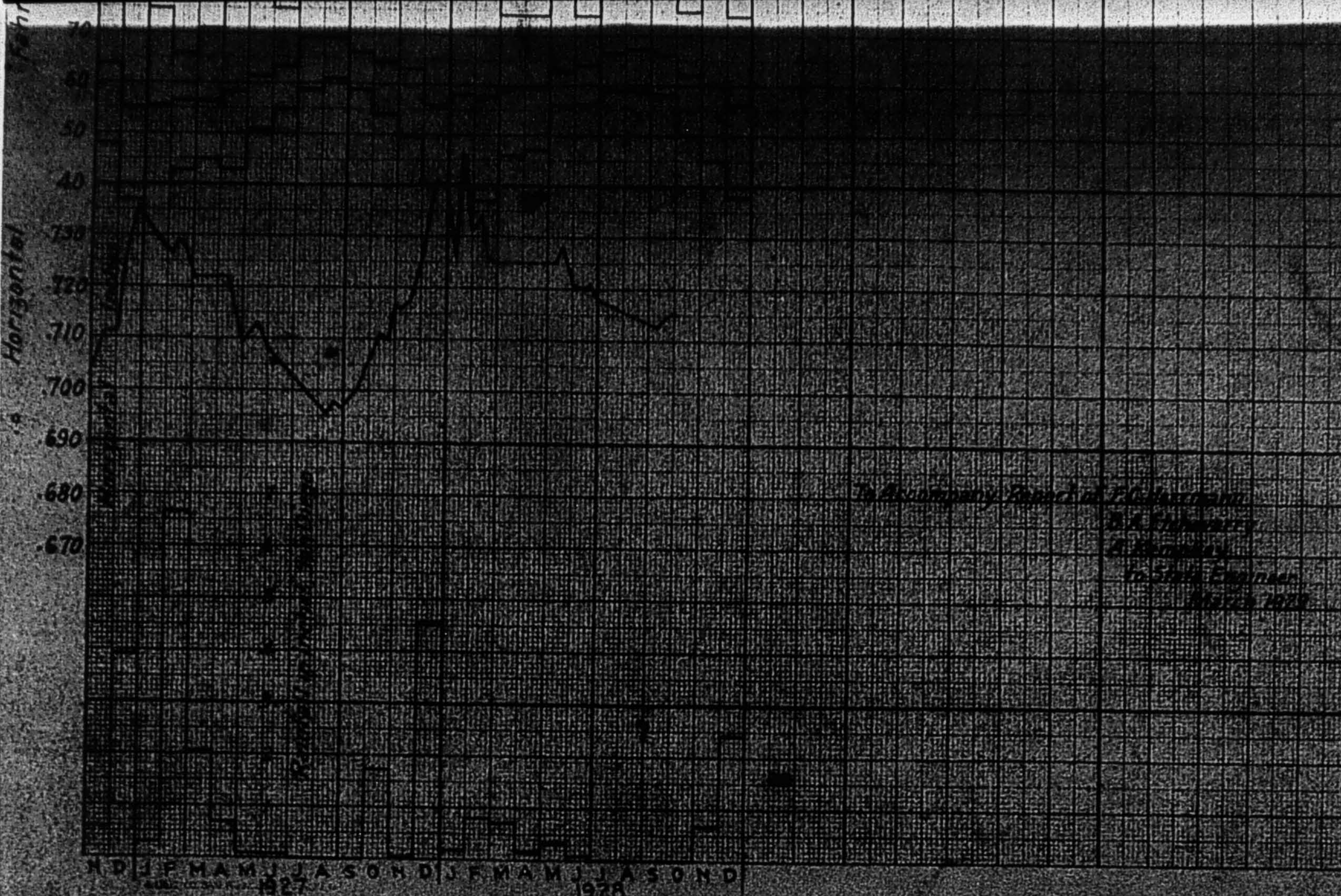
Lake Hodges Micrometer Reading
Buttress No. 15 South Side.

PLATE 3



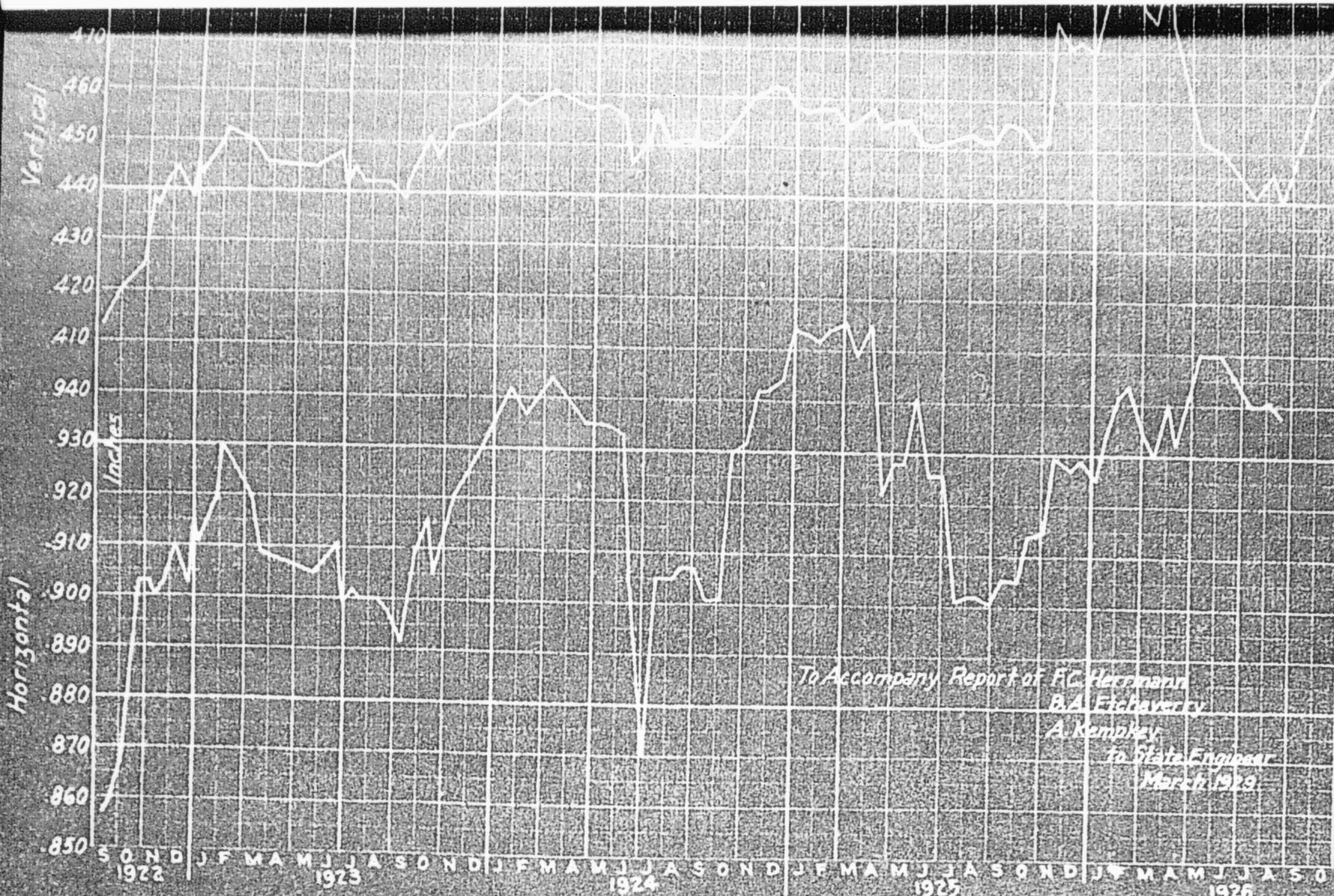
Lake Hodges Micrometer Readings
Buttress No 15 South Side

PLATE 3 A.



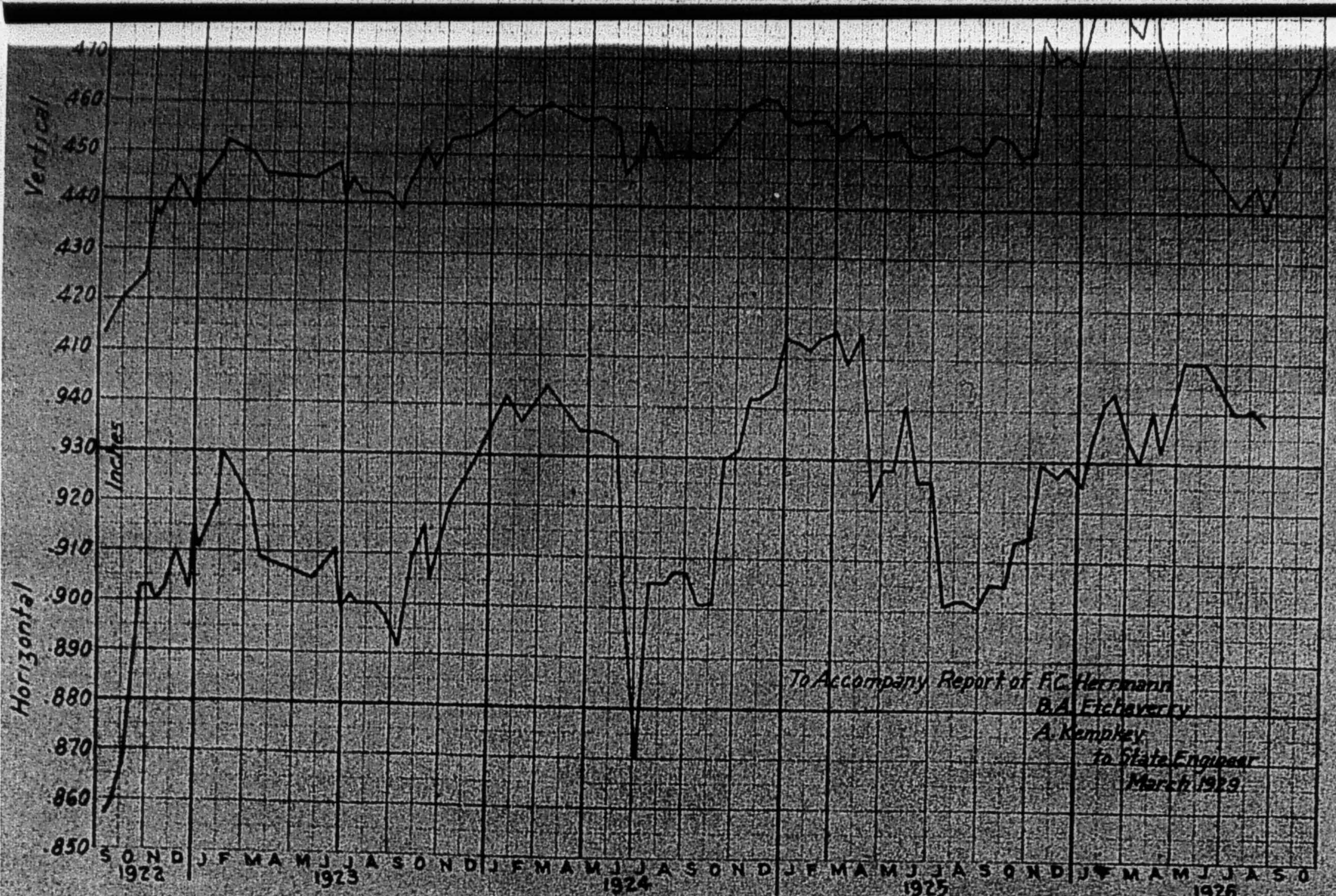
Lake Hodges Micrometer Readings
Buttress No 15 South Side

PLATE 3 A.



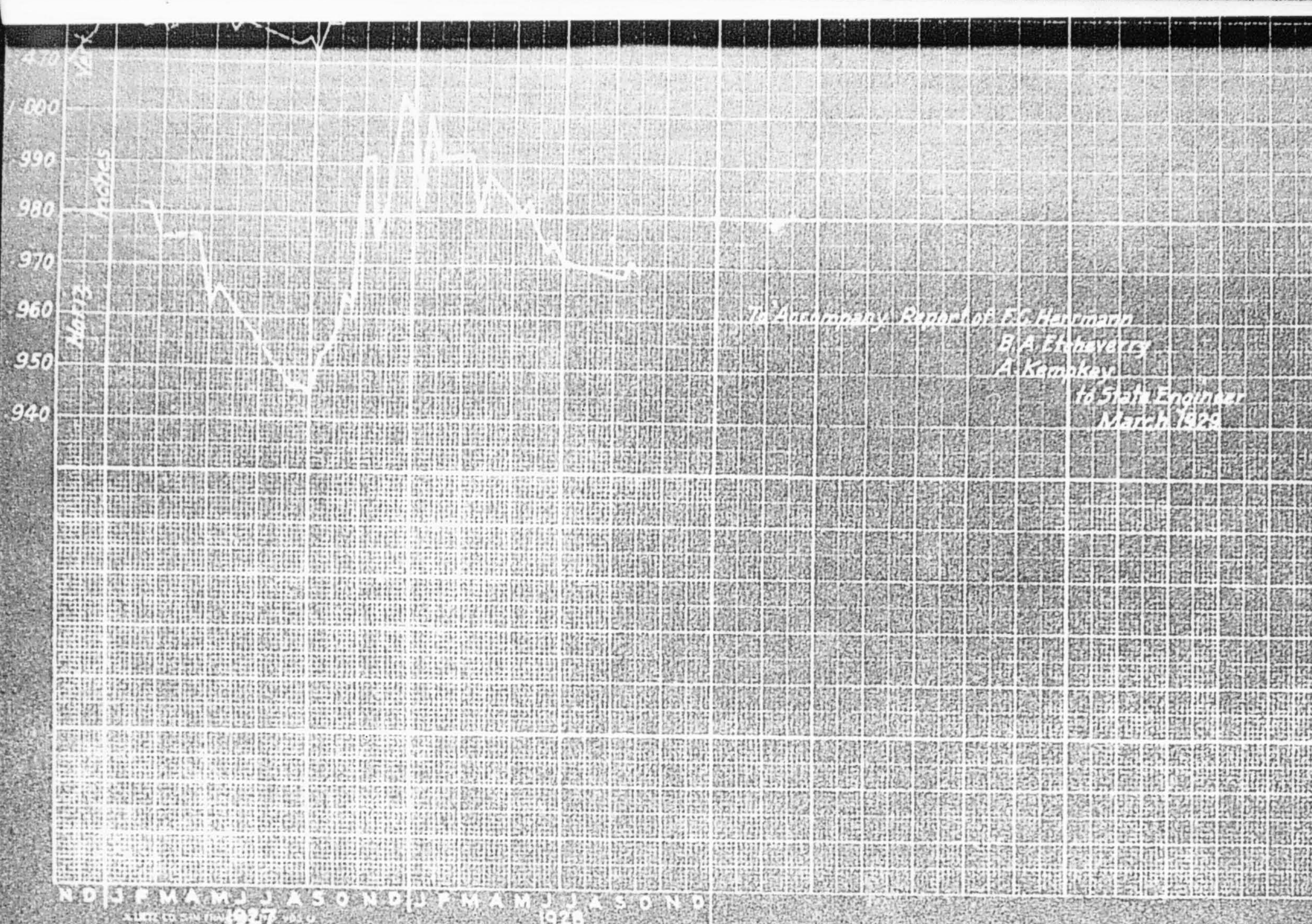
Lake Hodges Micrometer Readings
Buttress No. 16 South Side

PLATE 4



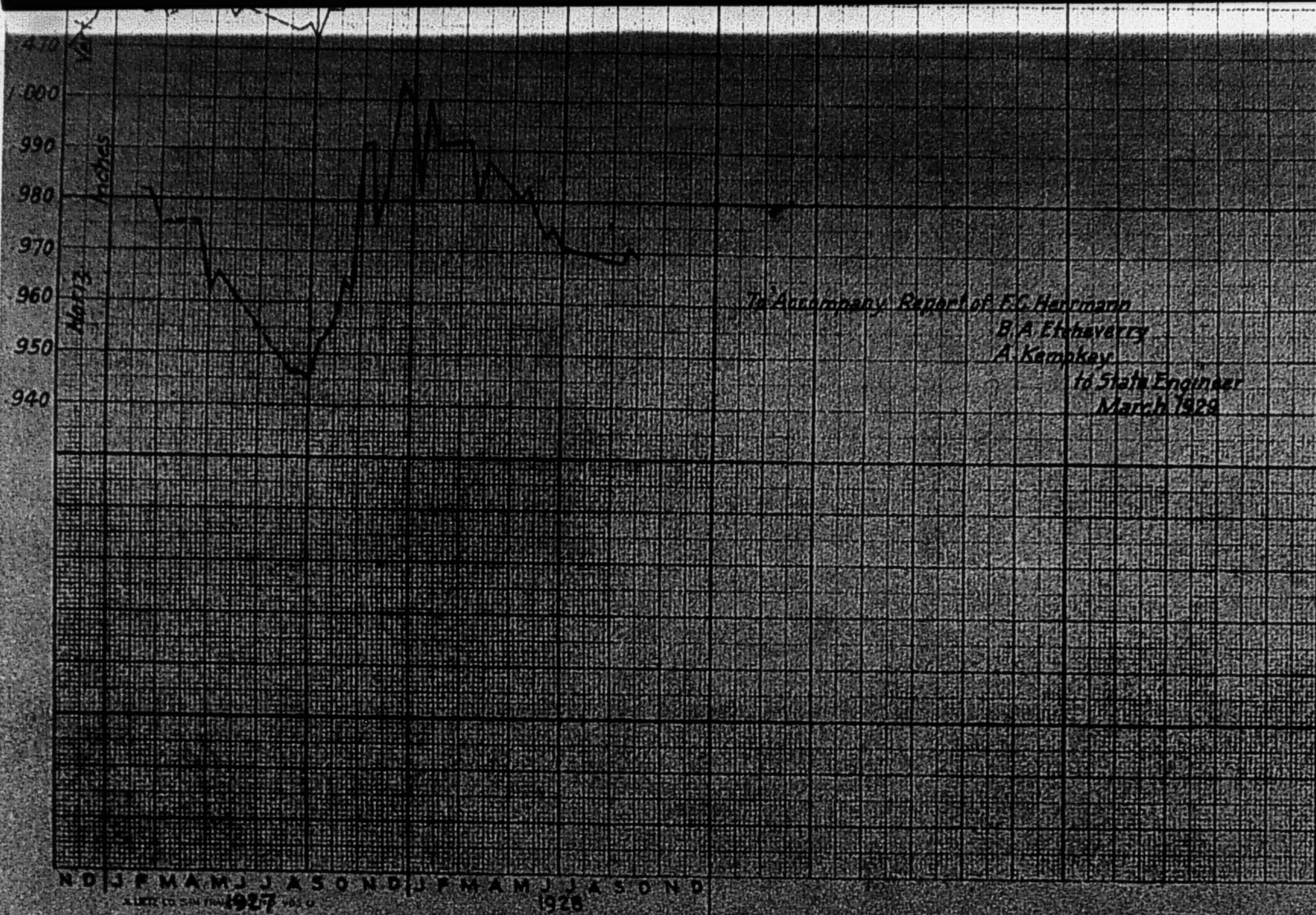
Lake Hodges Micrometer Readings
Buttress No. 16 South Side

