COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
VENTURA, CALIFORNIA

REVIEW OF MATILIJA DAM
1967

Prepared By
BECHTEL CORPORATION
SAN FRANCISCO

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BECHTEL CORPORATION

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1. INTRODUCTION, CONCLUSIONS AND RECOMMENDATIONS

1.1 General.

Matilija Dam, a concrete arch structure, near Ojai, California (Drawing 5716-67-1), has been under study and review by Bechtel Corporation of San Francisco since August 20, 1964. Shortly after the beginning of the study it was realized that alkali-aggregate reactivity of the concrete was responsible for cracking of the structure, and that the expansion of the concrete due to the reactivity lowered the safety factors to unacceptable levels.

Abutment yield measuring devices were then installed in both abutments to measure the yield, if any, of the abutments due to the movement of the structure because of water loads and continued concrete expansion. In a report issued in February 1965 and titled "Review of Matilija Dam", Bechtel Corporation recommended monitoring the dam and its abutments before any modifications of the dam were effected. It was further recommended that such program of monitoring should continue until sufficient time had elapsed for the newly installed abutment meters to yield meaningful data as to the behavior of the abutments. At the end of such a period of observation, recommendations would then be made as to how and to what extent the dam should be modified and made safer.

However, in the Spring of 1965 the County of Ventura, Department of Public Works, Ventura, California, decided to modify the dam immediately in order to permanently lower the maximum water surface elevation. In July 1965, Bechtel Corporation was authorized by the County to proceed with the design and the preparation of construction drawings for modifying the dam. Bechtel was further authorized to conduct engineering tests and make observations over a period of two years. This report summarizes the results of these tests and the conclusions drawn from the observations. It presents Bechtel's recommendations on the adequacy and safety of Matilija Dam in its present altered configuration, and recommendations for the future monitoring and observation of the dam.

1.2 Background and Scope.

Two reports have been issued by Bechtel Corporation on the review of Matilija Dam. The first report titled "Preliminary Report on Review of Matilija Dam" was issued in October 1964 and the second, titled "Review of Matilija Dam", was issued in February 1965.

The studies discussed in the reports established that deterioration of the concrete in the dam has occurred due to alkali-aggregate reaction and that expansion, cracking and disintegration of the concrete have been particularly severe in the upper 25 feet of the structure. The geologic studies and a study of past records raised questions as to the stability and reliability of the abutment rock, particularly that of the left abutment. A system of abutment yield measuring devices was installed and a program of surveillance was begun. In order to increase the safety factors immediately and remove the highly reactive portions of the dam, a "notch" was cut in the dam, between Stations 1+75 and 4+55, creating a new spillway with a crest at Elevation 1095, thus permanently lowering the normal maximum pool level to Elevation 1095 from its previous Elevation 1125. (See Drawing 5716-67-2.) The maximum reservoir level at the maximum probable flood of 70,000 cfs was thus reduced to Elevation 1113.7. The hydraulics of the new spillway were studied and are reported in Section 2.

In addition to constructing the new spillway, an additional 36-inch diameter valve was installed at Elevation 1025± on the existing 36-inch diameter outlet pipe. The new valve discharges on the concrete apron immediately downstream of the dam. (Drawing 5716-67-2.)

Various tests were conducted, whereby the reservoir was raised and lowered in controlled stages to observe its effects on the movements of the dam and abutments. The abutment movements indicated by the meters are discussed in Section 3.

Section 4 presents the studies conducted on the movement of the dam itself. These movements were measured at survey markers installed at various locations on the dam.

During the past two years, further study was made on the expansion of the concrete, both by closely observing the structure periodically and by performing volume-change tests on concrete core samples obtained in the earlier investigations by Bechtel. This report discusses the state of the concrete in Section 5.

Section 6 presents the stress analyses of the dam, performed by assuming various critical loading conditions and making other assumptions regarding abutment yield and other dam movements.

1.3 Conclusions.

1.3.1 General. After completing the study of the data gathered since the new spillway was constructed two years ago, it is possible to draw conclusions and make recommendations regarding the present condition and future safety of Matilija Dam. These conclusions and recommendations as well as all the plots and Bechtel's analyses have been discussed with the Board of Consultants, consisting of Messrs. Julian Hinds and Roger Rhoades and Dr. R. W. Carlson. The Board has summarized its opinions of the studies in a letter to Bechtel Corporation, dated August 14, 1967, a copy of which is included in Section 7.

Generally, the studies show that there is no reason to believe that the performance of the dam with respect to safety would be unsatisfactory in the foreseeable future. The summary of conclusions on various aspects of the studies follows:

1.3.2 Abutment Deformation. The meters installed in both abutments and which have been read and the readings analyzed for two years, while filling and emptying the reservoir in controlled cycles, indicate that the abutment rock, within the length of the meters, is adequately stable. The measured deformations ranged from a few thousandths to a few hundredths of an inch, far too small to be of consequence vis-a-vis the stresses in the dam. There was negligibly small permanent inelastic deformations indicated in both the direction of the arch thrust and in a downstream direction. One meter, the bottommost one in the left abutment, did indicate a slight permanent expansion, probably attributable to the rebound of the abutment rock due to decreasing the average loading due to the reservoir.

1.3.3 Structural Analyses. Structural analyses of the dam in its present altered configuration, assuming the most adverse reservoir, temperature drop, silt and seismic loading conditions showed that maximum stresses did not exceed 960 psi in compression and 130 psi in tension. These stresses are within the capability of the dam concrete. The analyses also showed that yield of the abutments much greater than that indicated by the abutment meters could be accommodated before stresses would become unacceptably high.

1.3.4 Concrete Deterioration. There is evidence of continued cracking and enlarging of existing cracks in the concrete remaining above Elevation 1095, which is the elevation of the new spillway crest. Volume change tests on specimens of the concrete taken in 1965 above Elevation 1095 have also shown that concrete expansion is continuing at a rapid rate due to the autogenous growth of the concrete from which the specimen was obtained. However, the continued deterioration of the concrete above Elevation 1095 presents no hazard to the integrity of the dam itself. Recent careful visual examination of the concrete below Elevation 1095 showed no evidence of concrete cracking, expansion or deterioration.

1.3.5 Movement of the Dam. At the present time it cannot be determined conclusively whether the dam arch is still moving upstream due to reactivity. The survey plates, which were located at the crown rusted away or were perforce removed when the new spillway was constructed. New plates have been installed recently (March 1967) but sufficient time has not elapsed to indicate whether any movement is still in progress. The structural analyses investigated the effect of crown movement upstream on the stresses in the dam. The analyses showed that total upstream movement of the crown at Elevation 1095 of $2\frac{1}{2}$ inches would still be acceptable, even when combined with the most unfavorable assumptions of an empty reservoir and temperature rise.

1.3.6 Stability of the Concrete Wingwalls. The stability of the concrete wingwalls at the ends of the new spillway which support the new footbridge was also analyzed. Based on the present configuration of cracking, the walls are safe even under maximum spillway discharge conditions with the reservoir at Elevation 1113.7. However, careful observations should be continued to ascertain that any additional cracking which may occur in the future will not affect the integrity of the walls as bridge abutments. It should be emphasized that the condition of the walls does not influence the safety of the main dam, since these walls no longer act as part of the arch dam itself.

1.3.7 Necessity for Future Observations and Testing. It should be noted that any assessment of the condition of the dam at present, must be based on its past behavior. Careful visual observations, monitoring of the abutment meters and measurements of the movement of the survey plates must be continued. Substantial departures in the future from the pattern established up to the present, should they occur, would have to be recorded and analyzed immediately. Also periodic testing of the dam concrete is of extreme importance, so that any deterioration, any development of micro-fracturing or the beginning of accelerated autogenous growth will not go unnoticed.

1.4 Recommendations

Following is a summary of Bechtel's recommendations regarding Matilija Dam:

1.4.1 It is recommended that the dam remain in service and be monitored in accord with good practice.

1.4.2 In order to properly monitor the future performance of the dam it is further recommended that:

a. The abutment yield measuring devices be read, recorded and plots similar to those shown in this report prepared. The devices should be read at least every two weeks, together with reservoir elevations and temperatures at the time the readings are taken.

b. Horizontal movements of all the survey plates, including the ones recently installed near the spillway crest, be measured and the results plotted. Such measurements should be performed at least once every three months, and immediately following the occurrence of any unusual events such as an earthquake or a major flood.

c. Not later than five years from now and at intervals of five years or less thereafter, strength, petrographic examination and soniscope tests be performed on the dam concrete. Should measurements performed under a. and b. above indicate any significant movements of the dam, the concrete testing noted herein should be initiated immediately.

d. All records, data and results of tests be plotted, evaluated, and any significant deviations from previous patterns be promptly analyzed.

2. HYDROLOGY AND HYDRAULICS

In the report, "Review of Matilija Dam", February 1965, Bechtel estimated the "maximum probable" peak flow to be between 70,000 and 80,000 cfs.

As stated earlier the crest of Matilija Dam was lowered to Elevation 1095 between Stations 1+75 and 4+55. A 20-foot wide by 8-foot high debris deflector, installed at the center of this section, protects the downstream outlet of the 48-inch sluice.

The modified section of the dam was rated under the following assumptions:

a. The crest was horizontal, flat and relatively smooth.

b. Silt had deposited behind the dam to Elevation 1014.0.

c. The upstream approach velocity distribution was uniform across the modified section.

The modified crest can pass: 16,000 cfs (maximum recorded flow) with the reservoir water surface at Elevation 1102.7, or 7.7 feet above the crest; and 70,000 cfs (approximate peak of the "probable maximum" flow) with the reservoir at Elevation 1113.7, or 18.7 feet above the crest.

The crest modification will reduce the maximum combined capacity of the two outlet pipes to about 630 cfs during the "probable maximum" peak flow. However, a new 36-inch diameter outlet valve was installed during the modification of the dam. This new valve discharges on the concrete apron downstream of the dam at Elevation 1025±.

It is also expected that energy dissipation will be improved by the modified configuration of the dam and the danger of erosion beyond the downstream apron will be practically eliminated.

3. ABUTMENT INSTRUMENTATION

3.1 Abutment Yield Measuring Devices

The two reports preceding this one established the need of investigating the movement of the abutments in response to the action of thrust, shear and moment resulting from the arch. Eight deformation meters were designed and installed in the abutments in order to establish the direction and magnitude of abutment movement at various locations and, most importantly, to determine whether or not there are indications of abutment yield, or inelastic deformations, which would be of extreme importance in calculating the stresses within the dam and in evaluating the stability of the abutments.

The type, location and methods of installing the meters were discussed in detail in the report "Review of Matilija Dam", February 1965. Their locations are shown on Drawing 5716-67-2 of this report. Table 3.1 shows various data pertaining to the meters.

TABLE 3.1

LOCATION AND DETAILS OF ABUTMENT METERS

	Drill Hole	Ground Elevation	Bearing	Angle from Horizontal	Inclined Length Ft.	Horizontal Length Ft.
Left Abutment	1L	1112.5	N82°E	20°	40.2	37.8
	2L	1070.3	N81°E	30°	50.2	43.5
	2aL	1071.9	N36°E	5°	35.3	35.2
	3L	982	N55°E	30°	55.2	47.8
Right Abutment	1R	1113	S40°E	5°	30.1	30.0
	2R	1049	S40°E	10°	34.5	34.0
	2aR	1051	S 5°W	5°	37.4	37.3
	3R	982	S40°E	15°	51.0	49.2

Briefly, the 8 measuring devices are Carlson joint meters, installed in 4-inch diameter holes, four in the left abutment and four in the right. Six meters (DH-1L, 2L, 3L and DH-1R, 2R, 3R) were installed at three elevations in the direction of arch thrust and two meters (DH-2aL and 2aR) were installed at 45° to the radial direction to detect any slippage of rock strata.

In subsequent paragraphs, plots of abutment deformation will be described and the data discussed and analyzed.

3.2 Control of Reservoir Operation.

Readings of the strain meters were taken and plotted versus reservoir elevation. For reasons that will be discussed subsequently, it was decided to fill and empty the reservoir in approximate 10-foot increments of elevation and keep it at each level for several days. The strain meters were read 4 or 5 times a day at constant reservoir while recording the temperatures at the same time. There were, of course, deviations from the ideal program of reservoir control because any such control was dependent on weather conditions. As far as practicable, an attempt was made to reproduce the same cycle of raising and lowering the reservoir, thus holding constant as many factors as possible. Figure 3.1 presents a plot of reservoir elevations versus time. As shown on the plot, there were three runs of controlled reservoir raising and lowering.

3.2.1 Run No. 1 - January through April 1966.

3.2.2 Run No. 2 - October 1966 through January 1967.

3.2.3 Run No. 3 - February through May 1967.

The plot shows the approximate 10-foot reservoir increments. The descending legs of the three runs were almost identical, except for the time of year at which they occurred. The ascending legs, however, were much more difficult to control, since they depended largely on natural inflow. The ascending leg of Run No. 2 digressed farthest from the planned control stages. This was partly responsible for the non-correlable deformation plots for the various meters, as will be discussed subsequently.

3.3 Data and Analyses.

3.3.1 Type of Plots. The data obtained from the strain meters and temperature readings were plotted in four different forms:

a. Plot of Reservoir Elevation versus Time (Fig. 3.1).

b. Plots of Deformation versus Temperature (Fig. 3.2).

c. Plots of Deformation versus Reservoir Elevation (Figs. 3.3.through 3.10).

d. Plots of Deformation versus Time (Figs. 3.11 through 3.18).

The most important of the plots are those of Deformation versus Reservoir Elevation, because by analyzing them it is possible to establish the correlation, if any, between the movement of the abutment rock and reservoir elevation and to determine whether the abutments deform elastically or whether there is a permanent inelastic deformation in any direction after several reservoir cycles.

It must be noted here that the "deformations" measured by the meters are those of the abutment rock within the length of the meter.

The meter is attached to the dam concrete on one end of a steel rod and measures the deflection of the dam at that point relative to the end of the rod anchored inside the abutment. See Table 3.1 for the lengths of the rods at the various meters. It was considered that the most significant rock movements would be detected by the meters as installed.

3.3.2 Normalized Deformation Readings. Initially, deformations were plotted directly against reservoir elevation. However, it was soon apparent that plotting the readings in this manner involved a scatter of points which rendered the plots almost meaningless. It was believed that temperature changes were affecting the readings and it was decided to normalize the readings to a constant temperature, when the reservoir was held at a constant elevation. The temperature chosen for the normalization was $60^{\circ}F$, which is close to the mean temperature throughout the year, thus decreasing the necessity to extrapolate.

In order to normalize the readings, the deformations were read several times a day for 4 or 5 days when the reservoir was held at a constant level. The temperature was also recorded at the time of each reading and then the readings were plotted versus temperatures. (Figure 3.2.) Through the points thus obtained, the most probable line was drawn, and the intersection of this line with the $60^{\circ}F$ line gave the normalized deformation at that particular reservoir level.

Figure 3.2 shows an example of Deformation versus Reservoir Elevation plot. It also shows the spread of points obtained when the readings are not normalized. As may be seen the normalized readings are much more consistent and meaningful. Figure 3.2 also shows the

Deformation versus Temperature plots at various elevations from which the normalized readings shown on the same figure were obtained. Similar plots were made for all other meters, but are not presented herein.

3.3.4 Analyses. The meter readings and the wealth of data accumulated during the two-year observation and surveillance programs have been meaningful and necessary to arrive at conclusions regarding the present condition and future safety of the dam. In analyzing the readings it can be stated that in the last two years there has been no indication at any of the meters of significant permanent inelastic deformation of the abutments that should cause concern. The measured deformations, elastic and inelastic, were very small, ranging from a few thousandths to a few hundredths of an inch. The structural analyses show that movements of magnitudes much greater than those indicated by the abutment meters are required before stresses are seriously increased and safety factors reduced to unacceptable values.

In analyzing the various plots, it is apparent that of the two abutments, the left exhibited more movement than the right. Further, of all the meters, meter DH-1L (Figure 3.3) which is the uppermost on the left abutment, showed the maximum movement. Meter DH-1R (Figure 3.7), the uppermost on the right abutment, showed the maximum movement for that abutment. For both DH-1L and DH-1R, the plots appear to be more erratic than those of the remaining meters. Particularly erratic is the plot showing the relationship between the meter reading and reservoir elevation for the filling cycle of Run No. 2.

These anomalies may be attributable to the fact that the uppermost meters were installed at Elevation 1112.5 for DH-1L and

Elevation 1113 for DH-1R, which are above the normal maximum pool (El. 1095) and just at maximum flood level during a maximum probable flood (El. 1113.7). This would render them most susceptible to temperature changes, further magnified by their exposed locations. Figures 3.11 and 3.15 are plots of Deformation versus Time for DH-1L and DH-1R. These plots appear to justify the explanation that temperature is in fact a probable cause of the anomalous behavior of the meters in that they appear to fluctuate generally with the seasons, in a more or less cyclic fashion. With certain exceptions, their behavior was not unlike that of Temperature versus Time shown on Figure 3.19. This type of behavior was evidenced for all three years for which the plots were prepared. During the warm months the dam expands and pushes into the abutment causing the meter to show compression and during the cold months the movement is reversed. The other meters did not exhibit such pronounced seasonal cyclic behavior.

The behavior of DH-1L and 1R is believed to be influenced by still another factor, that of continued expansion of the concrete remaining above Elevation 1095 (which is the crest of the new spillway). It was determined that the upper 4 or 5 lifts of the dam were those most seriously affected by the alkali reactivity. When the new spillway was constructed the restraint was removed and the expansion accelerated. This expansion undoubtedly affects the readings of DH-1L and 1R to a certain extent.