PLATE 4



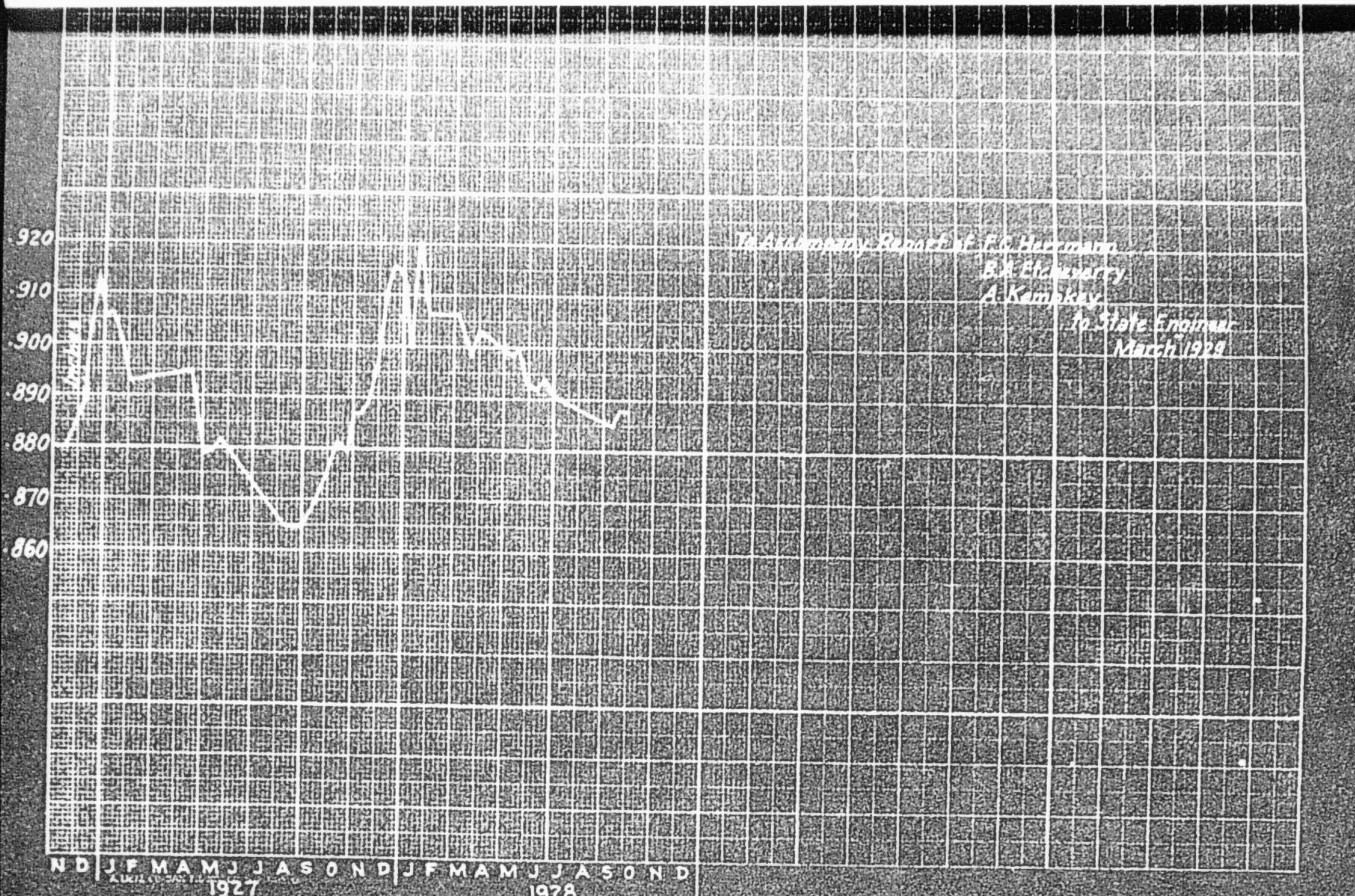
Lake Hodges Micrometer Readings
Buttress No. 16 South Side

PLATE 4A

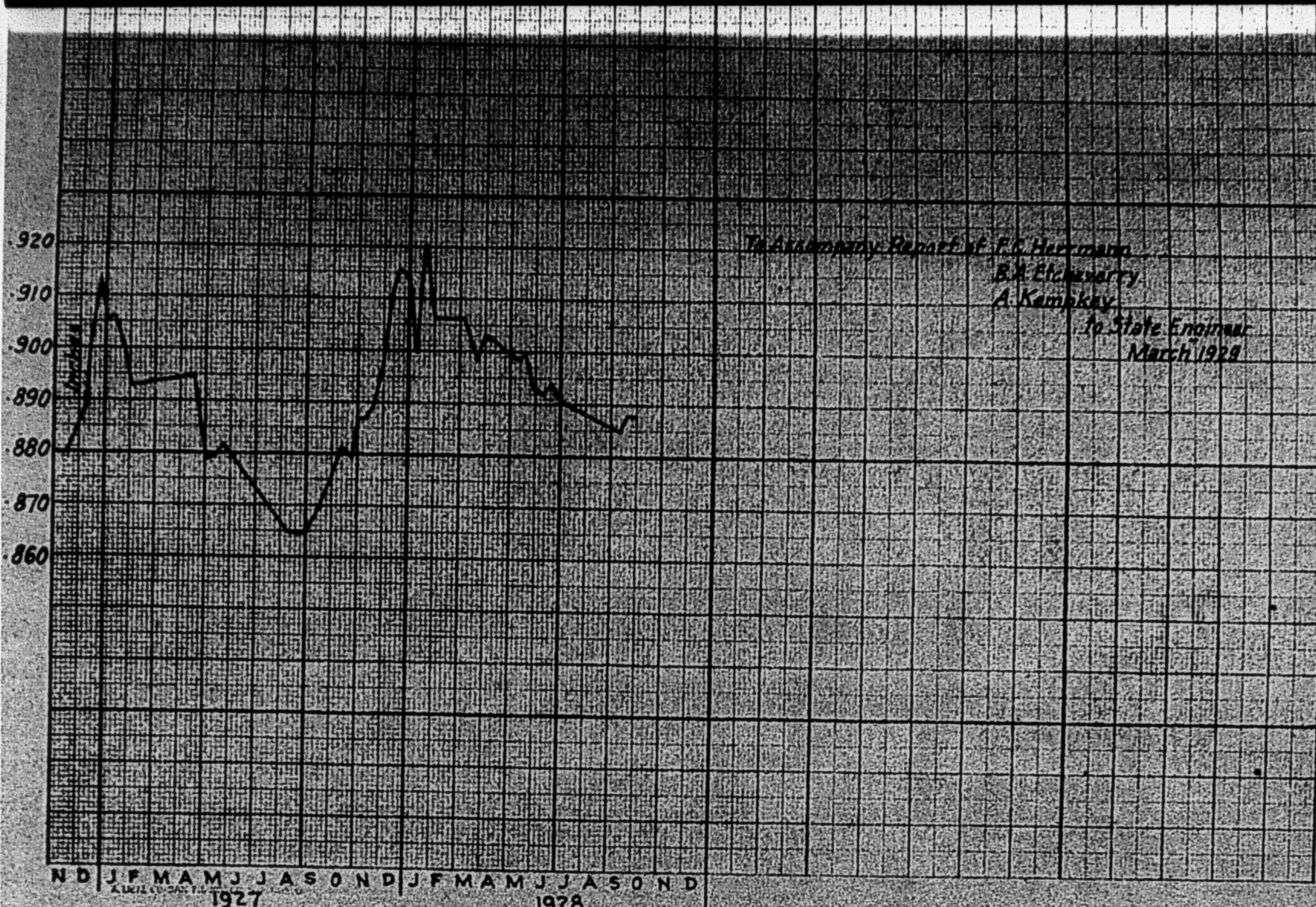


Lake Hodges Micrometer Readings
Buttress No. 16 South Side

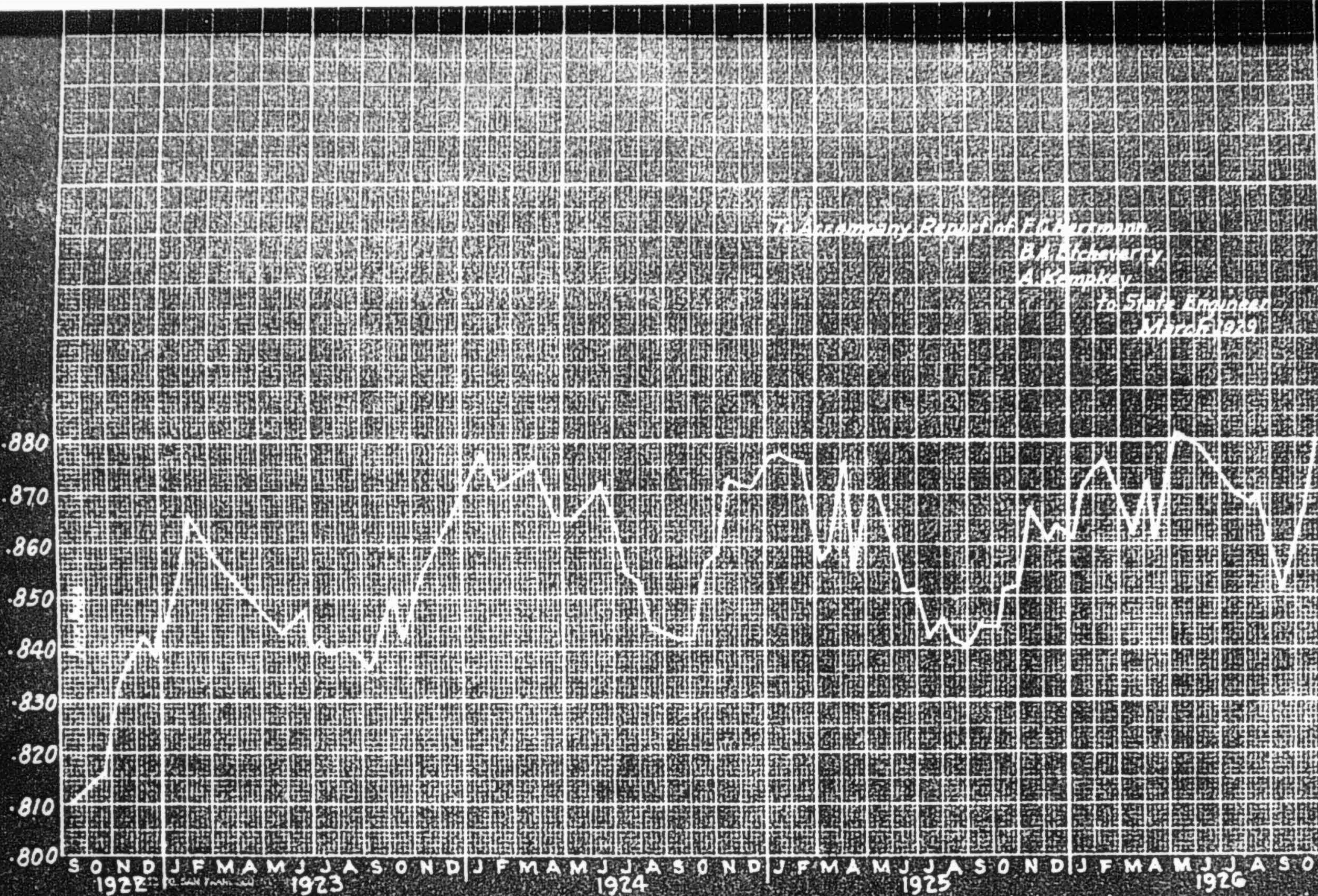
PLATE 4A



Lake Hodges Micrometer Readings
Buttress No. 17 North Side.
PLATE 5

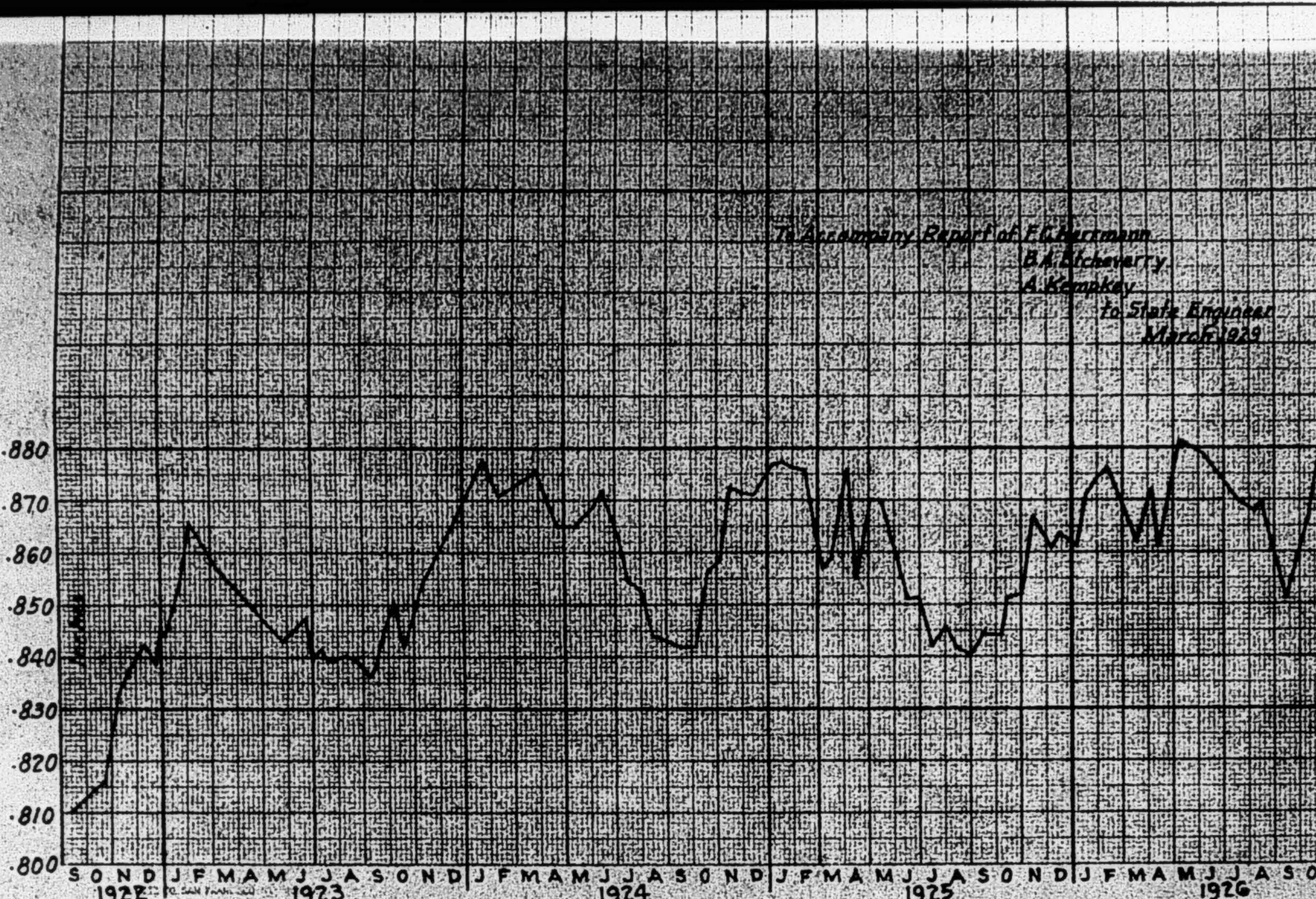


Lake Hodges Micrometer Readings
Buttress No. 17 North Side.
PLATE 5



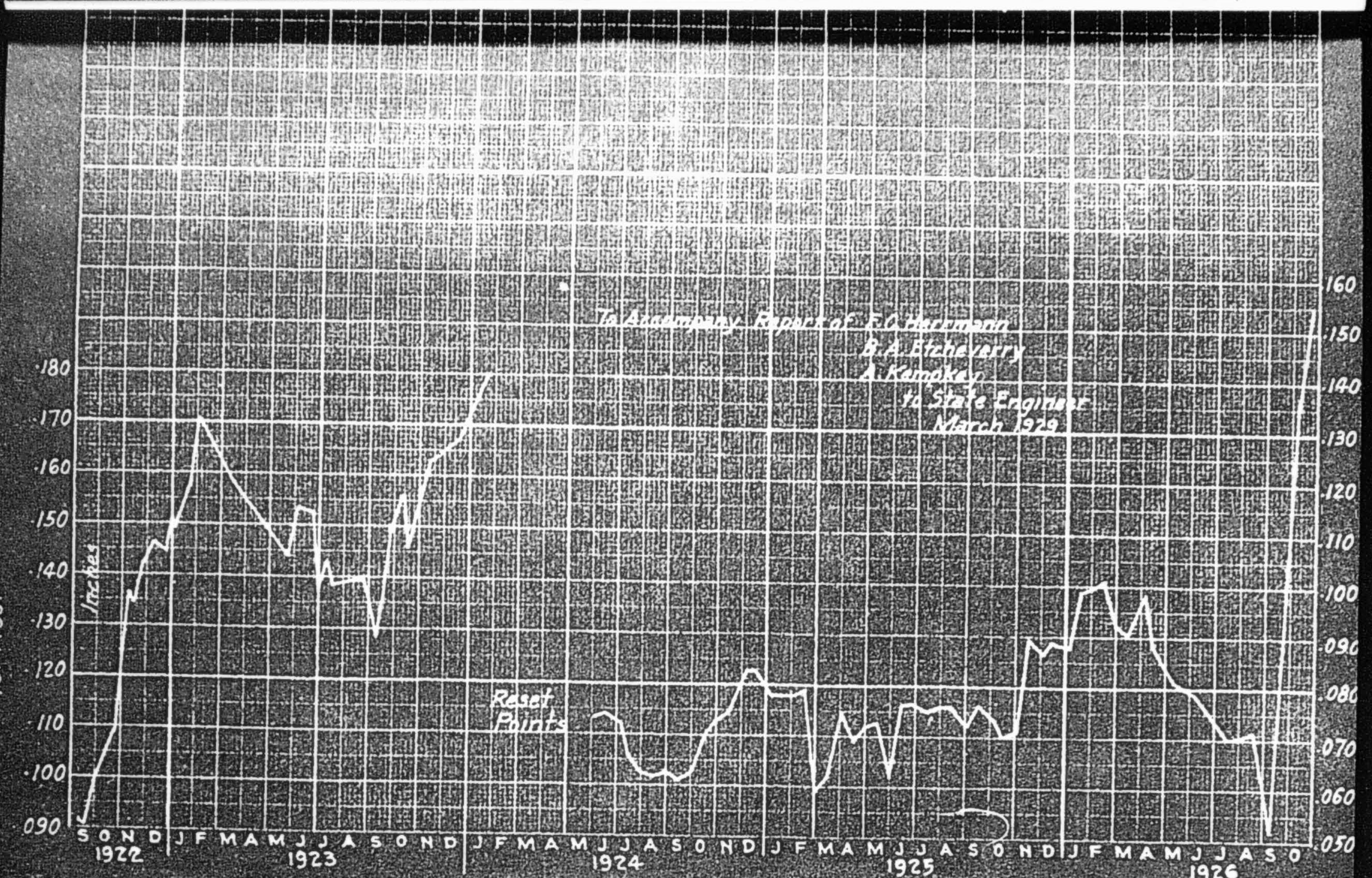
Lake Hodges Micrometer Readings
Buttress No. 17 North Side

PLATE 5A



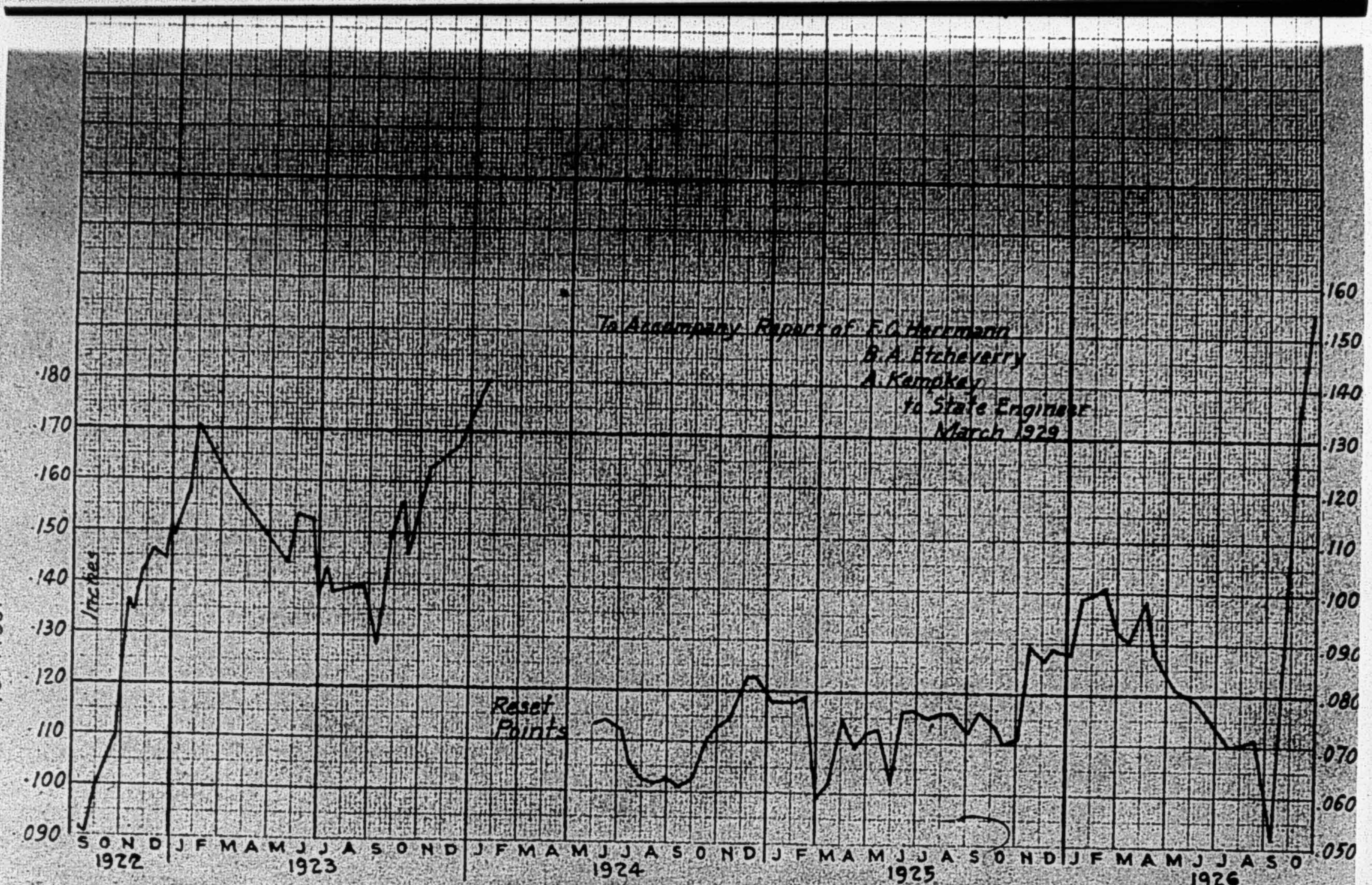
Lake Hodges Micrometer Readings
Buttress No. 17 North Side

PLATE 5A



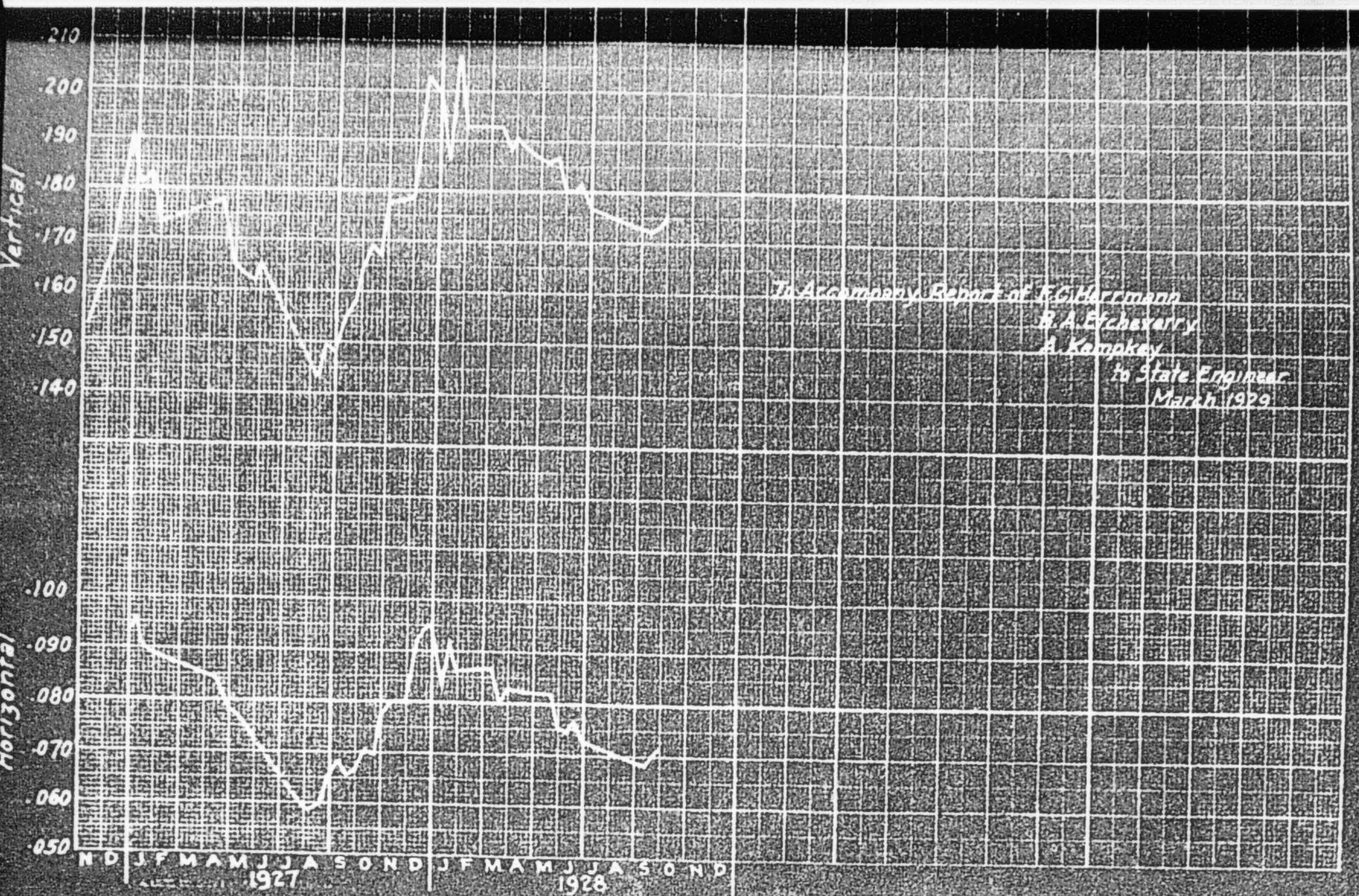
Lake Hodges Micrometer Readings
Buttress No. 18 South Side

PLATE 6



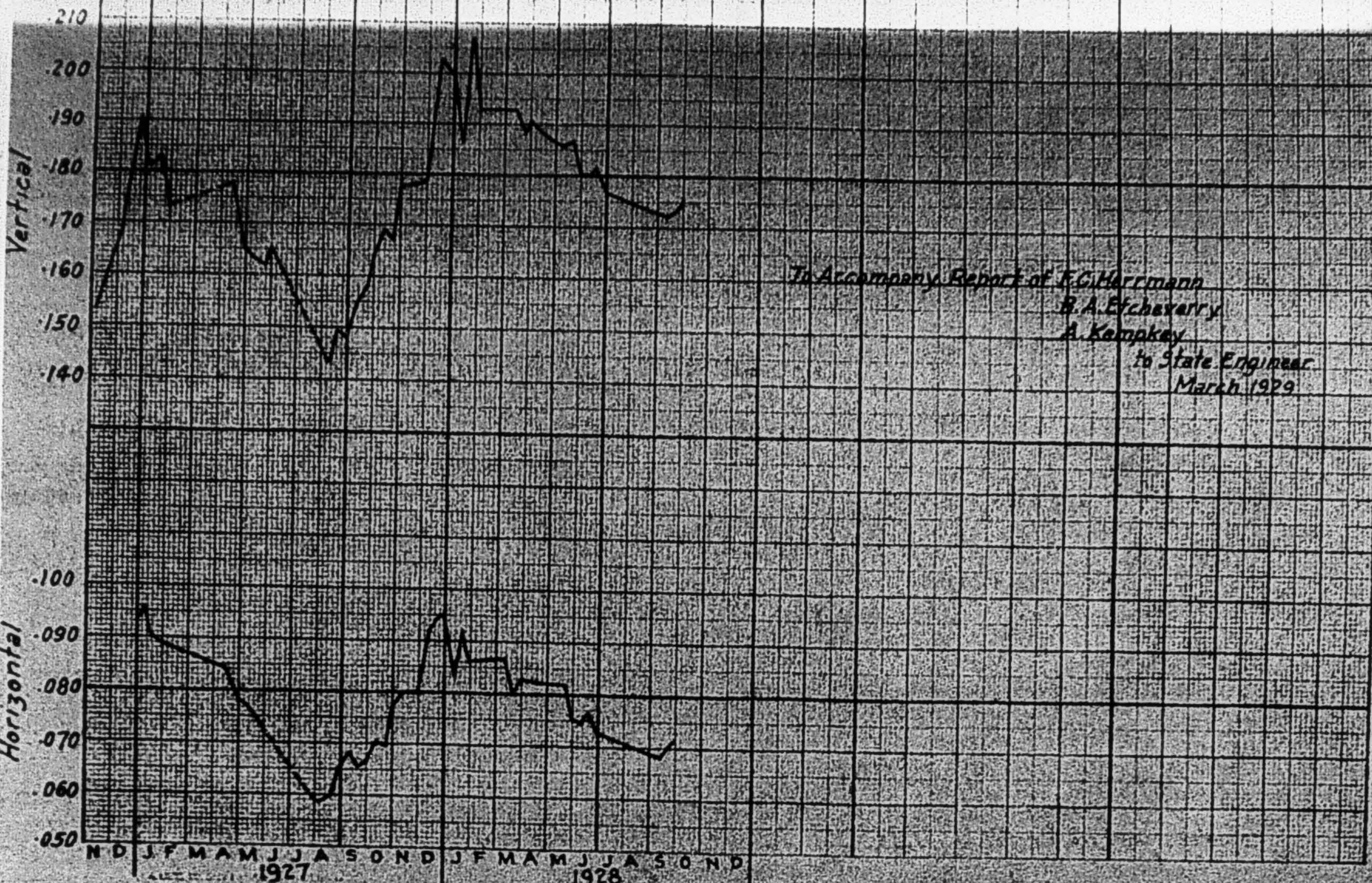
Lake Hodges Micrometer Readings
Buttress No. 18 South Side

PLATE 6



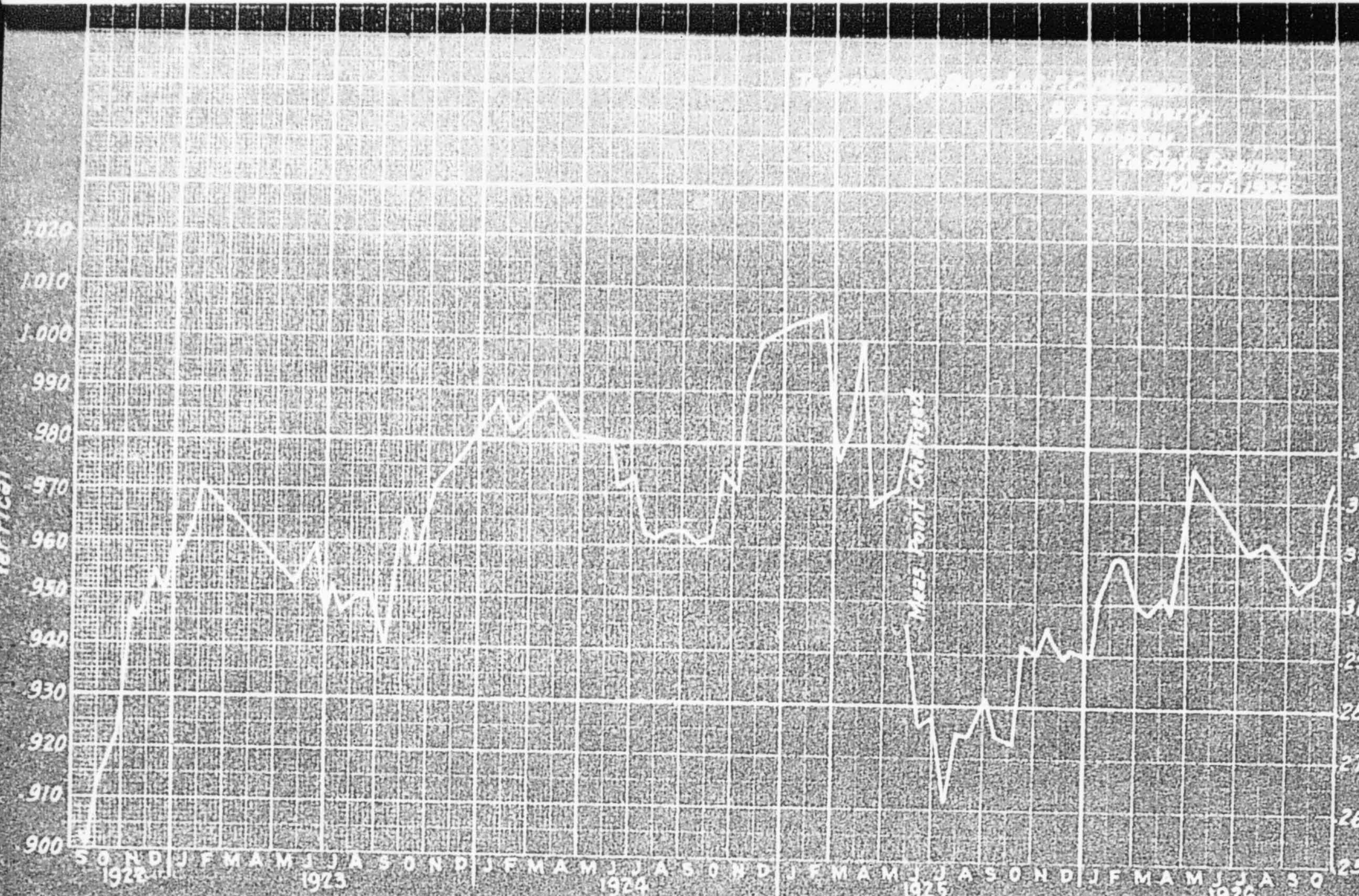
Lake Hodges Micrometer Readings
Buttress No 18 South Side

PLATE 6A



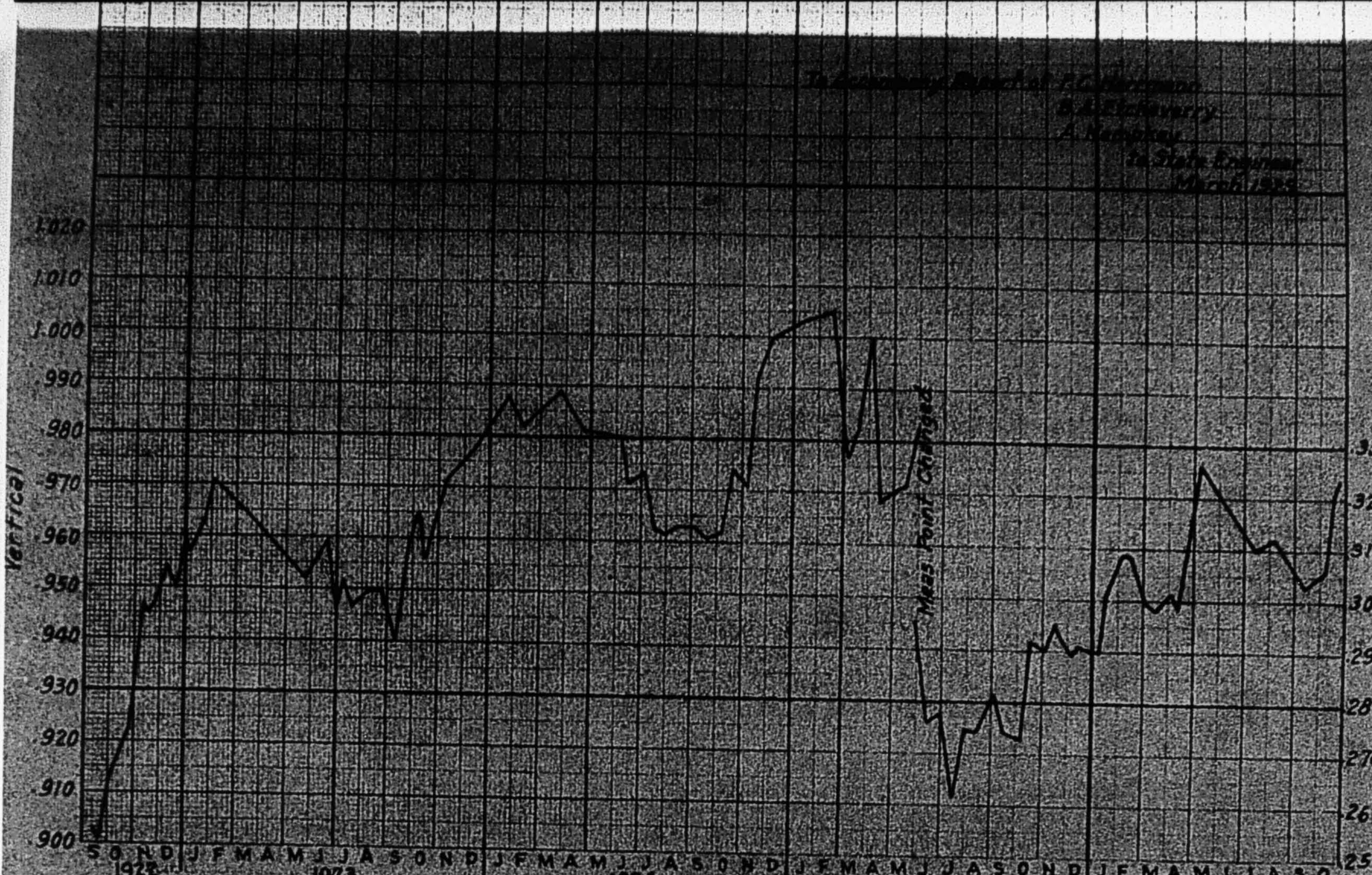
Lake Hodges Micrometer Readings
Buttress No 18 South Side