The anomaly of the behavior of the meters during the filling cycle of Run No. 2 is attributable to the fact that this cycle is quite different from the others. Figure 3.1 shows this difference, the reason for which is the difficulty of controlling the reservoir level since this is dependent on natural inflow conditions.

The behavior of Meters DH-1L and 1R are, therefore, believed to be not representative of the behavior of the dam, because of their high elevation and their being outside the region of influence of the main dam.

Meters DH-2L and 2aL (Figs. 3.4 and 3.5) and DH-2R and 2aR (Figures 3.8 and 3.9) show hardly any movement (about 0.005 inch). Thus it appears that there was negligible movement of the abutment near the mid-height of the dam. Stress analyses show that movements in the range of 2 inches of abutment yield at the approximate elevation of these meters would be required to induce unacceptably high tensile stresses in the dam. Of particular interest are Meters DH-2aL and 2aR which would show slippage of the dam downstream had any occurred. These meters indicate no significant movement (less than 0.005 inch).

Near the bottom of the dam, the right abutment moved a negligible amount (Figure 3.10) whereas the left abutment (Figure 3.6) shows a slight permanent movement where the rock appears to have expanded, or the dam to have moved slightly in a direction away from the abutment. As seen, there is approximately 0.02 inch expansion of Meter DH-3L when normalized readings for Run No. 1 are compared to those for Run No. 3 for the same reservoir level. This phenomenon could be attributed to the fact that when the spillway was lowered the average reservoir pressure over the two years of observation was permanently decreased. This decrease relieved some of the pressure on the abutment which allowed the rock to rebound somewhat and show the expansion as measured by the meter.

It is interesting to note that almost all of this rebound occurred between Run No. 1 and Run No. 2. Runs No. 2 and No. 3 are repetitive and show elastic deformation only.

3.4 Summary and Conclusions on Abutment Yield.

To summarize the above, Table 3.2 lists the various abutment movements as recorded by the meters for readings taken for Run No. 1 and Run No. 3 at the same reservoir level.

TABLE 3.2

SUMMARY OF ABUTMENT METER MOVEMENTS

All Movements in Inches

Left Abutment			Right Abutment		
Meter	Expansion	Compression	Meter	Expansion	Compression
DH-1L		0.06	DH-1R		0.025
DH-2L	0.005		DH-2R		0.005
DH-2aL	0.003		DH-2aR		0.003
DH-3L	0.02		DH-3R		0.015

These values are approximate and based on the assumptions discussed in the preceding paragraphs.

All the movements are, in fact, very small and range from a few thousandths to a few hundredths of an inch. The largest, 0.06 inch, at the uppermost meter in the left abutment, which for reasons discussed earlier, is not indicative of the behavior of the main dam itself.

One consistent indication shown by the table is that the left abutment meters, except DH-1L, appear to have expanded and the right abutment ones to have compressed. The values of expansion and compression are quite similar for the same elevations. This would indicate that the dam has moved slightly toward the right abutment. In the previous reports, it was stated that the dam appeared to be shifting toward the left abutment. It is possible that with the new spillway and the lowered reservoir, the dam is readjusting to its new configuration and is in fact shifting very slightly toward the right abutment.

In summary, it should be re-emphasized that all abutment deformations are very small and may very well be within the accuracy limits of the sensitive measuring devices. It is very important that monitoring and evaluation of the meter readings be continued in the future, which should preclude the possibility of slippage or inelastic deformation occurring unnoticed.

MATILIJA DAM
Abutment Yield Measurements
RESERVOIR ELEVATION VS TIME

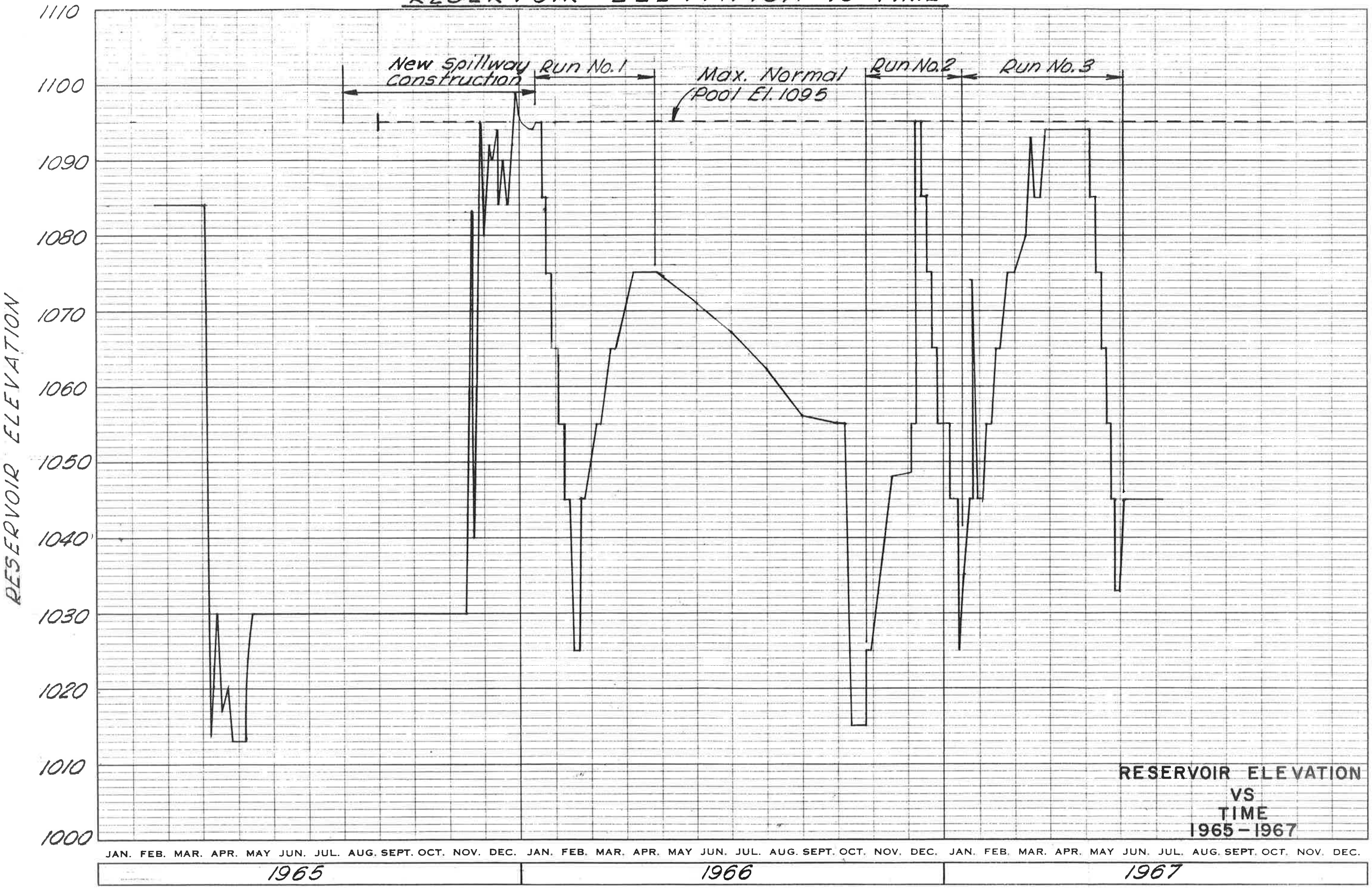
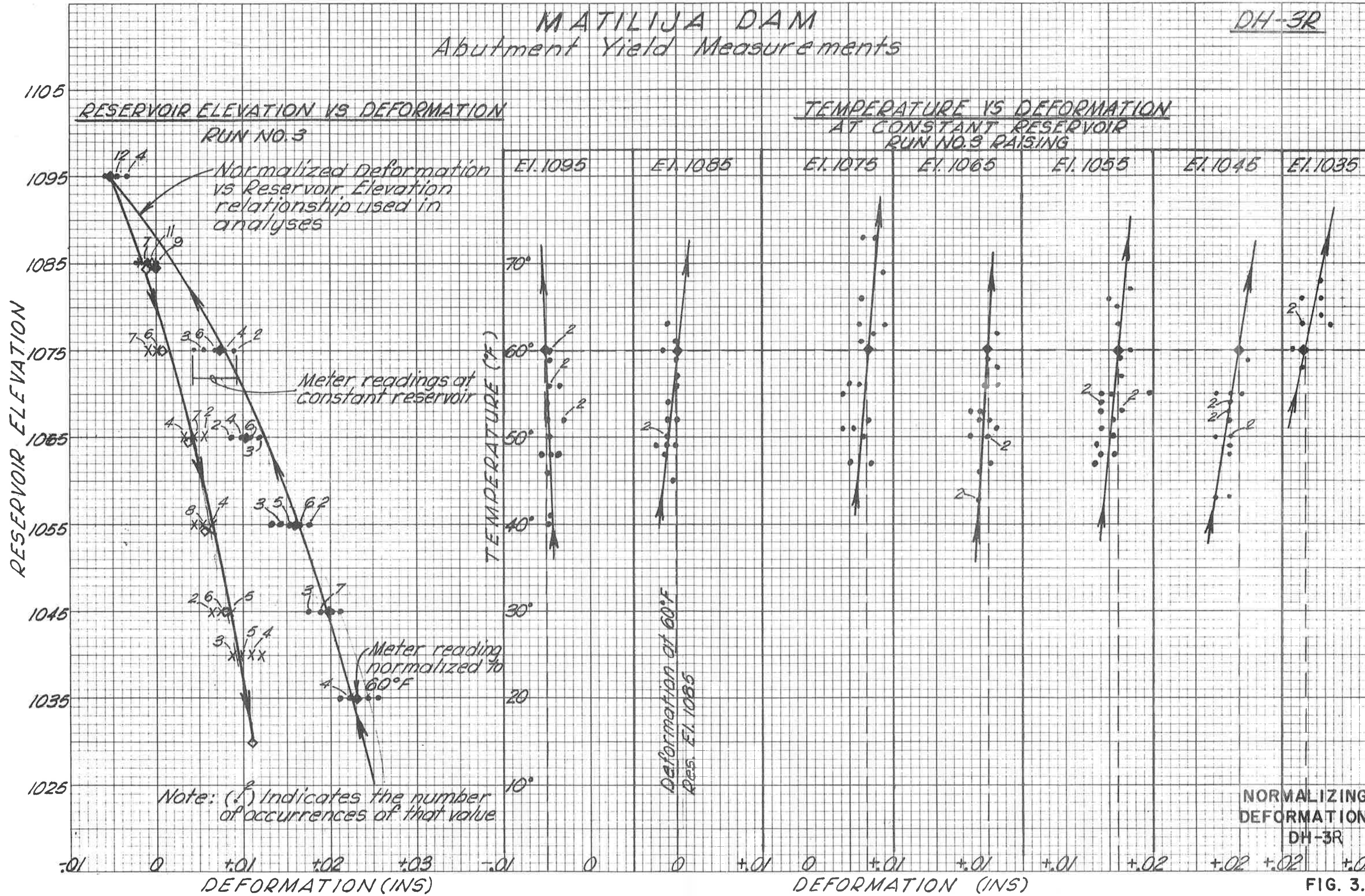


FIG. 3.1



MATILIJA DAM
Abutment Yield Measurements
DH-3R
RESERVOIR ELEVATION VS DEFORMATION
RUN NO. 3
Normalized Deformation vs Reservoir Elevation relationship used in analyses
Meter readings at constant reservoir
Meter reading normalized to 60°F
Note: (.²) Indicates the number of occurrences of that value
TEMPERATURE VS DEFORMATION
AT CONSTANT RESERVOIR
RUN NO. 3 RAISING
El. 1095
El. 1085
El. 1075
El. 1065
El. 1055
El. 1045
El. 1035
TEMPERATURE (°F)
Deformation at 60°F
Res. El. 1085
RESERVOIR ELEVATION
DEFORMATION (INS)
DEFORMATION (INS)
NORMALIZING
DEFORMATION
DH-3R

FIG. 3.2

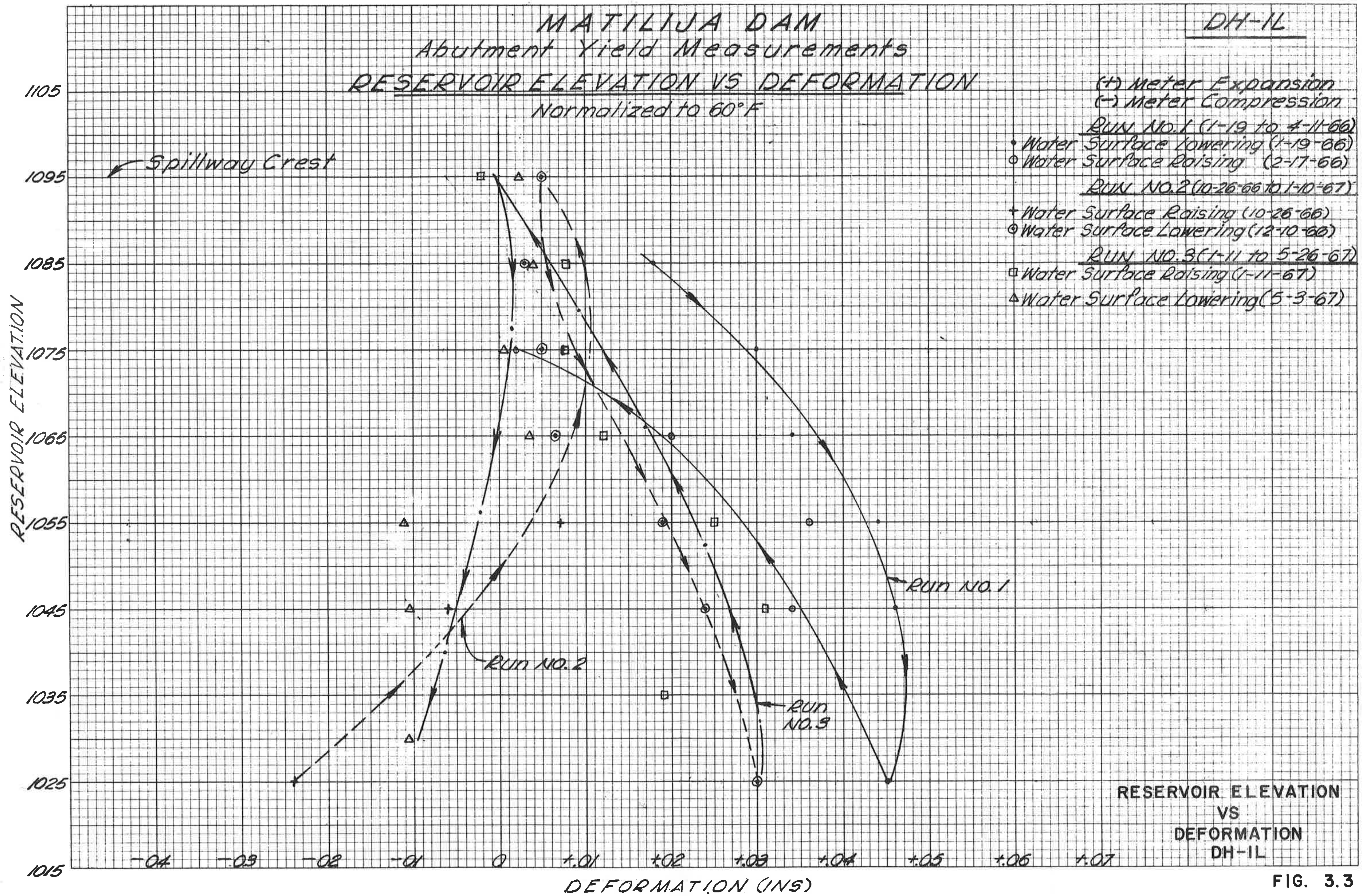

MATILIJA DAM
Abutment Yield Measurements
RESERVOIR ELEVATION VS DEFORMATION
Normalized to 60°F
DH-IL
(+) Meter Expansion
(−) Meter Compression
RUN No.1 (1-19 to 4-11-66)
Water Surface Lowering (1-19-66)
Water Surface Raising (2-17-66)
RUN NO.2 (10-26-66 to 1-10-67)
Water Surface Raising (10-26-66)
Water Surface Lowering (12-10-66)
RUN NO.3 (1-11 to 5-26-67)
Water Surface Raising (1-11-67)
Water Surface Lowering (5-3-67)
Spillway Crest
RUN NO.1
RUN NO.2
RUN NO.3
RESERVOIR ELEVATION
1105
1095
1085
1075
1065
1055
1045
1035
1025
1015
−.04
−.03
−.02
−.01
0
+.01
+.02
+.03
+.04
+.05
+.06
+.07
DEFORMATION (INS)
RESERVOIR ELEVATION VS DEFORMATION DH-IL