PLATE 6 A



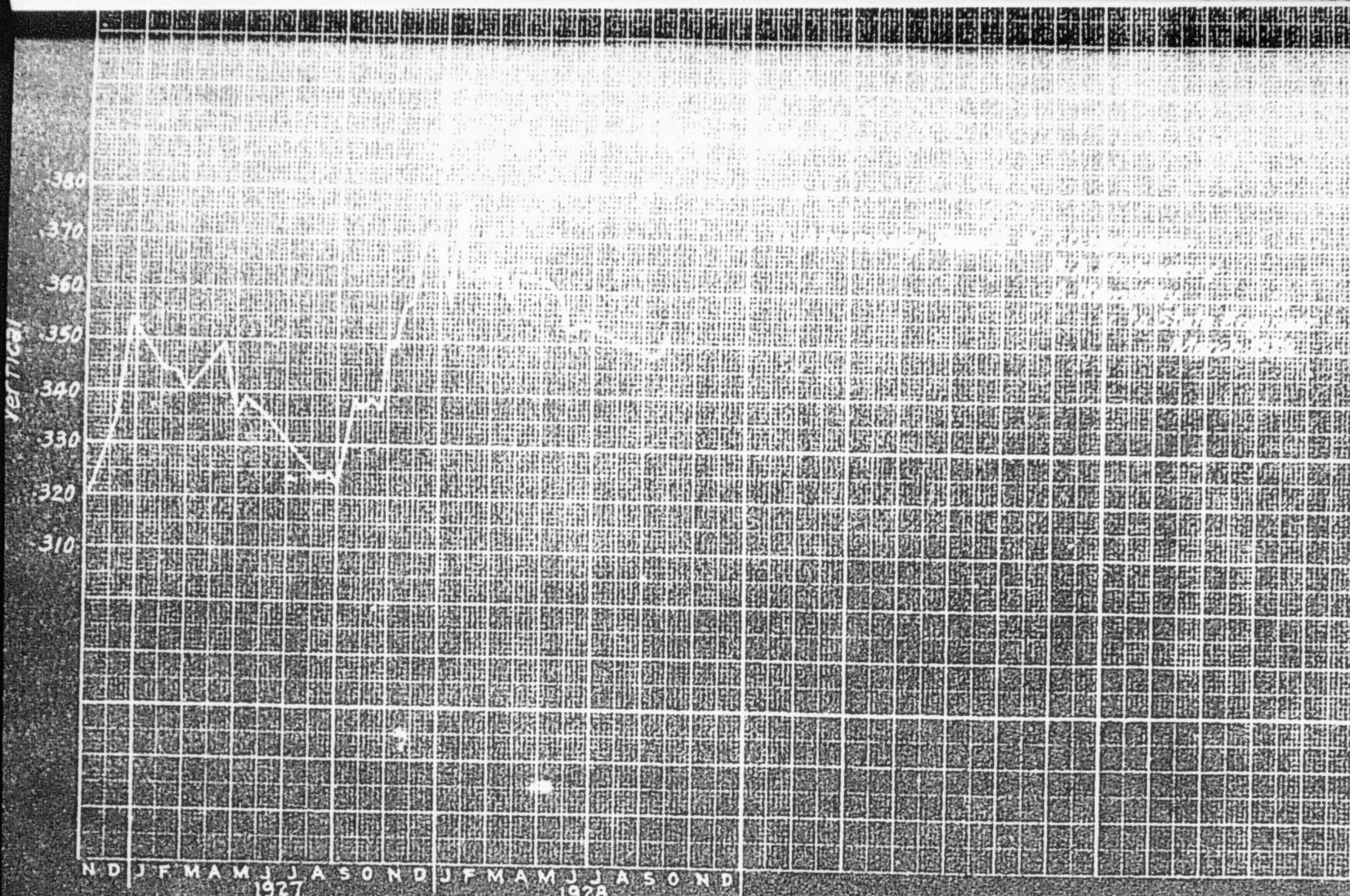
Lake Hodges Micrometer Readings
Buttress No. 19 South Side

PLATE 7

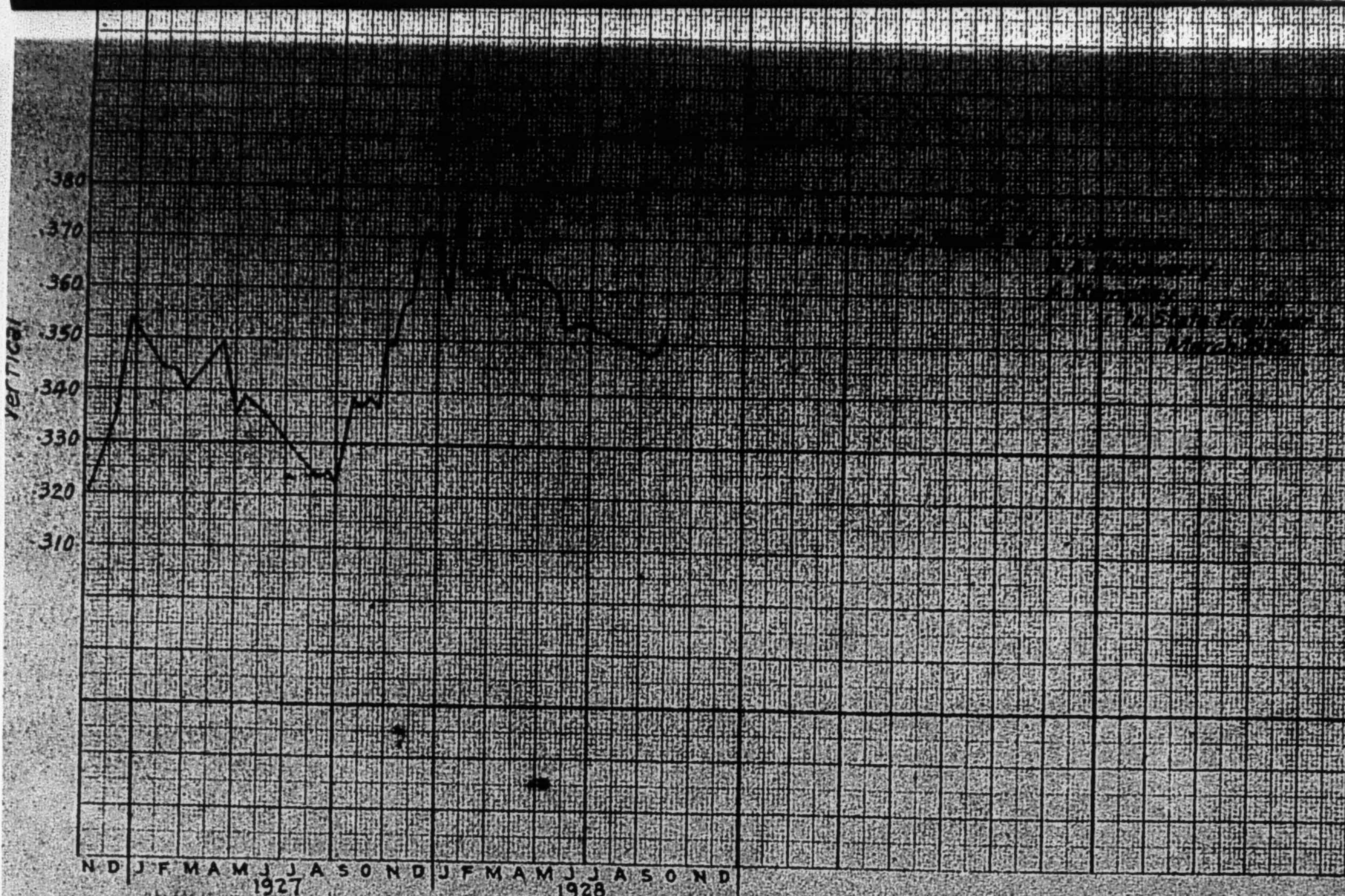


Lake Hodges Micrometer Readings
Buttress No. 19 South Side

PLATE 7



Lake Hodges Micrometer Readings
Buttress No. 19 South Side.
PLATE 7A



Lake Hodges Micrometer Readings
Buttress No. 19 South Side.
PLATE 7A

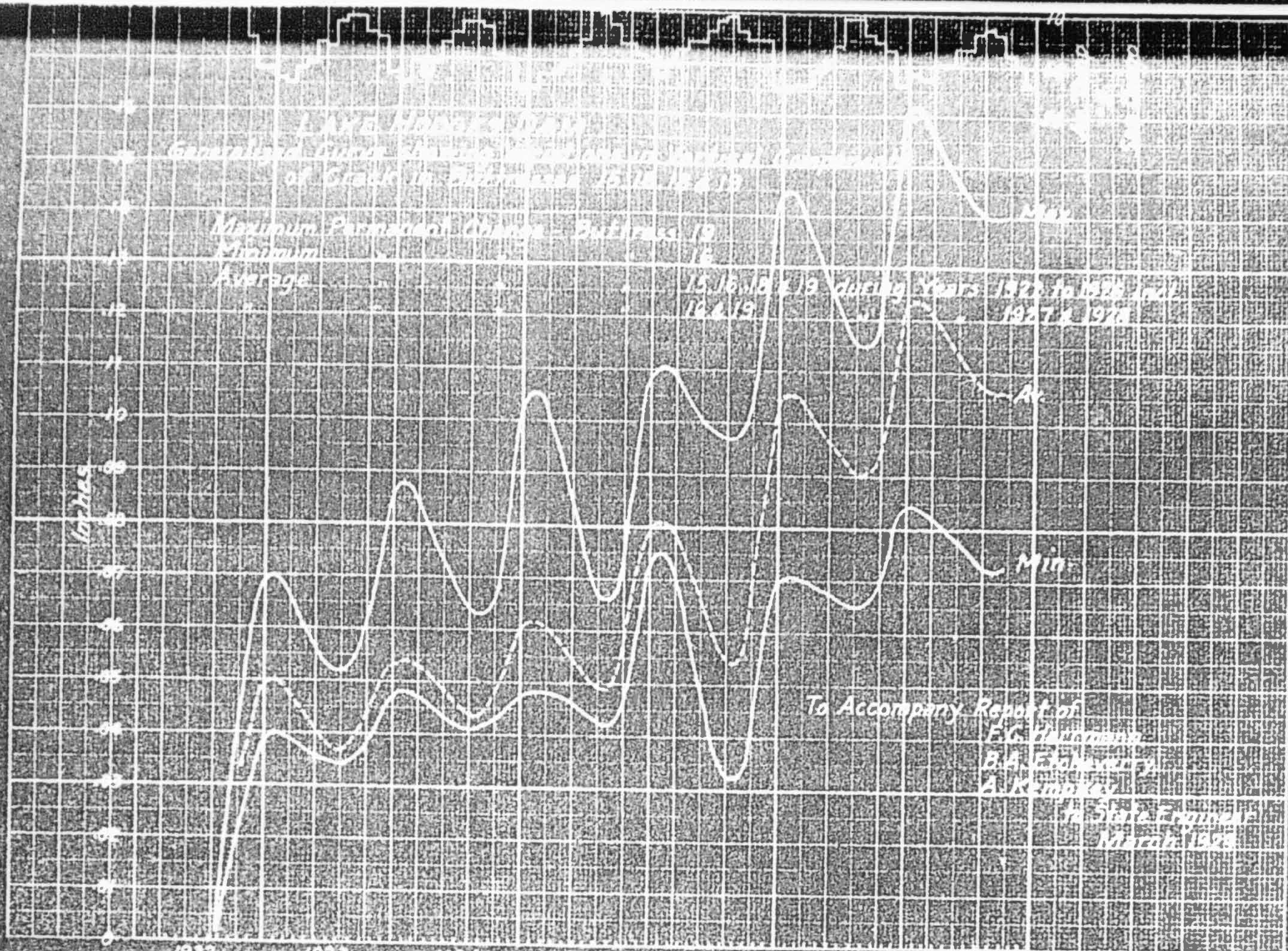

Maximum Permanent Change Buttress 19
Minimum 16
Average 15, 16, 18 & 19 during Years 1922 to 1926 incl.
16 & 19 1927 & 1928
Max.
Av.
Min.
Inches
1922
1923
1924
1925
1926
1927
1928
To Accompany Report of
State Engineer
March 1928

PLATE 8

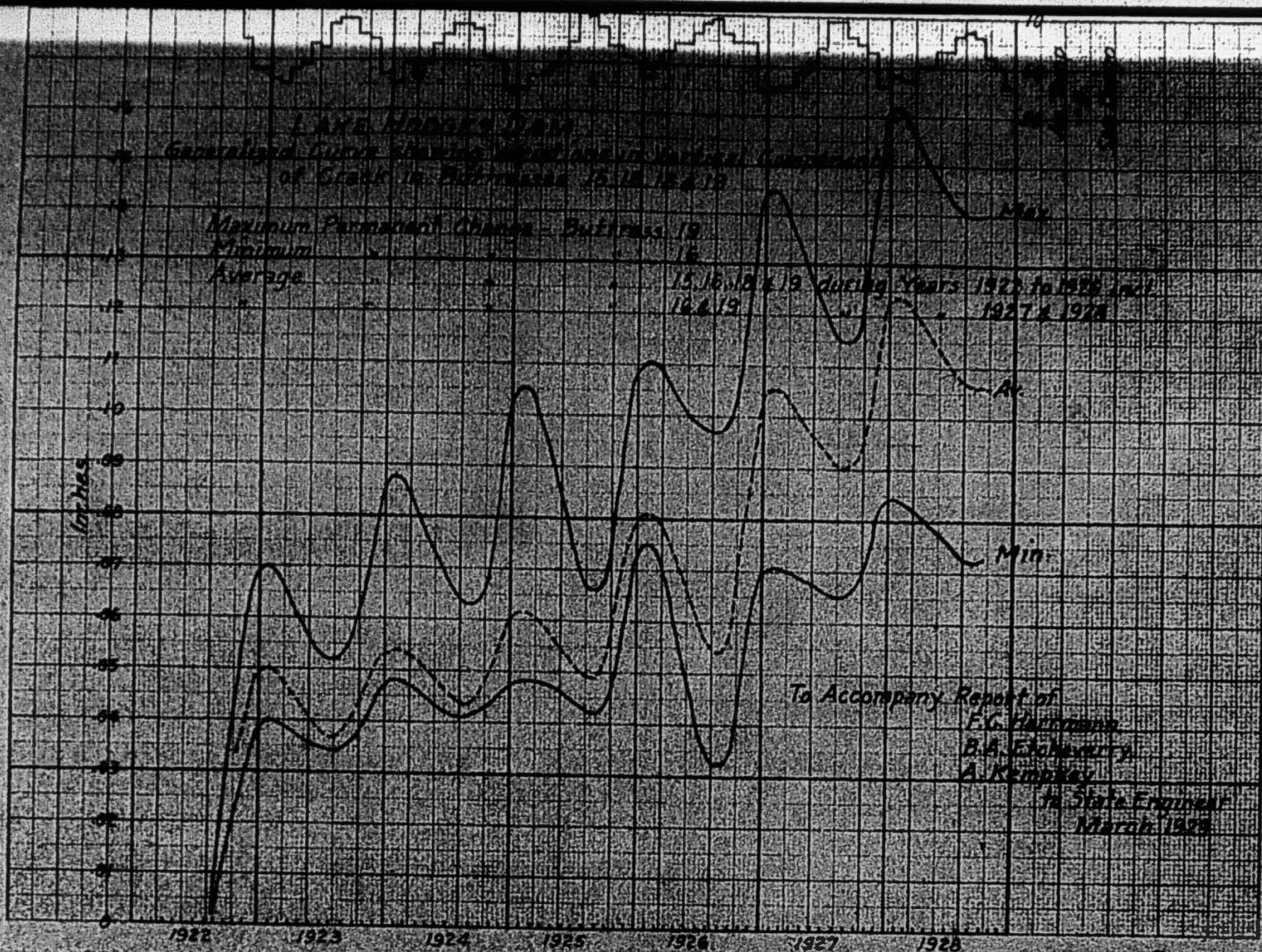
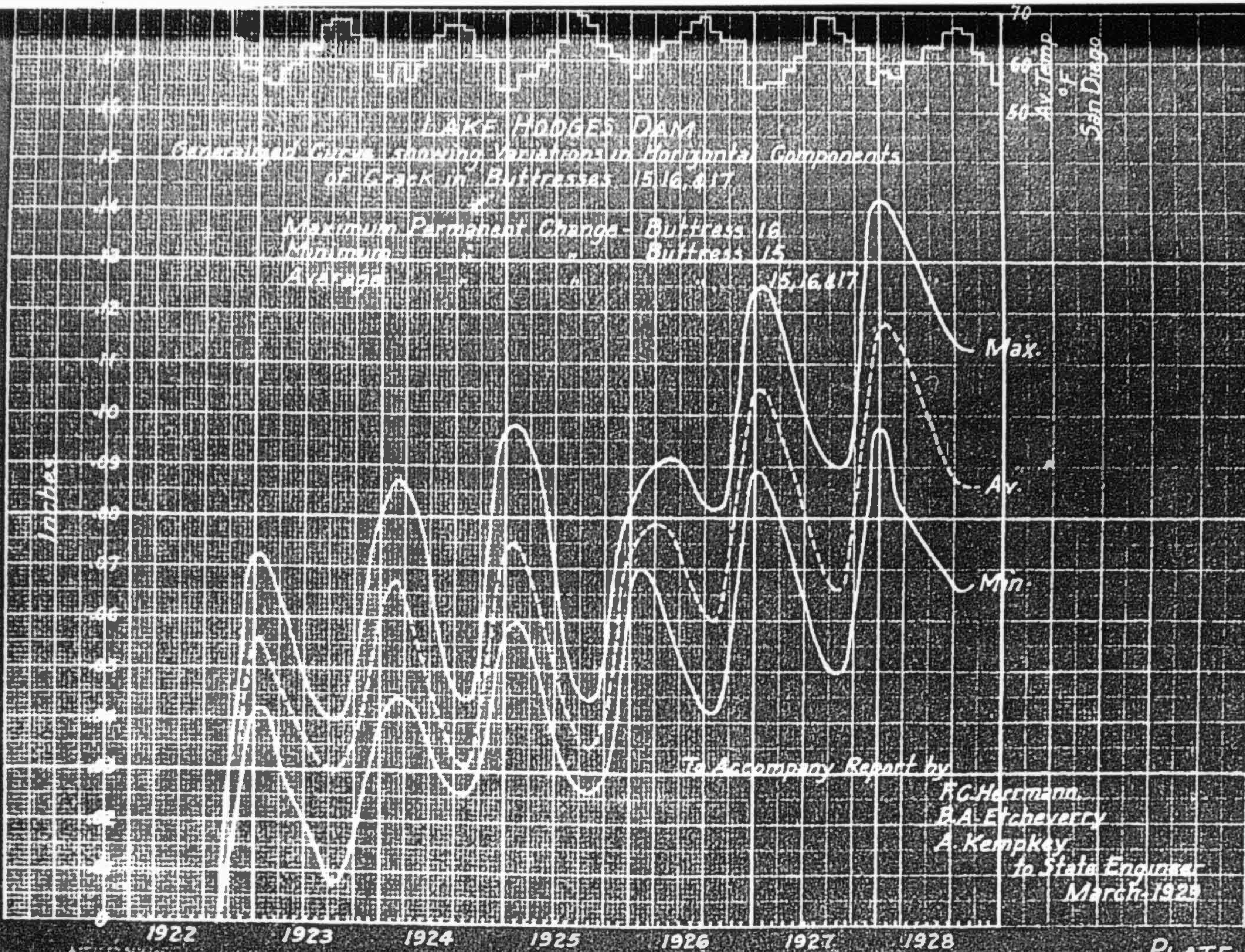
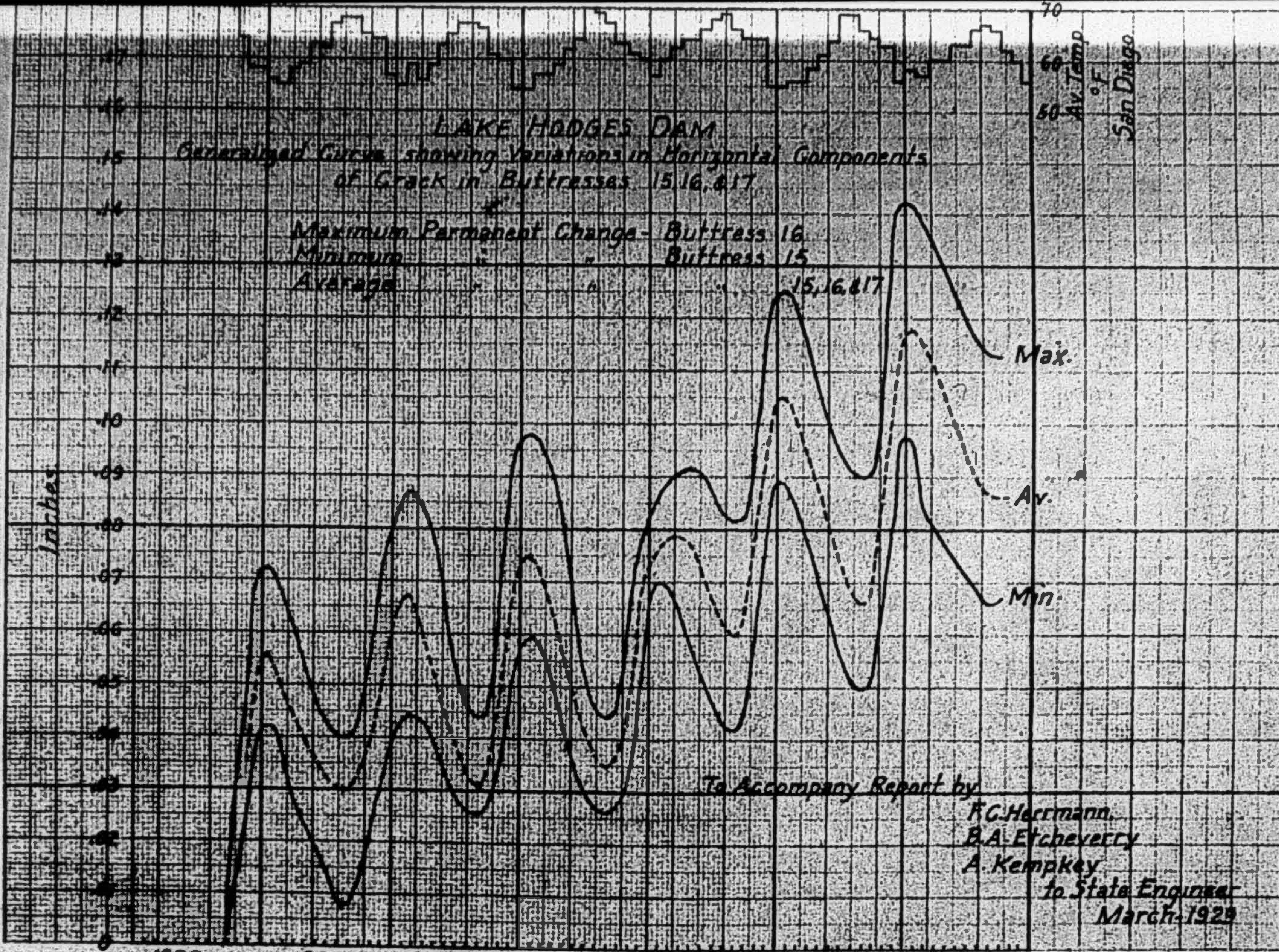

LAKE HODGES DAM
of Crack in Buttresses 15, 16, 18 & 19
Maximum Permanent Change - Buttress 19
Minimum 16
Average 15, 16, 18 & 19 during Years 1922 to 1926 incl.
16 & 19 1927 & 1928
Max
Av.
Min
Inches
1922
1923
1924
1925
1926
1927
1928
To Accompany Report of
B.A. Etcheverry
A. Kempkey
to State Engineer
March 1929

PLATE 8



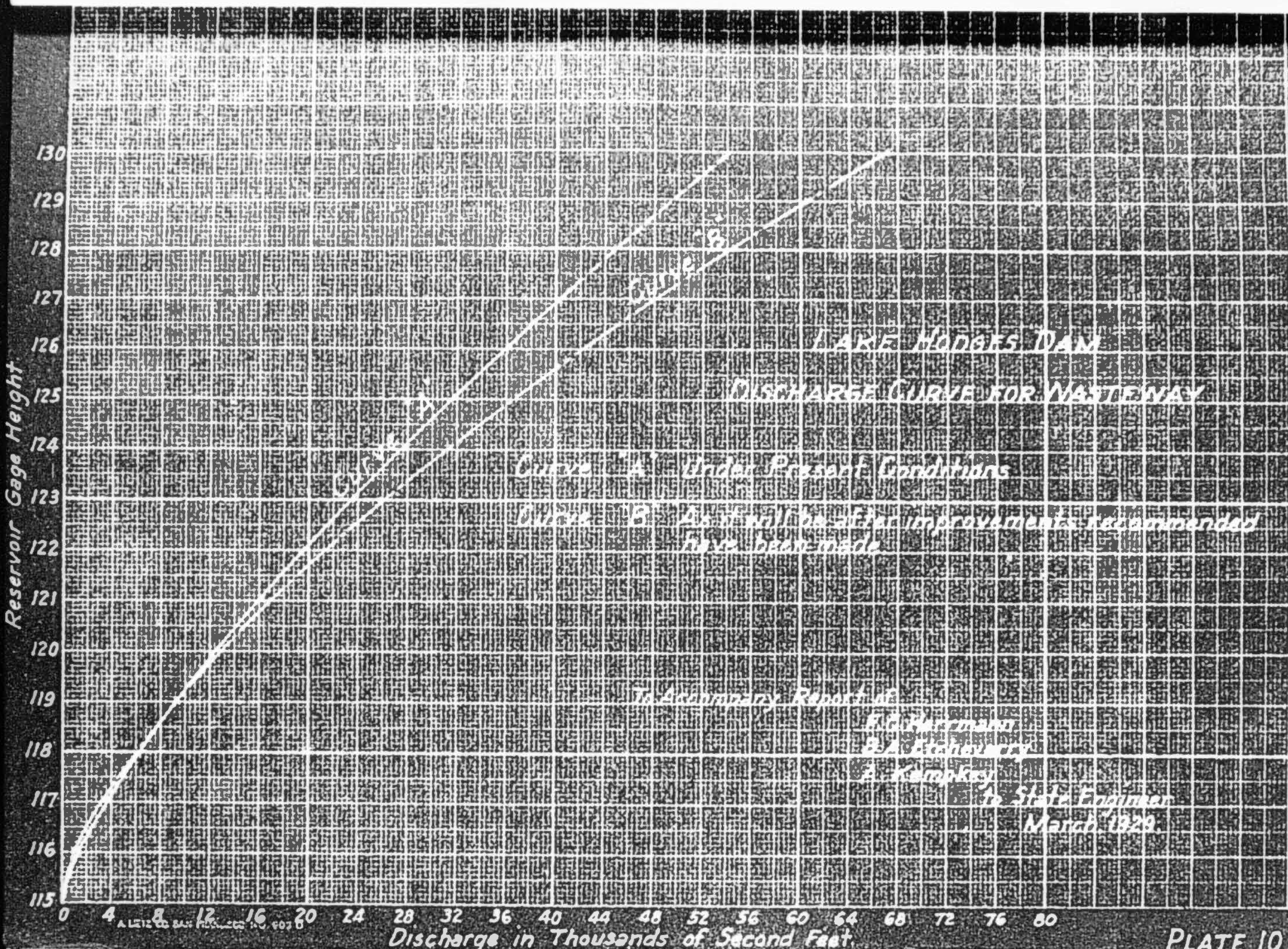
LAKE HODGES DAM
Generalized Curve showing Variations in Horizontal Components
of Crack in Buttresses 15, 16, & 17
Maximum Permanent Change - Buttress 16
Minimum " " Buttress 15
Average " " 15, 16, & 17
Max.
Av.
Min.
Inches
Av. Temp. °F San Diego
70
60
50
1922
1923
1924
1925
1926
1927
1928
To Accompany Report by
F.C. Herrmann
B.A. Etcheverry
A. Kempkey
to State Engineer
March 1928

PLATE 9



LAKE HODGES DAM
Generalized Curve showing Variations in Horizontal Components of Crack in Buttresses 15, 16, & 17
Maximum Permanent Change - Buttress 16
Minimum " " Buttress 15
Average " " 15, 16, & 17
Max.
Av.
Min.
Inches
Av. Temp °F San Diego
70
60
50
1922
1923
1924
1925
1926
1927
1928
To Accompany Report by
F. C. Herrmann.
B. A. Etcheverry
A. Kempkey
to State Engineer
March 1929

PLATE 9



LAKE HODGES DAM
DISCHARGE CURVE FOR WASTEWAY
Curve "A" Under Present Conditions
Curve "B" As it will be after improvements recommended have been made
Curve A
Curve B
To Accompany Report of
to State Engineer
March 1929.
Reservoir Gage Height
115
116
117
118
119
120
121
122
123
124
125
126
127
128
129
130
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
76
80
Discharge in Thousands of Second Feet

PLATE 10

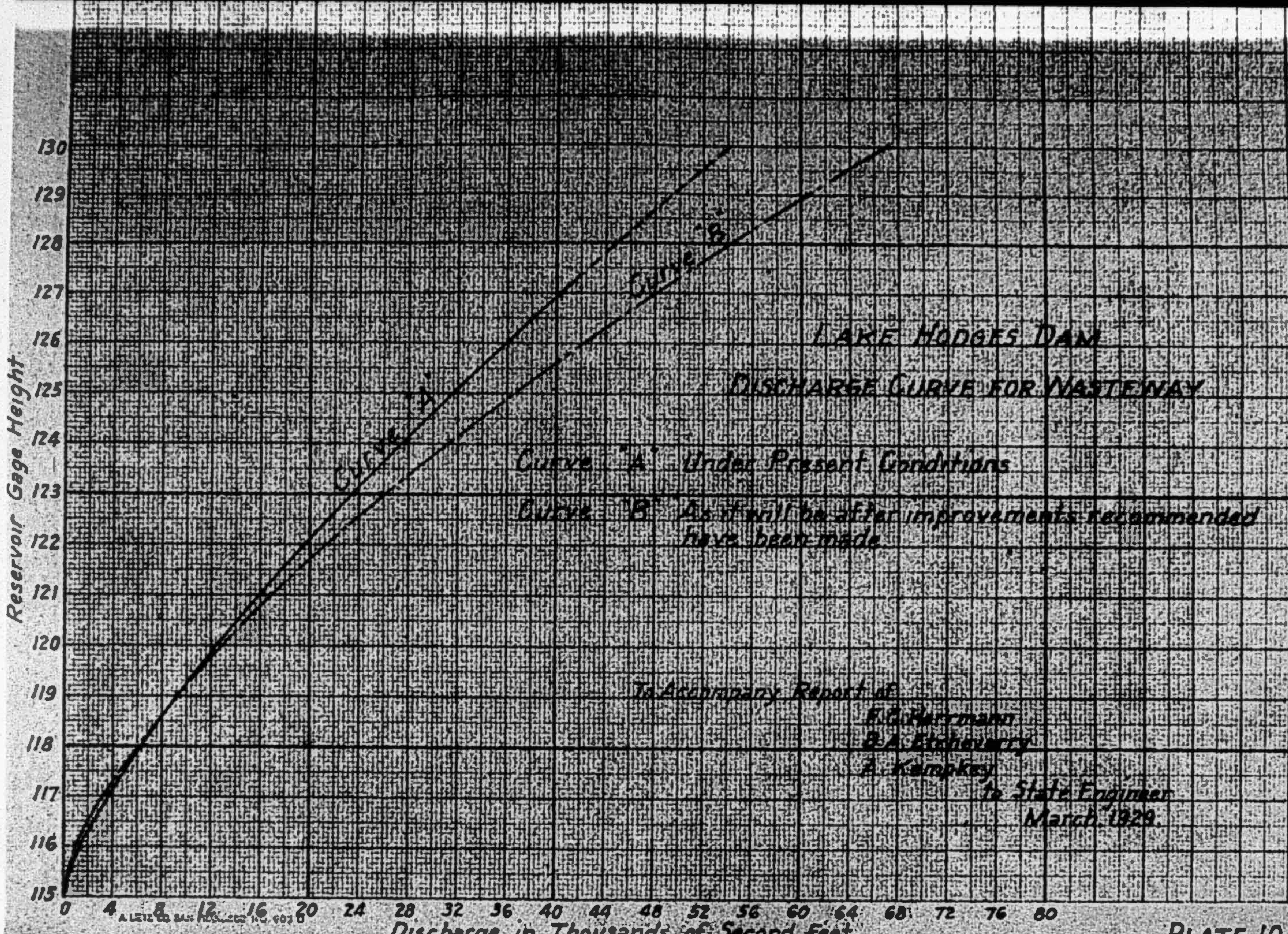
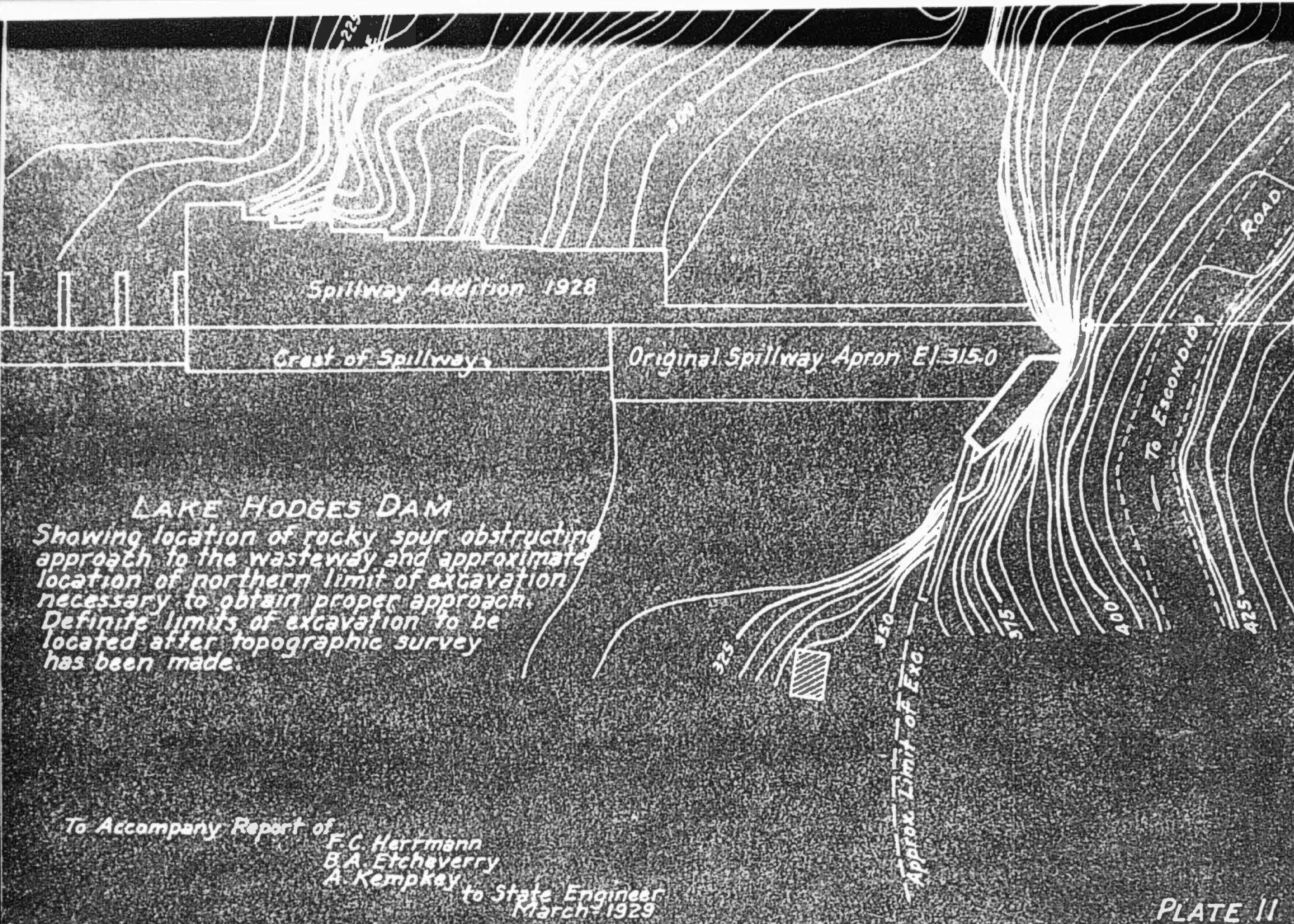

LAKE HODGES DAM
DISCHARGE CURVE FOR WASTEWAY
Curve "A" Under Present Conditions
Curve "B" As it will be after improvements recommended have been made
To Accompany Report of
F. C. Herrmann
B. A. Etcheverry
A. Kempkey
to State Engineer
March 1929.
Curve "A"
Curve "B"
Reservoir Gage Height
115
116
117
118
119
120
121
122
123
124
125
126
127
128
129
130
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
76
80
Discharge in Thousands of Second Feet

PLATE 10



Spillway Addition 1928
Crest of Spillway
Original Spillway Apron El 315.0
ROAD
To Escondido
225
300
325
350
375
400
425
Approx. Limit of Exc.
LAKE HODGES DAM
Showing location of rocky spur obstructing approach to the wasteway and approximate location of northern limit of excavation necessary to obtain proper approach. Definite limits of excavation to be located after topographic survey has been made.
To Accompany Report of F.C. Herrmann B.A. Etcheverry A. Kempkey to State Engineer March 1929