FIG. 3.3

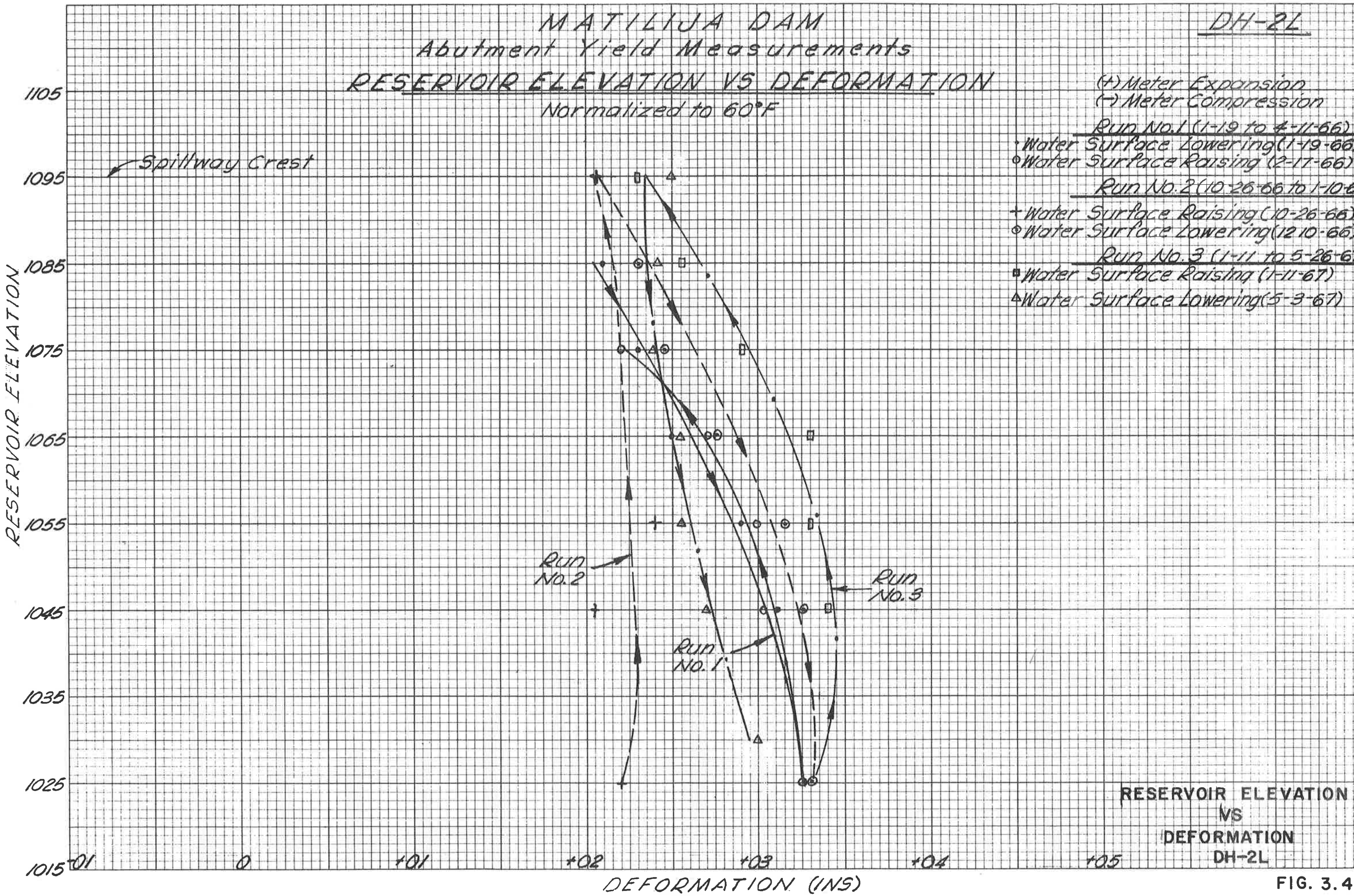

MATILIJA DAM
Abutment Yield Measurements
RESERVOIR ELEVATION VS DEFORMATION
Normalized to 60°F
DH-2L
(+) Meter Expansion
(−) Meter Compression
Run No.1 (1-19 to 4-11-66)
· Water Surface Lowering (1-19-66)
⊙ Water Surface Raising (2-17-66)
Run No.2 (10-26-66 to 1-10-67)
+ Water Surface Raising (10-26-66)
⊙ Water Surface Lowering (12-10-66)
Run No.3 (1-11 to 5-26-67)
□ Water Surface Raising (1-11-67)
△ Water Surface Lowering (5-3-67)
Spillway Crest
Run No.1
Run No.2
Run No.3
RESERVOIR ELEVATION
1105
1095
1085
1075
1065
1055
1045
1035
1025
1015
-.01
0
+.01
+.02
+.03
+.04
+.05
DEFORMATION (INS)
RESERVOIR ELEVATION VS DEFORMATION DH-2L

FIG. 3.4

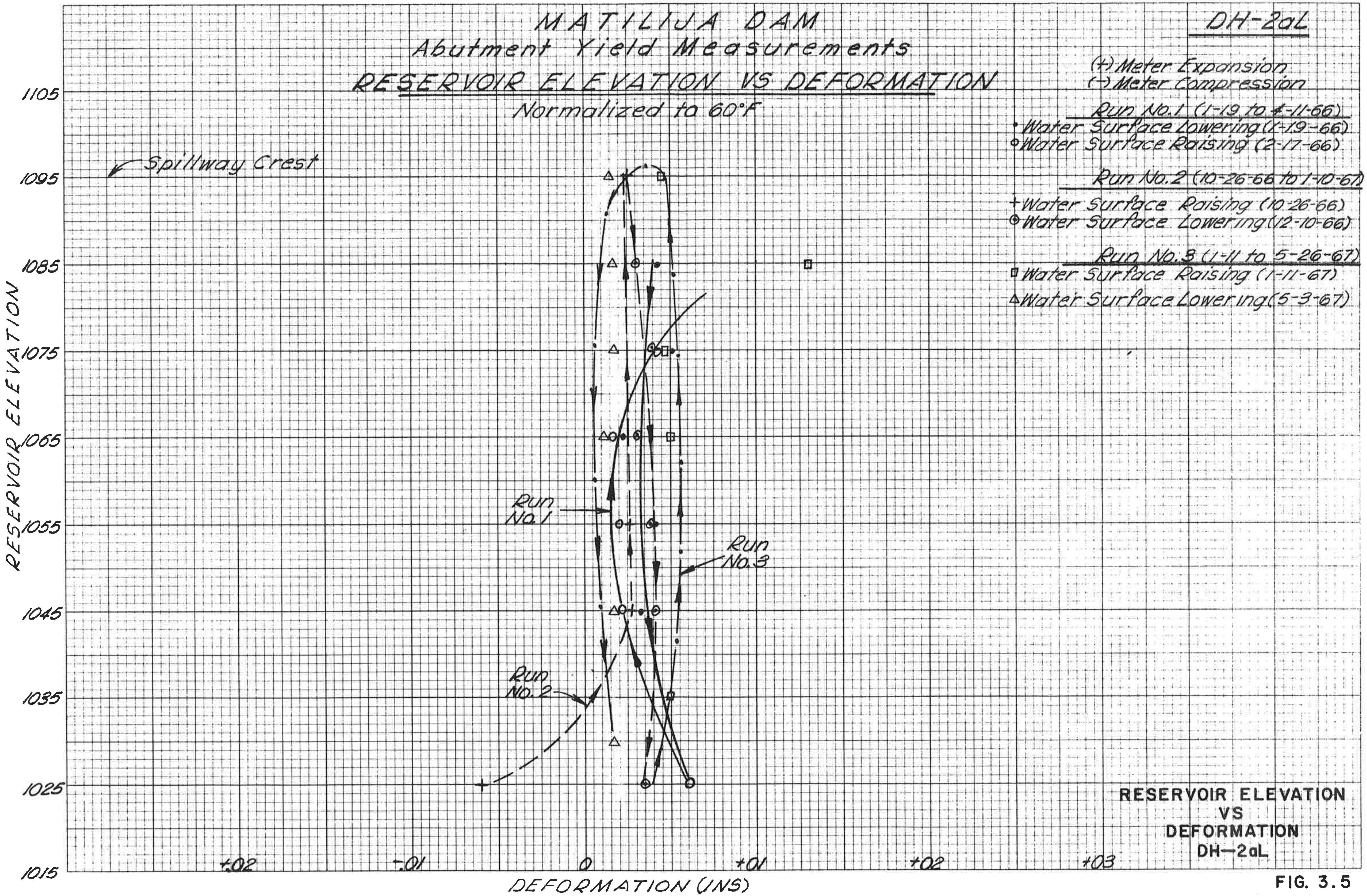
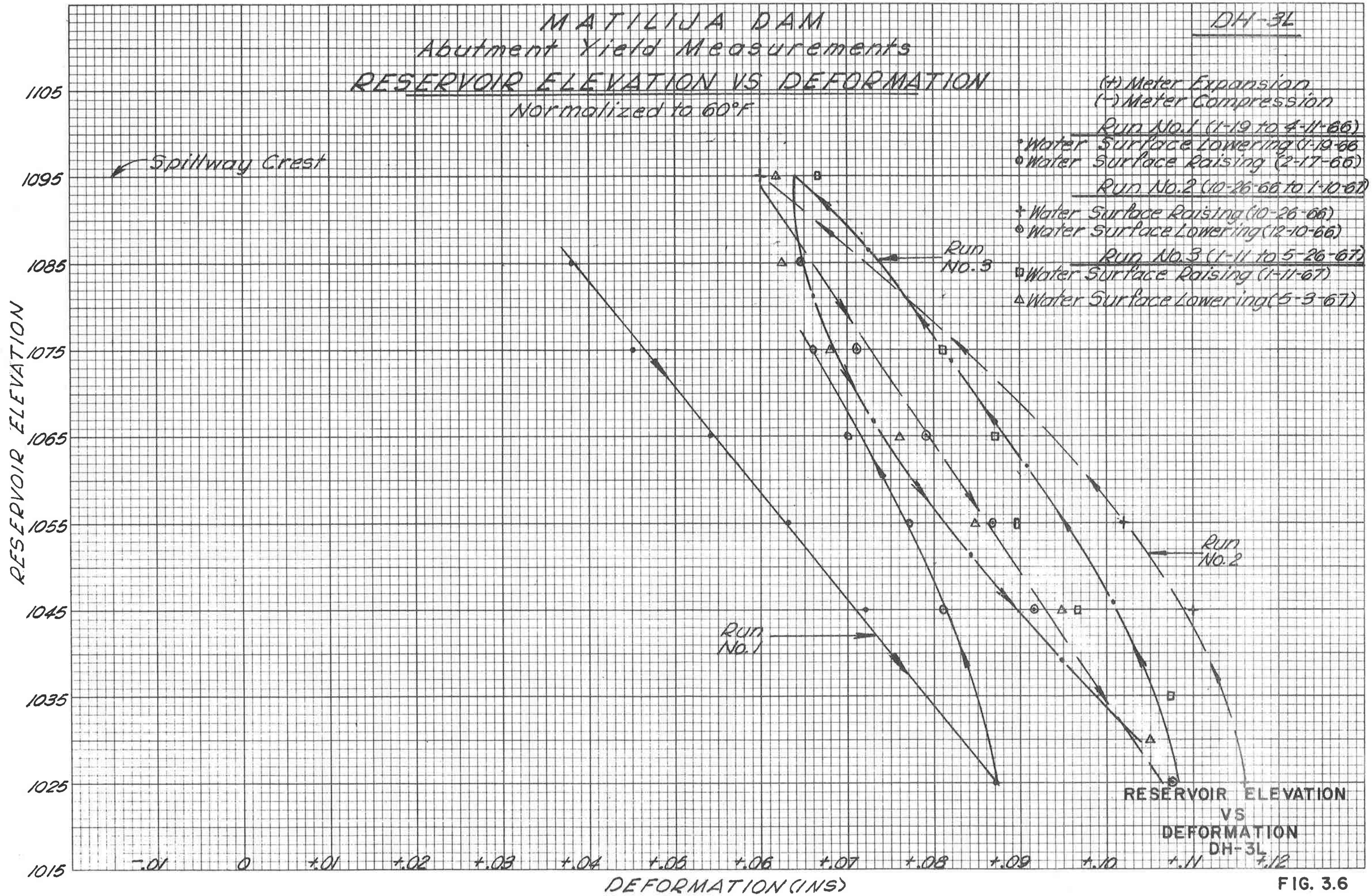
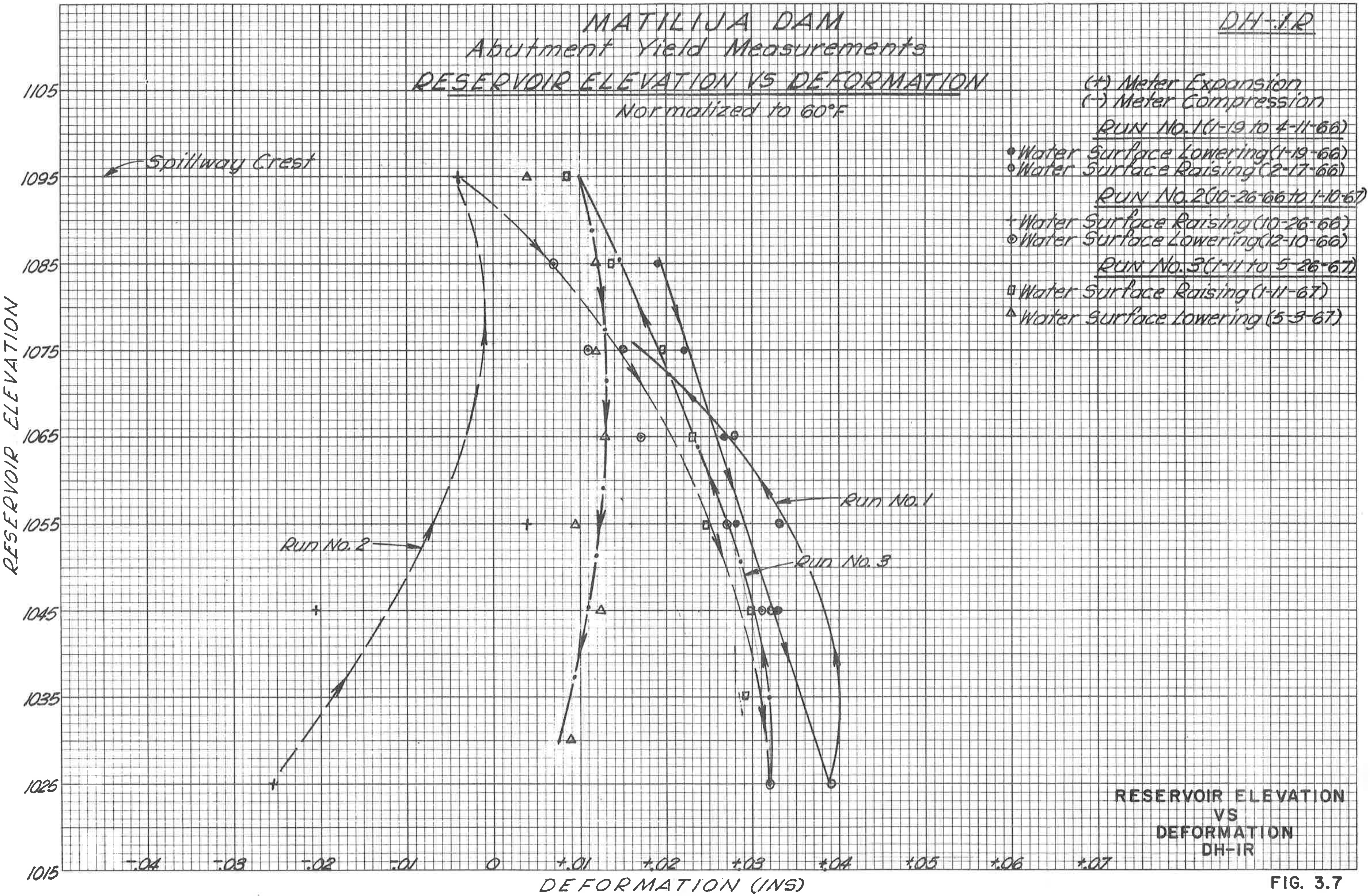

MATILIJA DAM
Abutment Yield Measurements
RESERVOIR ELEVATION VS DEFORMATION
Normalized to 60°F
DH-2aL
(+) Meter Expansion
(−) Meter Compression
Run No. 1 (1-19 to 4-11-66)
Water Surface Lowering (1-19-66)
Water Surface Raising (2-17-66)
Run No. 2 (10-26-66 to 1-10-67)
Water Surface Raising (10-26-66)
Water Surface Lowering (12-10-66)
Run No. 3 (1-11 to 5-26-67)
Water Surface Raising (1-11-67)
Water Surface Lowering (5-3-67)
Spillway Crest
Run No. 1
Run No. 2
Run No. 3
RESERVOIR ELEVATION
1105
1095
1085
1075
1065
1055
1045
1035
1025
1015
DEFORMATION (INS)
+.02
-.01
0
+.01
+.02
+.03
RESERVOIR ELEVATION VS DEFORMATION DH-2aL