PLATE II

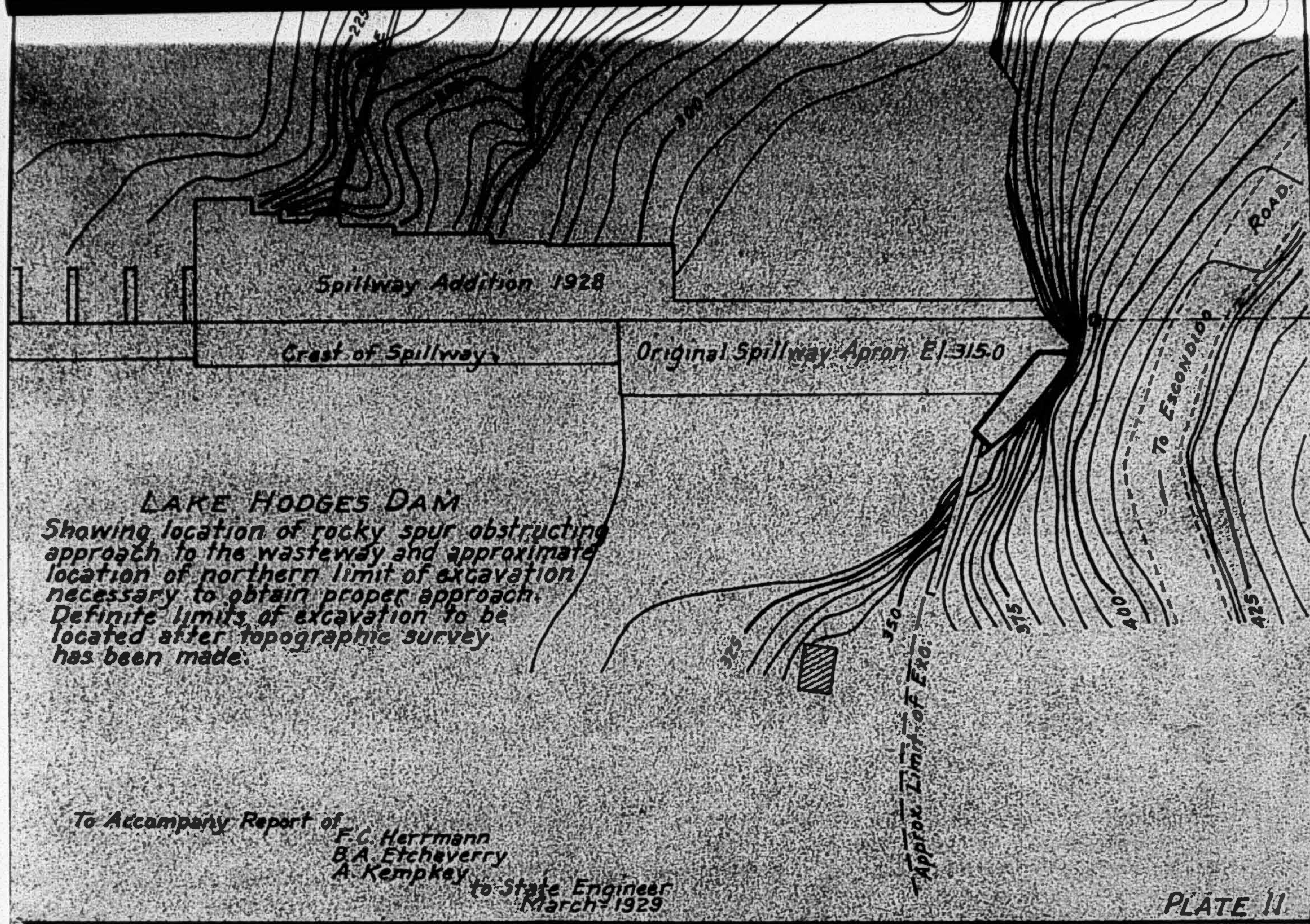

Spillway Addition 1928
Crest of Spillway
Original Spillway Apron El. 315.0
To Escondido Road
Approx. Limit of Exc.
225
300
325
350
375
400
425
LAKE HODGES DAM
Showing location of rocky spur obstructing approach to the wasteway and approximate location of northern limit of excavation necessary to obtain proper approach. Definite limits of excavation to be located after topographic survey has been made.
To Accompany Report of
F. C. Herrmann
B. A. Etcheverry
A. Kempkey
to State Engineer
March 1929

PLATE II

REPORT

ON

STRUCTURAL STRENGTH

OF

LAKE HODGES DAM

SAN DIEGO COUNTY

CALIFORNIA

Report made to

Messrs. B. A. Etcheverry,
F. C. Herrmann,
A. Kempkey,
Consulting Engineers

By

Henry D. Dewell,
Consulting Engineer
San Francisco

March 11, 1929

Messrs. B. A. Etcheverry,
F. C. Herrmann, Consulting Engineers
A. Kempkey

Subject: Lake Hodges Dam

Dear Sirs:

I report herewith the results of my investigations on the structural features of the Lake Hodges Dam. This investigation was made at your request. In detail, I was asked to review the strength of the dam to resist (1) normal water loads and (2) such earthquakes as might reasonably be expected in this vicinity. My attention was particularly called to certain cracks in the buttresses and I was asked to determine, if possible, the cause of these cracks.

To assist me in this work, I was given a set of plans of the dam; a copy of a report on the strength of the dam prepared by J. H. A. Brahtz for the Bureau of Water Development of the City of San Diego, California, under date of July 4, 1928; and also a copy of a report on the dam by Professor Chas. Derleth, Dean of the Department of Civil Engineering of the University of California to Mr. J. B. Lippincott, under date of July 18, 1922. The copy of Professor Derleth's report given me is incomplete to the extent that some of the material of the appendices is missing. I have also had the results of a stress analysis of the arches of the dam as made under the direction of Mr. W. A. Perkins, Assoc. Hydraulic Engineer, Division of Engineering, State Department of Public Works.

The strength of the dam against the normal water loads involves the investigation of the strength of the arches; the strength of the buttresses; the strength of the structure, considered as a unit, against sliding and overturning; and, finally, the foundation pressures. In conference with you gentlemen, it was agreed that my investigation of this feature of the subject should be rather a review of the reports and studies that have been made by Professor Derleth, Mr. Brahtz and Mr. Perkins, rather than making additional detailed calculations of the parts examined by them. This procedure, I have followed.

Strength of Arches

The subject of the stresses in arched dams, both of the single and multiple types, has received a great deal of study on the part of engineers in recent years. The reports of these studies have appeared in technical publications, particularly the "Proceedings" and "Transactions" of the American Society of Civil Engineers. Of especial importance in this respect is the work of Professor William Cain, dealing with the subject of thick concrete circular arches under normal loads, in which account is taken of the effect of shear. Mention of Professor Cain's studies should be accompanied by reference to the work of Mr. Frederick H. Fowler, M.Am.Soc.C.E., who has presented in the "Transactions" of the American Society of Civil Engineers, Vol. 92, 1928, graphical charts for quickly finding the stresses in circular arches under normal load, by means of the formulas of Professor Cain.

I have analyzed, by a graphical method, the stresses in the arch rings at Elevation 205 feet, taking into account (1) the normal water load, (2) the component of the weight of the arch ring perpendicular to the inclined face of the dam, (3) the effect of rib-shortening and (4) a rise and fall of temperature of 15° Fahrenheit (a total range of 30° Fahrenheit). This analysis did NOT take into account the effect of shear.

Table I below, shows the unit stresses found, in poundsper square inch. The plus sign (+) indicates compression, and the minus (-) sign indicates tension.

Table I

Approximate Unit Stresses in Arch Ring at Elevation 205 Feet.

(Effect of Shear not Included)

	Crown		Haunch	
	Intrados	Extrados	Intrados	Extrados
Without Temperature	+ 142	+ 396	+ 652	- 72
With Temperature	- 20	+ 504	+ 892	- 338
Temperature Stresses Direct	± 27	- 27	± 13	13
Bending	± 135	± 135	± 253	± 253
Maximum (-15°)	- 162	+ 108	+ 240	- 266
Maximum (15°)	+ 162	- 108	- 240	+ 266

Investigation of Arches by Fowler's Diagrams

I have also used Fowler's diagrams to find the approximate stresses due to dead weight and normal water load. To these have been added the temperature stresses shown in Table I. The results of this study are shown in Table II, below. Again, stresses of compression are indicated by the plus sign (+), and tensile stresses are shown by the minus sign (-). All stresses are in pounds per square inch.

Table II

Approximate Unit Stresses in Arch Ring at Elevation 205 Feet.

Stresses of Dead Load and Water by Fowler's Diagrams

	Crown		Haunch	
	Intrados	Extrados	Intrados	Extrados
Without Temperature	+ 157	+ 402	+ 634	- 6
Temperature Stresses	- 162	+ 108	+ 240	-266
Maximum Stresses Including Temperature (-15)	- 5	+ 510	+ 874	-272

Arch Stresses found by Brahtz

The unit stresses as found by Mr. Brahtz are given in Table III, below.

Table III

Unit Stresses in Arch Ring at Elevations 205 and 280

By J. H. A. Brahtz

Elev. of Arch Strip	Normal Stress due to "Gravity"	Normal Stress due to Uniform Drop in Temp.(-15°F)	Normal Stress due to Diff.of 5°F between Extrados & Intrados.	Total Normal Stress due to all effects Maximum	Maximum Compression due to Failure of Concrete in Tension	Maximum Shear in Arch
	+ 425	+ 108	- 41	+ 533	+ 600	Extrados (Crown)
	+ 130	- 158	+ 41	- 28		Intrados
205	+ 8	- 264	- 41	- 297		Extrados (Haunch)
	+ 575	+ 239	+ 41	+ 855	+1203 Intrados	
	+ 350	+ 90	- 23	+ 440		Extrados Crown
	+ 225	- 103	+ 23	+ 122		Intrados
280	+ 169	- 185	- 23	- 37		Extrados Haunch
	+ 408	+ 174	+ 23	+ 605	+652	Intrados

Hydrostatic pressure and normal component of arch weight or taken together and designated as "Gravity".

With reference to the unit compressive stress of 1203 lbs. per sq. inch, as found by Mr. Brahtz in the extrados at the haunch at Elevation 205, Mr. Brahtz says (See page 50 of his report)

> "At Elev. 205, the tension in extrados at haunch reaches 297 lbs. per sq. inch, which is far in excess of the reinforcing steel area. There is a possibility that the concrete and steel have failed in tension at this location. However, the arch has adjusted itself in compression, so that the theoretical maximum compression in intrados is computed to be 1203 lbs. per sq. inch. This is an upper limit because, when the arch fails in tension at the haunches it simultaneously behaves much like a two-hinged arch and naturally unloads moment from haunch to crown section. Consequently, the crown stress will accommodate the haunch stress; the former will be increased and the latter, decreased, until a new state of equilibrium has taken place. There is no failure in tension apparent at crown. This would show up as a vertically sloping crack in the intrados, and none is visible."

Mr. Brahtz is apparently incorrect in his conclusion that the maximum stress in the concrete is +1203 lbs. per sq. inch. The steel reinforcing of the arch rings, ½" square bars at 12 inches centers, can take the tensile forces at stresses within the usual working unit stresses. A computation, using Mr. Brahtz's values of total moment and thrust, and including the effect of the steel, shows that the maximum compression in the concrete is 920 lbs. per sq. inch; that the steel is stressed in compression to less than 14,000 lbs. per sq. inch, and, in tension, to only 5100 lbs. per sq. inch. Neglecting drop in temperature, the maximum stress in the concrete is 560 lbs. per sq. inch, when the effect of the steel reinforcing is included.

Results of Studies by Mr. W. A. Perkins

Mr. Perkins made an analysis of the arch ring stresses, taking account of shear, rib-shortening, and a rise or fall of temperature of

20°F. Three sections of the arch ring were examined, viz., at Elevation 280, 240 and 200 feet. The section at Elevation 200 is practically the bottom of the dam, and computed stresses based on the assumption of arch action at that level are larger than can exist, due to the restraining action of the foundation.

A study of the results found by Mr. Perkins shows that the maximum compressive stress in the arches of the dam, with an assumed fall in temperature, is around 900 lbs. per sq. inch.

The effect on the stress in the arch rings by absorption of water by the concrete is the same as a rise of temperature, and therefore is opposed to a drop in temperature which, in combination with dead load and water loads, gives the maximum unit stresses in the arch rings. Experiments by F. R. McMillan, M. Am. Soc. C. E., and by R. E. Davis, M. Am. Soc. C. E., indicate that concrete cured in air may expand, under the influence of water soaking by amounts varying from 0.03% to 0.05%. Such expansion is equivalent to a rise in temperature of from 60°F to 100°F.

Any attempts to evaluate in more detail the stresses due to water soaking are not justified. Much further data on the subject is needed. The extent to which the arch rings have absorbed water is unknown. In this connection, it may be remarked that the designer of reinforced concrete structures, i. e., concrete structures in which reinforcing steel is embedded, and which will have to withstand heavy hydraulic pressures, endeavors to obtain as dense a concrete as possible in order that water will not penetrate the concrete. Both disintegration of concrete and steel are feared.

It may be remarked that if the arch rings be assumed to be so completely water soaked that the effect is equivalent to a rise in temperature of 60°F, the combination of this assumed water soaking with an actual rise in temperature of 15°F, will, when added to the dead loads and water loads, result in a computed unit compressive stress of approximately 1000 lbs. per sq. inch. If the water-soaking be assumed to be equivalent to a 100° rise in temperature, the maximum unit strosses compute to reach approximately 1900 lbs. per sq. inch, in compression.

Again, shrinkage stresses, the amount of which depends on the method of construction, would decrease the stresses of water soaking. The complexity of the subject is evident.

Taking all investigations, which have been made of the stresses in the arch rings, into account; and remembering that the arch rings have absorbed some water, it seems reasonable to say that the maximum compressive stresses in these arch rings do not exceed 900 lbs. per sq. inch. It is more likely that the effect of water absorption by the concrete is enough to offset the stresses due to drop in temperature, and that the maximum arch stresses do not exceed 700 lbs. per sq. inch.

In view of the above considerations, I am of the opinion that the arches of the dam are safe for the dead and water loads that they have to carry.

Buttresses

The buttresses are the weakest feature of the dam. They are high slender walls, unreinforced, and tied together only by widely separated and relatively weak reinforced concrete struts. The fact that the buttresses have taken their load without indication of buckling for ome ten years is evidence that these struts are able to function. eaving aside for the moment any discussion of the cause of the cracking hich exists in these buttresses, it may be said that these buttress alls are similar to long slender columns. The details of the cross truts and their connections to the buttress walls are poor. Without hese struts the buttress walls would undoubtedly fail; to this extent herefore the struts are efficient. Any attempt to definitely set the ncrease of loading on these buttress walls which would cause their collapse as long columns would be futile.

Building codes do not encourage the use of unreinforced conrete walls. Such walls are subject to the regulations governing brick asonry walls. The limiting thickness of wall is usually specified in erms of its unsupported length. A common specification allows an unupported length not to exceed sixteen times the thickness. For comparison, he ratio of unsupported length to average thickness of buttress in the ake Hodges dam is about the same, if the stiffening effect of any one trut is considered as fully effective over that length of wall which is t right angles to the assumed direction of compression, and which is equal o the spacing between struts. Such "point support" would not be considered as effective by any building code, which requires "line support", .e., lateral support as furnished by a wall, buttress, pier, etc. There re no comparable specifications for dams.

I have stated above that, if the bracing struts be considered to e effective over the areas which they subtend, the walls have a ratio of nsupported length to average thickness of approximately sixteen. This is rue only for the <u>uncracked</u> buttresses. The delineation of the existing hey will continue to do so, unless loading conditions are adversely hanged.

Diagonal Cracks

The diagonal cracks in the buttress walls have been the subject of much discussion. Evidence seems conclusive that those cracks appeared before the dam was loaded with water, but that they have increased in width, and are still increasing in width. Measurements, apparently made with great accuracy, indicate that the width of these cracks varies with the season of the year, and also that there is an actual small increase in width from year to year. Evidence is lacking with respect to any increase in length. Professor Derleth, after analyzing the nature of the internal stresses in the buttress walls, and giving consideration to the probable limits of horizontal and vertical stresses, came to the conclusion that it was improbable that tensile stresses, due to dead weight of structure and water load, were the cause of these cracks. He found, in fact, that the "principal stresses" on planes passing through the cracks at the horizon of the gallery opening were stresses of compression.

It is practically impossible to determine accurately the stresses at any point in the buttresses of a multiple arch dam. The approximate stresses may be found in a dam of the gravity type. Professor William Cain gives a method in Appendix II. of his "Practical Designing of Retaining Walls, with Appendices on Stresses in Masonry Dams", Sixth Edition, Revised and Enlarged, D. Van Nostrand Company, 1910. The method used comprises the computation of the areas and moments of inertia of three successive horizontal sections through the dam, separated by constant small intervals, the median plane passing through the point at which the stresses are desired; the computation of the vertical unit pressures on each of these planes, assuming a trapezoidal variation of these pressures;

e computation of the unit shears on vertical planes, which, for any e portion of the horizontal block, must be equal to the algebraic sum f the upward and downward pressures on the horizontal planes bounding he block plus the weight of the block of masonry; the assumption that he shear intensity increases uniformly, vertically, from the highest to he lowest horizontal plane under consideration; the use of the established rinciple that the intensity of shear at a point, on vertical or horizontal lanes, is the same; the calculation by means of the three fundamental ormulas for equilibrium, viz., EX = 0, EY = 0, and EM = 0, of the average ntensity of stress on vertical planes; and, finally, from the preceding, he calculation of the principal and conjugate stresses.

The application of this method to the buttress of a multiple rch dam involves some difficulties. Certain assumptions, additional to those given above, must be made. Nevertheless, I have employed this method to a buttress of the Lake Hodges Dam, and have calculated thereby the principal and conjugate stresses at four points on a horizontal plane through the gallery opening, or at Elevation 235.

These four points, and the stresses found, are shown on Figure 1. The four points are at distances 61' - 6", 66'-6", 71'-6" and 81'-6", respectively, from the back face of the dam. The point which is 71'-6" from the back face is at the center of the gallery opening. Table V, below gives the results of these computations.

Table V

Results of Computations for "Principal Stresses" in Walls

All points on a horizontal plane through the gallery opening, or at Elevation 235 ft.

Distance of point from Back of Dam	Angle Θ between Horizon and the Plane to which the Principal Stress is Normal	Stresses in Pounds per sq. inch.				
		f	f'	q	p	p'
61'-6"	48°-50'	+293	+94	+122	+153$^{\underline{5}}$	+186$^{\underline{4}}$
66'-6"	51°-47'	+310	+43	+130	+145	+207
71'-6" (Gallery Opening)	54°-04	+327	+36	+138$^{\underline{4}}$	+136$^{\underline{5}}$	+226$^{\underline{7}}$
81'6"	57°-30'	+366	+20	+156$^{\underline{8}}$	+119$^{\underline{5}}$	+266$^{\underline{5}}$

NOTE: Compression is indicated by Plus (+) sign.

p = vertical unit stress on a horizontal plane.

q = shearing unit stress on horizontal or vertical planes.

p'= horizontal unit stress on vertical planes.

f = maximum normal stress acting on a plane inclined to the horizontal at the angle, Θ.

f'= minimum principal stress or conjugate stress, normal to the direction of stress "f".

These results indicate that, for the uncracked buttress, there is no tension across an inclined plane following the present crack. Certain points, which may have some significance, are to be noted from the figures of Table V.

(1) The plane on which the minimum compressive stress acts has the same direction as the present crack.

(2) There is seen to be a sudden increase in the amount of the minimum compressive stress as points are taken

down stream from the gallery opening. Thus, beginning at the point 81'-6" from the back of the dam, the unit minimum compressive stress increases from 20 lbs. per sq. inch to 36 lbs. per sq. inch, an increase of 16 lbs. per sq. inch in 10 feet; in the next five feet, this stress increases from 36 lbs. per sq. ft. to 43# per sq. ft., an increase at the rate of 14 lbs. per 10 ft.; in the succeeding five feet, the stress jumps from 43 lbs. per sq. inch to 94 lbs. per sq. inch, or an increase of 51 lbs. per sq. inch in 10 feet.

(3) At the point in the gallery opening, the values of "p" and "q" are practically equal. This is not even approximately true at the points investigated, at the right or left.

(4) It is probable that the minimum principal stress, found to have an intensity of 20 lbs. per sq. inch, at the point 81'-6" distant from the back face, changes to tension as successive points are taken nearer to the front face.

In making the above calculations, the arch rings and the buttresses, including the back pilaster, were assumed to act integrally when resisting moment due to the tendency of the water load to overturn the dam. Engineers are not agreed as to the validity of this assumption. The assumption is, however, logical and proper, in my opinion. The arches were not considered as resisting shear. This assumption is commonly made, but is probably in error.

No other points in the buttress walls other than those just mentioned were investigated. The calculations incident to this method of finding principal stresses are long and tedious. Great accuracy in numerical work must be maintained. The use of slide rule is prohibitive, and seven place logarithms or a calculating machine must be used. This statement is made as an explanation of why additional points in the buttresses were not examined.