FIG. 3.5



RESERVOIR ELEVATION VS DEFORMATION DH-3L

FIG. 3.6



RESERVOIR ELEVATION VS DEFORMATION DH-IR

FIG. 3.7

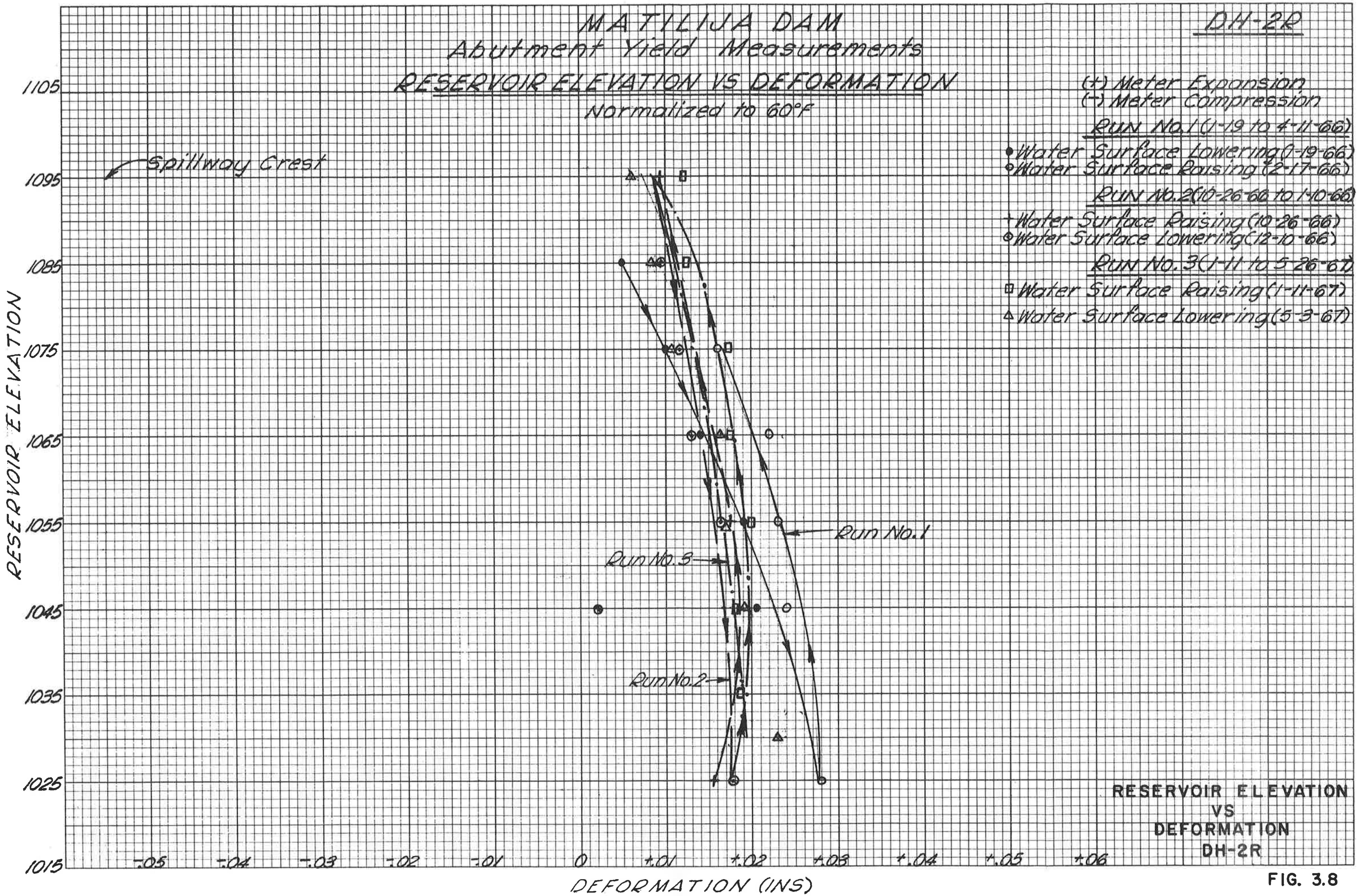
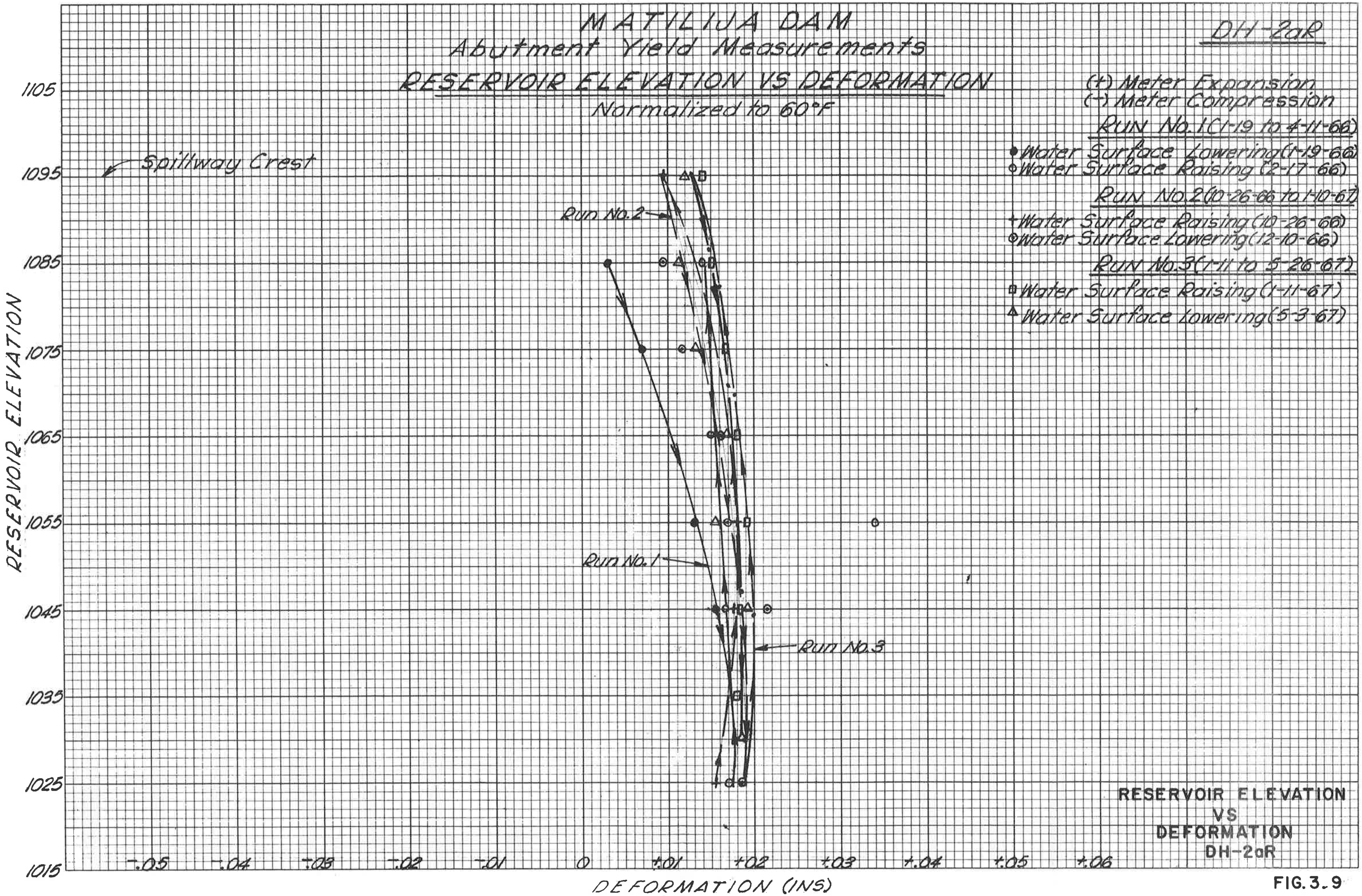

MATILIJA DAM
Abutment Yield Measurements
RESERVOIR ELEVATION VS DEFORMATION
Normalized to 60°F
DH-2R
(+) Meter Expansion
(−) Meter Compression
RUN No.1 (1-19 to 4-11-66)
Water Surface Lowering (1-19-66)
Water Surface Raising (2-17-66)
RUN No.2 (10-26-66 to 1-10-66)
Water Surface Raising (10-26-66)
Water Surface Lowering (12-10-66)
RUN No.3 (1-11 to 5-26-67)
Water Surface Raising (1-11-67)
Water Surface Lowering (5-3-67)
Spillway Crest
Run No.1
Run No.3
Run No.2
RESERVOIR ELEVATION
1105
1095
1085
1075
1065
1055
1045
1035
1025
1015
DEFORMATION (INS)
-.05
-.04
-.03
-.02
-.01
0
+.01
+.02
+.03
+.04
+.05
+.06
RESERVOIR ELEVATION VS DEFORMATION DH-2R

FIG. 3.8



MATILIJA DAM
Abutment Yield Measurements
RESERVOIR ELEVATION VS DEFORMATION
Normalized to 60°F
DH-2aR
(+) Meter Expansion
(-) Meter Compression
RUN No.1 (1-19 to 4-11-66)
Water Surface Lowering (1-19-66)
Water Surface Raising (2-17-66)
RUN No.2 (10-26-66 to 1-10-67)
Water Surface Raising (10-26-66)
Water Surface Lowering (12-10-66)
RUN No.3 (1-11 to 5-26-67)
Water Surface Raising (1-11-67)
Water Surface Lowering (5-3-67)
Spillway Crest
Run No.2
Run No.1
Run No.3
RESERVOIR ELEVATION
1105
1095
1085
1075
1065
1055
1045
1035
1025
1015
-.05
-.04
-.03
-.02
-.01
0
+.01
+.02
+.03
+.04
+.05
+.06
DEFORMATION (INS)
RESERVOIR ELEVATION VS DEFORMATION DH-2aR