As a result of the above calculations, together with a study of he reports of Professor Derleth and Mr. Brahtz, it appears that the racks in the buttresses are primarily due to shrinkage of the concrete nd to temperature changes. It seems to me reasonable that these cracks, fter being widened by the seasonal change of temperature, cannot completely close for two reasons:

(1) The closing of the cracks must be made against the outward pressure of the water and

(2) The possible and probable infiltration of dirt and sand.

However, as the validity of the results of the computations for amount of principal stresses rests upon assumptions that may be more in error than we realize, it is possible that there has been tension across the cracks due to the loads on the dam. Mr. Perkins informs me that his calculations indicate actual tension at these points. I have not had opportunity to examine his studies on this part of the work.

Mr. Brahtz has made a detailed study of the effect of the cracks, assuming that they extend completely through the height of the buttress, i. e., from arch to foundation. Without giving details of his computations, he finds that the dam would be safe with the buttresses so cracked, except for the "long-column" action discussed previously.

Under the above assumption, i.e., that the dam is divided into two separate units by the cracks, Mr. Brahtz finds the following distribution of foundation pressure (See Fig. 2).

The maximum unit soil pressure at the downstream toe, assuming he dam to be uncracked, is found by Mr. Brahtz to be 19.3 tons per sq. t., and the maximum shear along the base 142.5# per sq. inch. The orresponding values, as found by Professor Derleth, are 20.8 tons per q. ft., and 145 lbs. per sq. inch, respectively.

The coefficients of sliding, and safety factors against overurning, assuming the dam to be completely cracked through, are found by r. Brahtz to be as follows:

Table VI

Values for Cracked Dam - by Brahtz

	Coefficient of Sliding	Minimum Safety Factor against Overturning
pstream Portion	0.822	3.22 (El. 200)
ownstream Portion	0.554	1.42 (El. 194) 1.50 (El. 200)

Professor Derleth finds the following values for the dam, conidered as uncracked.

Table VII

Values of Coefficient of Sliding and Safety Factors against Overturning

Values for Uncracked Dam - By C. Derleth, Jr.

	Coefficient of Sliding		Safety Factors against Overturning
	Calculated	Recommended	
ection - Elev.			Resultant of Weight of Dam and Water Pressure fall within middle third
200	0.75	0.65	
" Elev. 235	0.81	0.65	
" Elev. 255	0.83	0.65	

The weight considered is the weight of the buttress plus the weight component of the arch barrels normal to the inclined face of the buttress.

The figures of Tables V and VII indicate that the dam is in o danger of sliding or overturning, although the safety factors in ach case are too low from the standpoint of conservative design.

Effect of Possible Earthquakes on the Dam.

The statement has been made that the Lake Hodges Dam withstood n earthquake at about the time of its completion, and that examination mmediately thereafter failed to disclose any cracking. Mr. Fred A. oetzli states in the "Proceedings" Am.Soc.C.E., March, 1929, Page 817, According to Mr. Stanley Bent, shortly after the completion of the Lake odges and the Murray multiple-arch dams, near San Diego, California, the urrounding country experienced a fairly severe earthquake. An immediate nspection failed to show any cracks or other visible signs of the effect f the earthquake on the structures".

The reference is probably to the "San Jacinto Earthquake" of pril 21, 1918. This earthquake is described and discussed in detail in he Bulletin of the Seismological Society of America, June-September, 1918. he following quotations are from the publication.

> "Damage to Buildings:- The greatest damage caused by the earthquake was in the business district of San Jacinto. In the principal business block of the town but two buildings remained standing - a new concrete building and a frame building. The buildings wrecked were mostly brick or artificial stone of poor construction, or old frame buildings. Concrete buildings, substantial frame buildings, and well-constructed brick buildings were not seriously damaged. The damage at Hemet was considerably less than at San Jacinto. No buildings were completely destroyed and none of substantial construction were seriously damaged. The property loss at San Jacinto was estimated at $125,000 to $150,000, and that at Hemet not more than $75,000. There was also some damage to buildings at various places within a hundred-mile radius of San Jacinto. This damage consisted for the most part of broken plaster, chimneys,

and plate-glass windows. The writer did not make any investigation of these damages outside of San Jacinto and Hemet.

"Intensities and Isoseismals:- One hundred and sixty-six reports of the San Jacinto earthquake were collected by Mr. Hamlin, five were added by Messrs. Strong and Rolfe, and three by the author. The reports were studied by Mr. Rolfe and the intensity of the earthquake determined by him at each place from the data given by each observer. The observer making the report is not asked to estimate the intensity of the shock, but merely to answer questions and relate his experiences. The estimate of intensities is then made by the person who works up the reports. The writer has also studied these reports and made an independent estimate of intensity for each. The isoseismals shown in Fig. 3 are the result of the combined estimates of Rolfe and Townley, but the curves have been drawn by the author without consultation with Mr. Rolfe.

"At Hemet Lake the caretaker told us that no damage had been done to the high masonry dam at that place--not even a crack in the plastering. Although the shock at this place was severe, neither the caretaker nor his wife, both of whom were looking at the lake when it occurred, observed any wave action on its surface. Cracks appeared in the moist ground, and a large spring dried up; the flow of the spring, however, is gradually coming back."

A study of the isoseismals indicates that the intensity of the earthquake at the site of Lake Hodges Dam was about Grade VI, in the Rossi-Forel Scale of earthquake intensities. This intensity of earthquakes as defined by this scale is as follows:

"Fairly strong shock: general awakening of those asleep; general ringing of house bells; oscillation of chandeliers; stopping of pendulum clocks; visible agitation of trees and shrubs; some startled persons leave their dwellings."

It would thus appear that the intensity of the San Jacinto earthquake at Lake Hodges was not such to have caused damage to even the poorest construction.

A study of the earthquake records shows that no earthquake of ater intensity than Grade V or VI of the Rossi-Forel scale have curred in the vicinity of Lake Hodges since 1917. The statement may erefore be made that the Lake Hodges Dam has never been subjected to y earthquake approaching that of major intensity, or even of sufficient rength to crack masonry walls. The Fault Map published by the Seismo- gical Society of America shows a probable fault running through Lake dges reservoir. Dr. Bailey Willis, Past President of the Society, forms me that there is no question but that this fault exists, but that ere is no definite information as to whether it has been historically tive. He considers that San Diego lies at the extreme northerly end of seismic zone, the major portion of which is in Lower California and bout which there is consequently little known. Records indicate that n Diego was severely shaken in 1812 and again in 1857, the latter earth- hake originating on the great San Andreas fault, with maximum intensity ear Fort Tejon. In 1892 and 1894, San Diego was visited by earthquakes f sufficient intensity to crack walls of buildings. All four of these rthquakes mentioned had an intensity of at least VIII, and approaching X, in the Rossi-Forel scale, at San Diego. There seems no reason to be- ieve otherwise than that the Lake Hodges Dam will be subjected at some ime in the future to an earthquake of major intensity. Conjectures as o the probable time of such occurrence are not in order. The region is t immune to earthquakes; practically all of California is seismically ctive.

Coming to the question of the action of an earthquake on a ltiple arch dam, like the Lake Hodges Dam, two conditions of direction shock are to be recognized:

(1) that in which the shock and the direction of vibrations parallel to the direction of the stream bed and

(2) that in which the shock and direction of vibrations is rallel to the face of the dam, or normal to the direction of the stream d. Next, two conditions of action are to be recognized.

(1) that of a single shock tending to move the structure as unit and

(2) a series of vibrations, portions of which may be of simple rmonic type, tending to rack the structure. The dam may first be invesigated on the assumption that it acts as a unit mass, approximately rigid. he force is then equal to the product of the mass of the structure by the aximum acceleration of the earthquake wave. It is customary to consider he force as acting in a horizontal direction only, the effect of the ertical component being neglected, as more than over-balanced by the dead eight of the structure. Before the value of any stresses in the structure y be estimated, it becomes necessary to evaluate the effect of the cceleration of the earthquake. Accelerations in excess of gravity are elieved to have occurred, but only at very rare intervals. The values of cceleration, rather commonly accepted as being reasonable, vary from 1/20 1/3 of gravity. It may be remarked at this point that the actual values accelerations that have occurred in earthquakes of major intensity are considerable doubt. I believe that I am correct in stating that there but one actual seismographic record of a major earthquake shock recorded the point where major damage was done to structures. I refer to the east d west component of the Tokyo shock of September 1, 1923. In other cases,

ismographic records available are those taken at points some distance om locations in which the greatest damage occurred, or such seismo-aphic records are incomplete due to damage to the seismographs. Val-tions of the maximum accelerations occurring as made by computations of e forces necessary to overturn objects observed to have been thrown down e open to serious question as to their accuracy. In fact, it may be own that the rather classic "West's formula", used by Omori, the late stinguished Japanese seismologist, in evaluating earthquake accelerations, ives erroneous results. It may also be remarked that the majority of the panese seismological records were made with seismographs with improper mping, so that their recorded amplitudes are likely to be inaccurate. his discussion may be summarized by saying that the actual maximum cceleration of a major earthquake is unknown. It is probable that the cceleration of an earthquake of major intensity at the site of the Lake odges dam would not be less than 1/20 of gravity, and not greater than /3 of gravity. I would place the reasonable limits for design at a inimum acceleration of 1/10 of gravity, or about three feet per second er second, and the maximum acceleration at 1/5 of gravity, or about six eet per second per second. Should the dam be situated directly over a ault which would be active, rupture of the structure might be expected. understand that a geological study of the site finds but a minor fault t the site.

It is encouraging to recall that the Crystal Springs dam, a ravity dam of massive section, situated near San Francisco, the Sears-ille Dam, and the Williams Dam, passed through the 1906 earthquake without mage, although the San Andreas fault, the seat of the disturbance was ediately adjacent to the first two dams. The Committee of the San ncisco Association of Members of the Am. Soc. C. E., on the Effect the Earthquake on Water Works Structures reported as follows on the Mateo or Crystal Springs Dam, and the Portola or Searsville Dam, hin a few miles of the former,

> "Both are built of irregular, interlocked blocks, formed in place, but are substantially monolithic. Each stands about parallel to the fault line and within a few hundred feet of it. Neither structure gives any evidence of injury in any particular. It is impossible to say how they would have been affected had they stood at right angles with the fault line, and had they been intersected by it, as in the case of the earthen dams, previously described. It seems reasonable to suppose that, had they been sheared through vertically and offset, as in the case of the earthen dams, or had they been torn up as to their foundations, nothing worse would have occurred than the gradual loss of water from the reservoir, as both sections are designed with heavy gravity sections."

The quotation is inserted to give the facts; the accuracy of e conjectures as to what might have happened are open to question.

The Williams dam near San Jose, a concrete dam of gravity secon, not arched and about 80' high, while not immediately adjacent to e active fault line, was in a territory violently shaken in 1906. The m was not damaged. Mention has already been made of the action of the m in the 1918 San Jacinto earthquake.

The Hemet dam, referred to above as enduring the San Jacinto rthquake of 1918 without damage, is a very heavy gravity dam, of rubble sonry, arched in plan. At the time of the earthquake it was 121 feet gh, about 100 feet thick at the base and approximately 35 feet thick at e top. This height of this dam was later increased, and a detailed amination made previous to the addition, but subsequent to the earthquake, closed no cracks.

A multiple-arch dam cannot be said to be of the same unit type the heavy Crystal Springs dam, nor of the Searsville or Williams dams. the other hand, the effective destructive force on the multiple-arch e would be less than on the gravity type, on account of smaller mass.

Some computations on the stresses likely to be induced in the rious structural units comprising the Lake Hodges dam in a destructive rthquake have been made, but they are regarded as qualitative rather an as possessing any quantitative accuracy.

An earthquake shock of destructive intensity, with a direction and down stream, might crack the arch barrels, but would probably not stroy them. The principal danger would be in the additional stresses duced in the high unreinforced buttress walls. These would undoubtedly further cracked; they might fail, due to lack of reinforcing and proper acing.

A shock of major intensity, with a direction perpendicular to e line of the stream bed, or parallel to the face of the dam, would, in opinion, be likely to cause more damage than one in the other direc- on. Such a shock would cause the buttresses to vibrate, and tear out e light and widely spaced bracing. These buttresses are of varying ights; consequently, they will vibrate with different periods, and fferential motion must result, unless the bracing is of such strength to cause them to act together. The existing bracing has no such rength, and, in my opinion, would probably be torn out.

A dam is a different type of structure from a building, from the andpoint of earthquake resistance. In the usual building, the shock is lied to the structure in one horizontal plane. In the case of a dam, shock is presumably applied at all points of its height, and at both s. The amount and direction of the shock at both ends of the dam may ay not be simultaneously the same. The distribution of forces is efore much more complex than in the case of a building, and, with the sent state of our knowledge on the subject, none but rough calculations he stresses induced in the structure are warranted.

My studies of the construction of the Lake Hodges dam lead me the conclusion that, to provide reasonable security against earthquakes major intensity, the buttresses should be stiffened and braced by a tem of portal bracing. Such bracing, properly and consistently designed constructed, would make the dam reasonably safe from failure due to thquake.

To give a multiple arch dam the proper resistance against an thquake of a major intensity, which would cause the buttresses to rate in a plane parallel to the face of the dam, the buttresses should braced crosswise of the stream by ties made capable of resisting bending esses. The various buttresses might, for economy, be braced in pairs, ilar to the method used in bracing a high viaduct. It is probable that s bracing could be designed to resist the stresses due to an earthquake eleration as high as that of 1/3 of gravity, without producing abnormal tions. The sections of the buttresses should be well reinforced between h lines of bracing. In the Lake Hodges Dam, since the buttresses are reinforced, a proper provision for resisting destructive earthquakes ld mean that the lines of bracing introduced would have to be at closer ervals than if the buttresses were reinforced. For the same reason, the buttresses should be stiffened by horizontal beams attached to each side of the buttresses, and connecting the lines of bracing. Professor Derleth, in his report, has sketched such a system of bracing. It is of the type that I would recommend. I have made no attempt to calculate either the sizes required for such bracing or the probable cost.

Such bracing would, in a structural sense, also remove the dam from its present proximity to the danger zone, and place it within the field of safety.

To sum up my report on the Lake Hodges Dam, then, I find

(1) The arches are safe against failure from the ordinary loads of dead weight and water. They are reasonably safe against failure from earthquake, but might be severely cracked.

(2) The buttresses, as high unreinforced walls, will probably continue to carry their loads of dead weight and water. Their factor of safety is not as large as the importance of the dam warrants. Their effectiveness, in this respect, is definitely decreased by the cracks which exist.

(3) The present cracks are probably almost entirely due to shrinkage of the concrete, and to temperature changes. Except for weakening the effectiveness of the lateral bracing, these cracks are not found to be dangerous.

(4) The buttresses, as they are now braced, are not sufficiently strong to withstand a destructive earthquake. It seems probable that such an earthquake would cause them to collapse.

(5) The recommendation is made that these buttresses be braced; primarily from the standpoint of earthquake resistance, but also to give

e dam a safety factor commensurate with other important structures upon
ose stability the lives of many people depend.

Respectfully submitted,

(Signed) HENRY D. DEWELL
Consulting Engineer

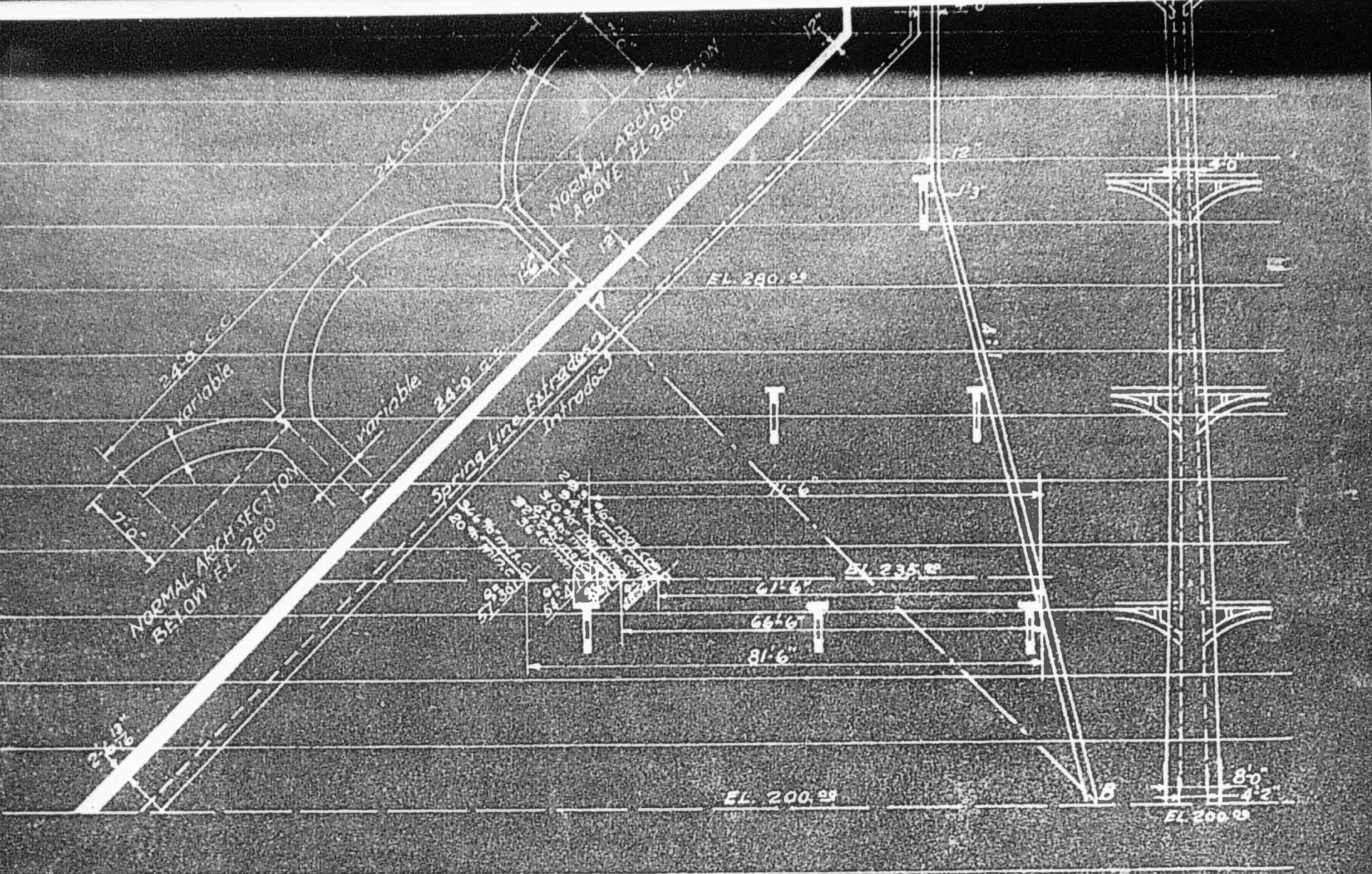


DIAGRAM OF BUTTRESS, LAKE HODGES DAM
SAN DIEGO COUNTY, CALIFORNIA
To accompany Report of
Henry D. Dewell, Consulting Engineer
Dated March 12, 1925

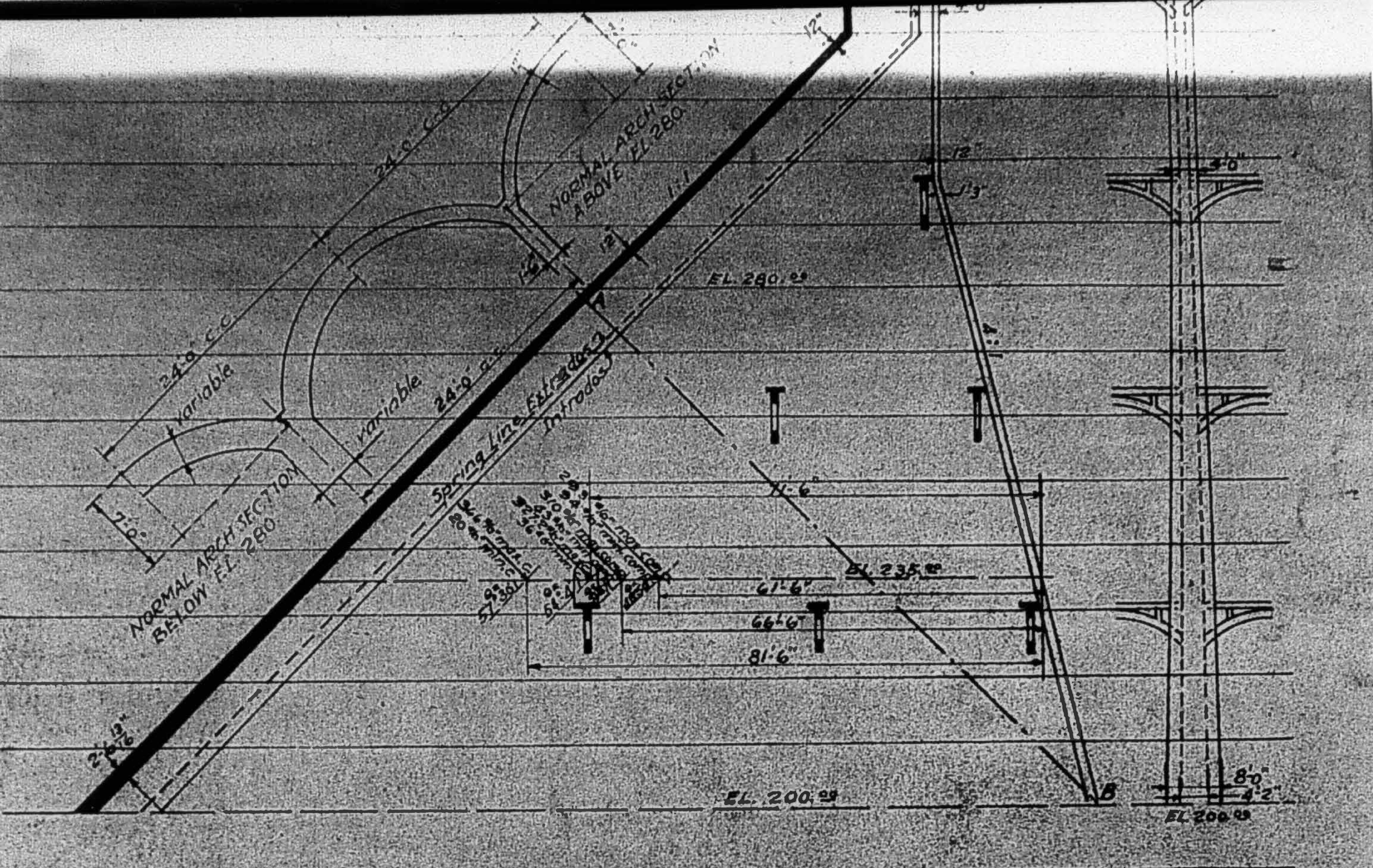


DIAGRAM OF BUTTRESS, LAKE HODGES DAM
SAN DIEGO COUNTY, CALIFORNIA
To accompany Report of
Henry D. Dewell, Consulting Engineer
Dated March 12, 1925

COMPUTED SOIL PRESSURES FOR DAM WHEN ASSUMED TO BE DIVIDED BY CRACK INTO TWO PORTIONS.

By J.A.H. Brahtz

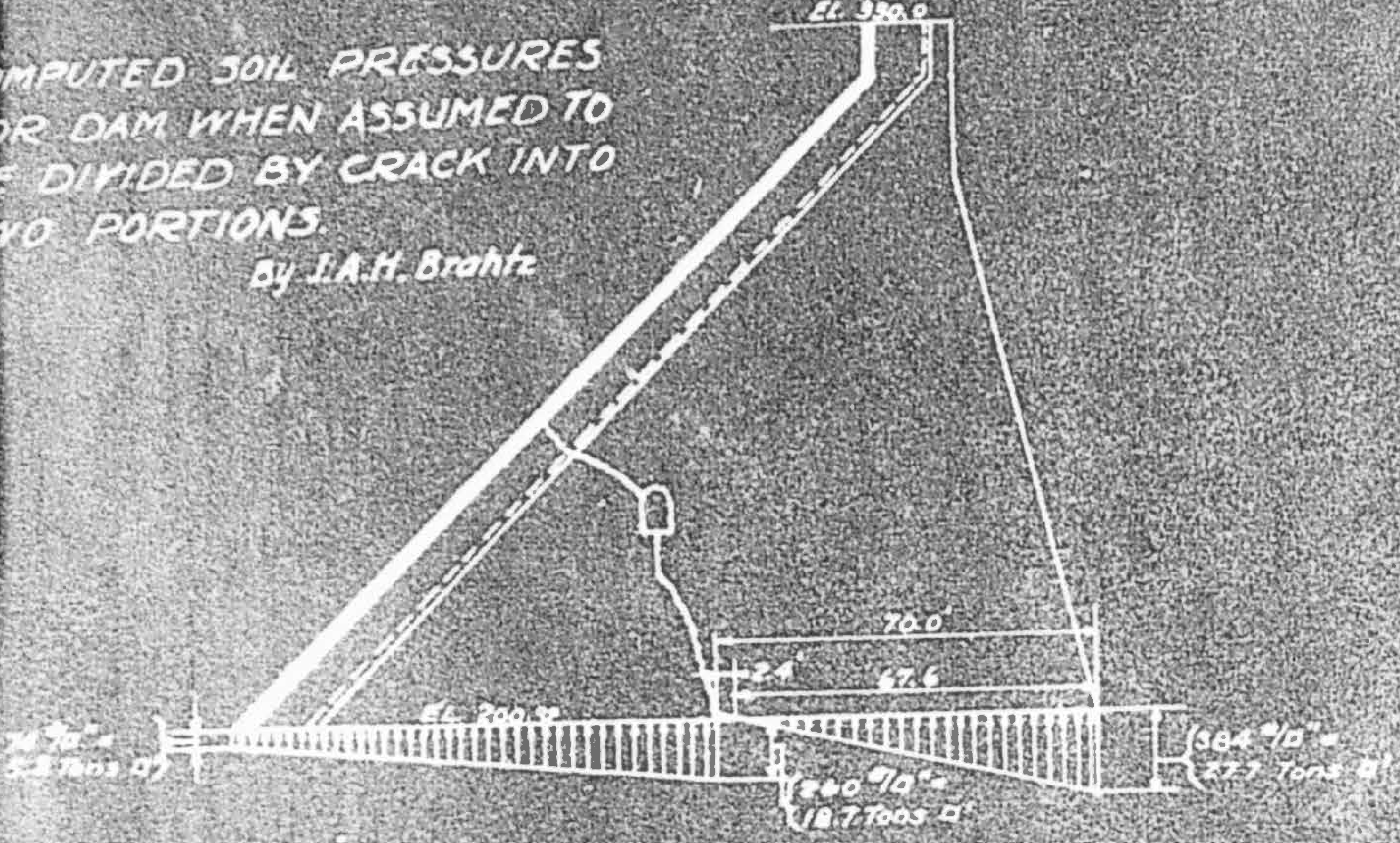


Fig. 2

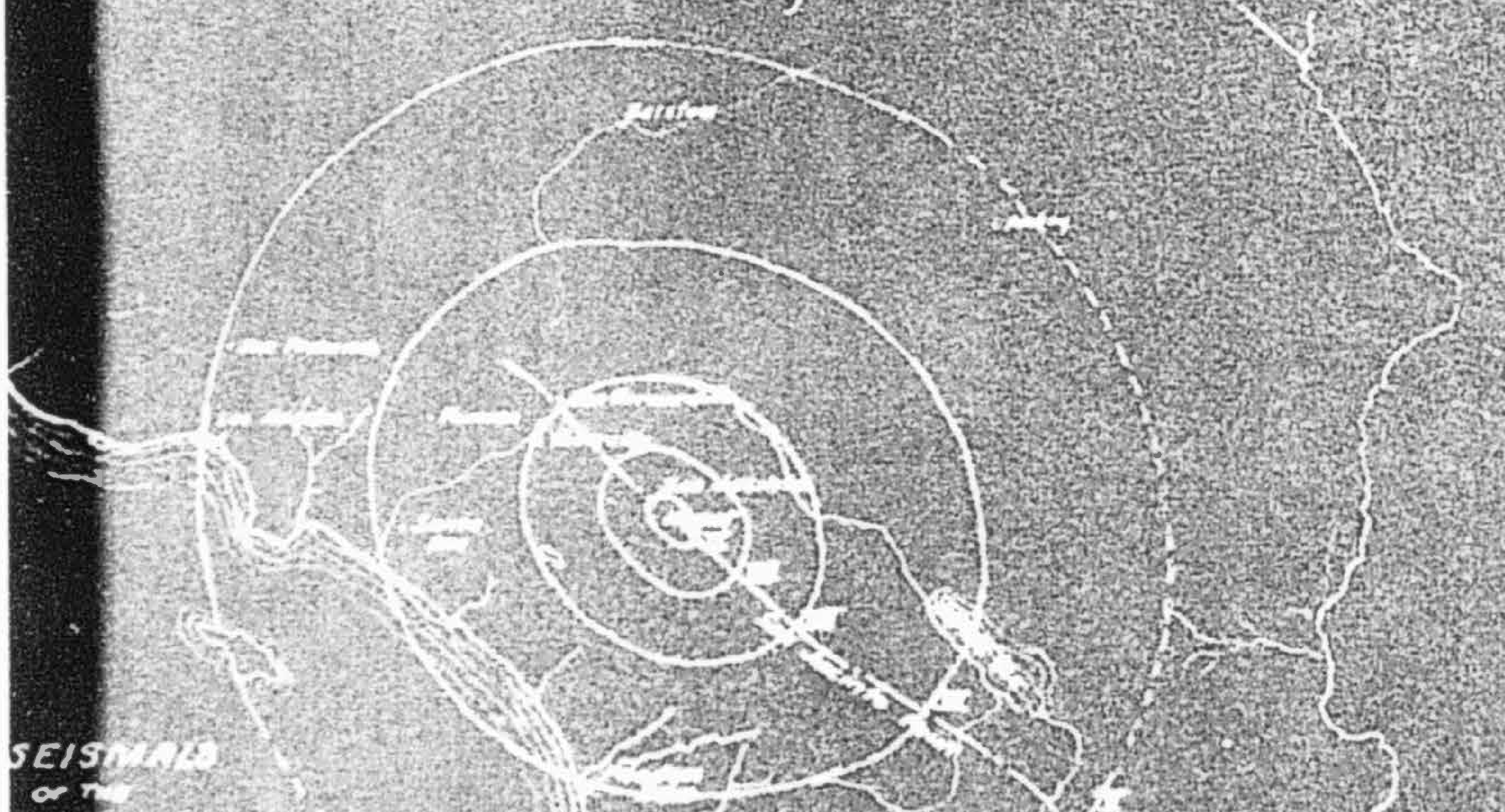


Fig. 3

LAKE HODGES DAM
SAN DIEGO COUNTY, CALIFORNIA.

To accompany Report of
Henry D. Dewell, Consulting Engineer
Dated March 12, 1929.

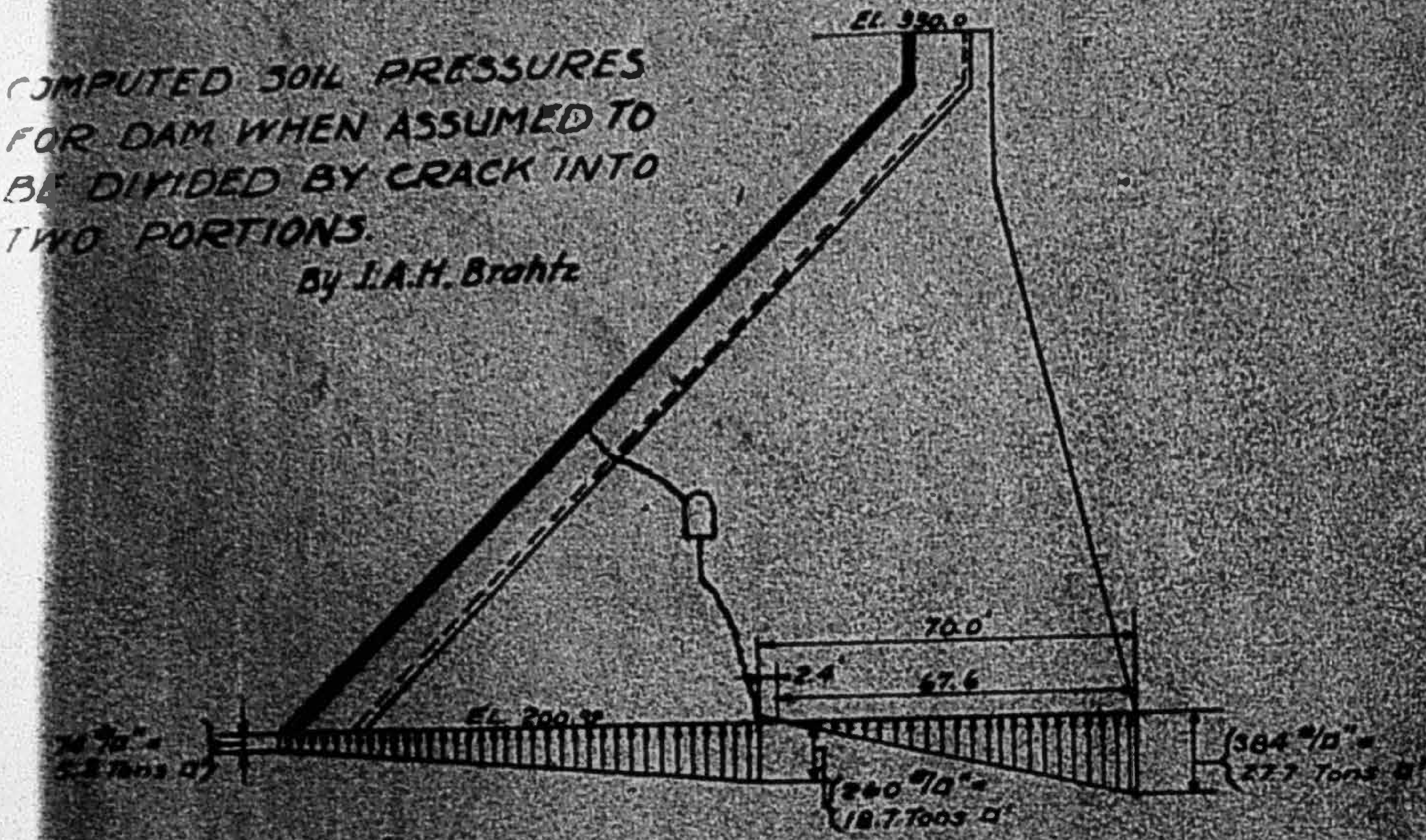


Fig. 2

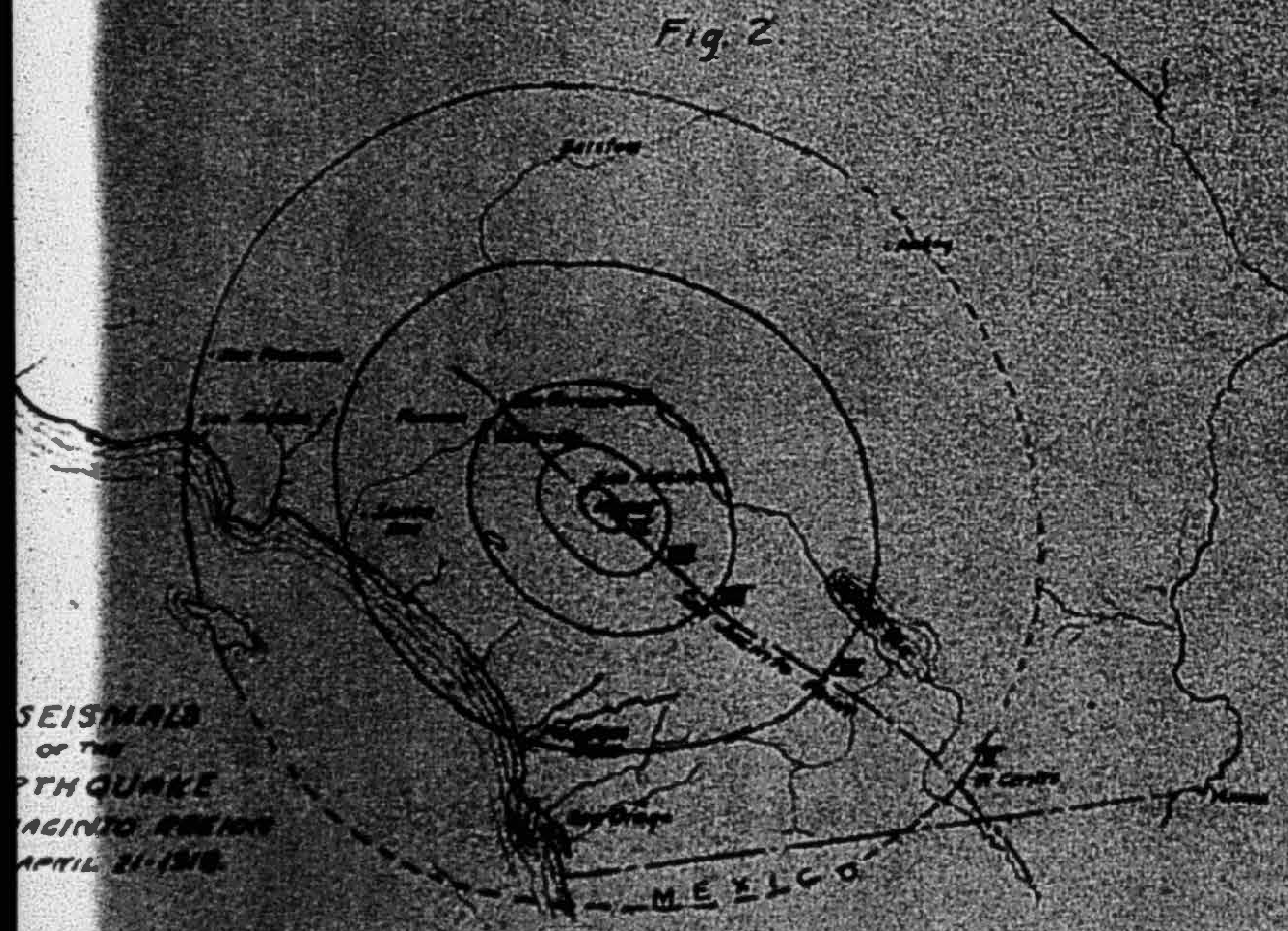


Fig. 3

LAKE HODGES DAM
SAN DIEGO COUNTY, CALIFORNIA.

To accompany Report of
Henry D. Dewell, Consulting Engineer
Dated March 12, 1929.

Ed Fletcher Papers

1870-1955

MSS.81

Box: 38 Folder: 9

Business Records - Reports - Hyatt, Edward - "Reports of Consulting Board on Lake Hodges Dam"

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