FIG. 3.9

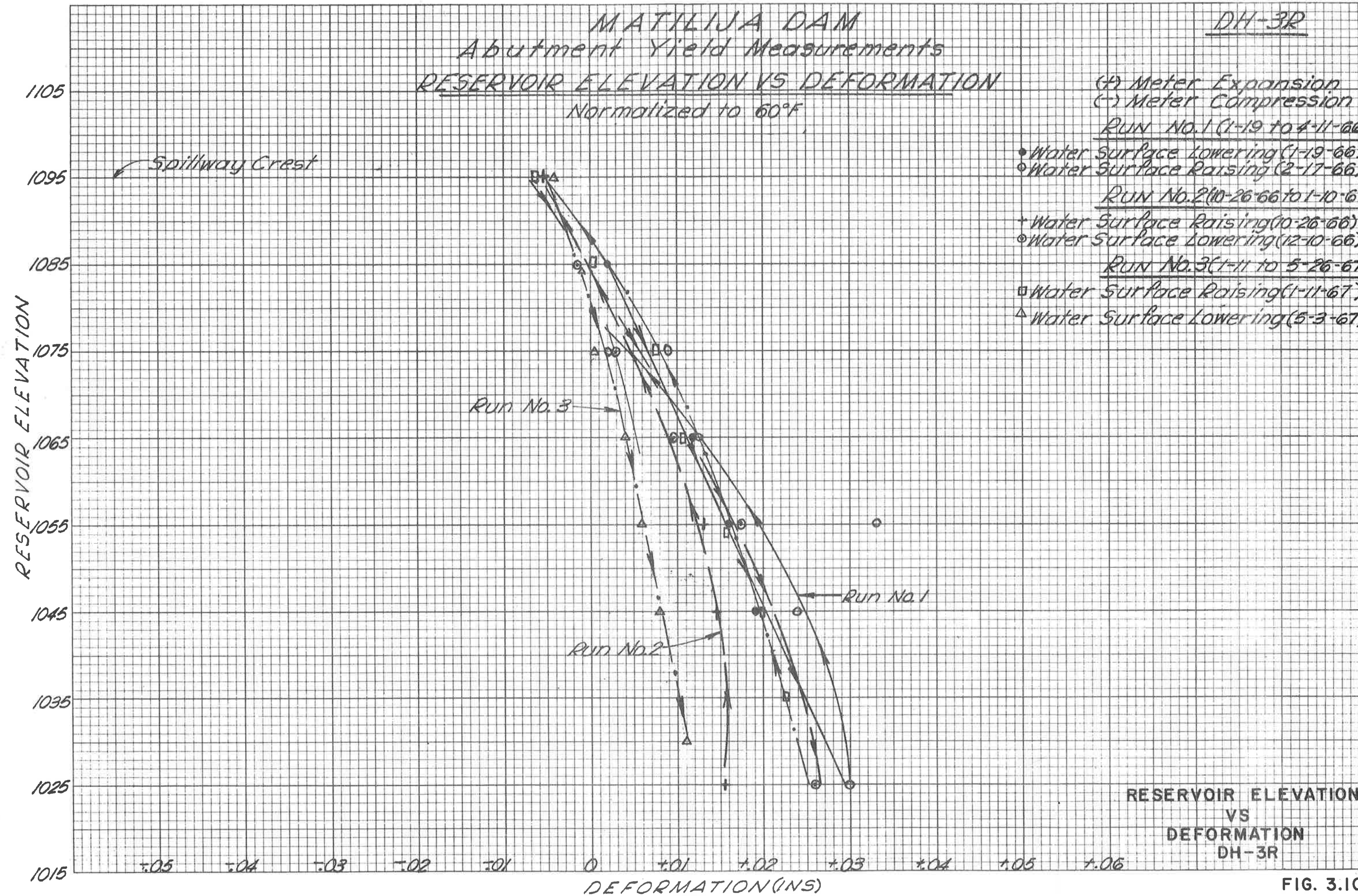
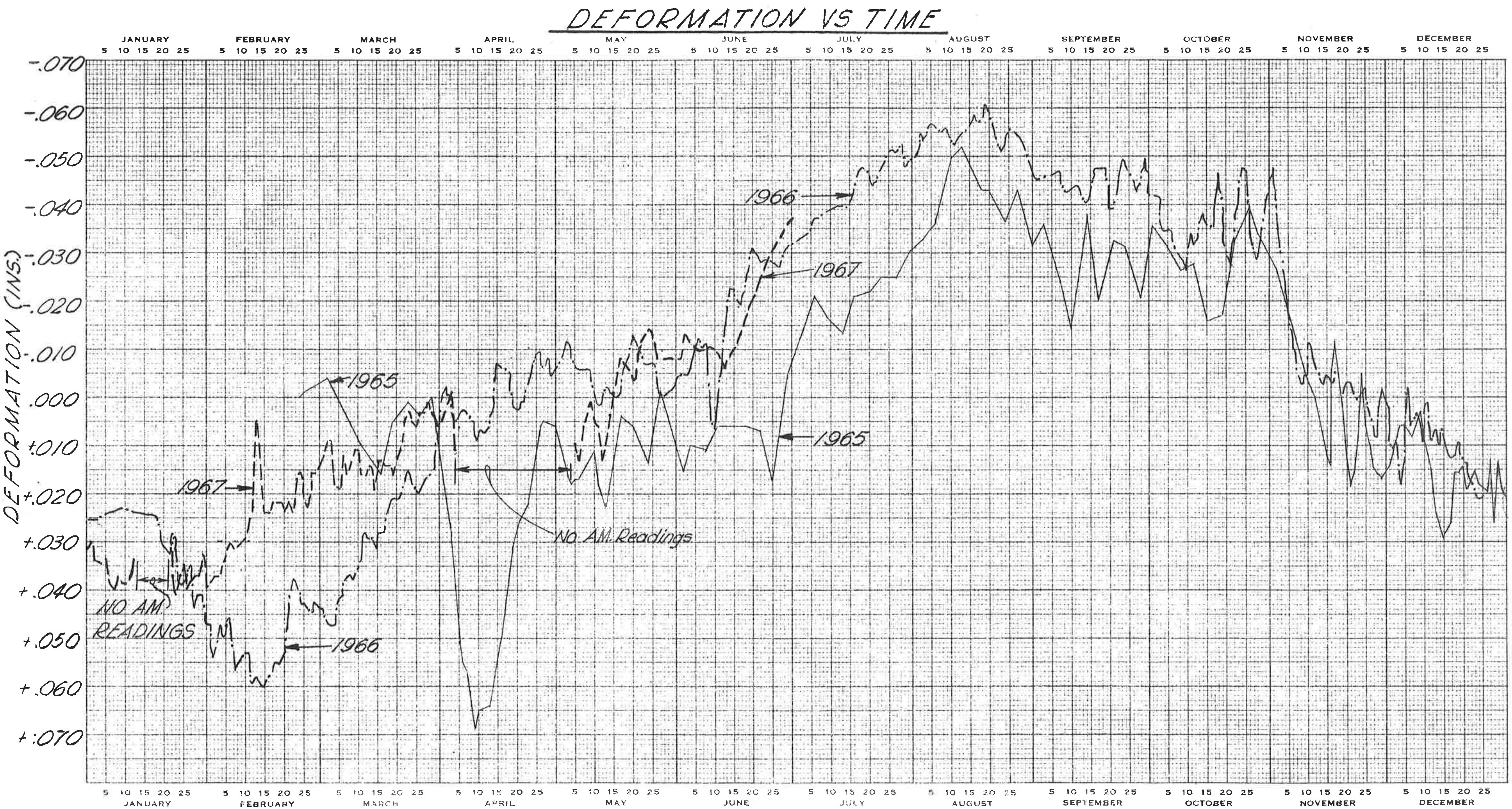

MATILIJA DAM
Abutment Yield Measurements
RESERVOIR ELEVATION VS DEFORMATION
Normalized to 60°F
DH-3R
(+) Meter Expansion
(-) Meter Compression
RUN No. 1 (1-19 to 4-11-66)
● Water Surface Lowering (1-19-66)
⊙ Water Surface Raising (2-17-66)
RUN No. 2 (10-26-66 to 1-10-67)
+ Water Surface Raising (10-26-66)
⊙ Water Surface Lowering (12-10-66)
RUN No. 3 (1-11 to 5-26-67)
□ Water Surface Raising (1-11-67)
△ Water Surface Lowering (5-3-67)
Spillway Crest
Run No. 3
Run No. 2
Run No. 1
RESERVOIR ELEVATION
1105
1095
1085
1075
1065
1055
1045
1035
1025
1015
-.05
-.04
-.03
-.02
-.01
0
+.01
+.02
+.03
+.04
+.05
+.06
DEFORMATION (INS)
RESERVOIR ELEVATION
VS
DEFORMATION
DH-3R

FIG. 3.10

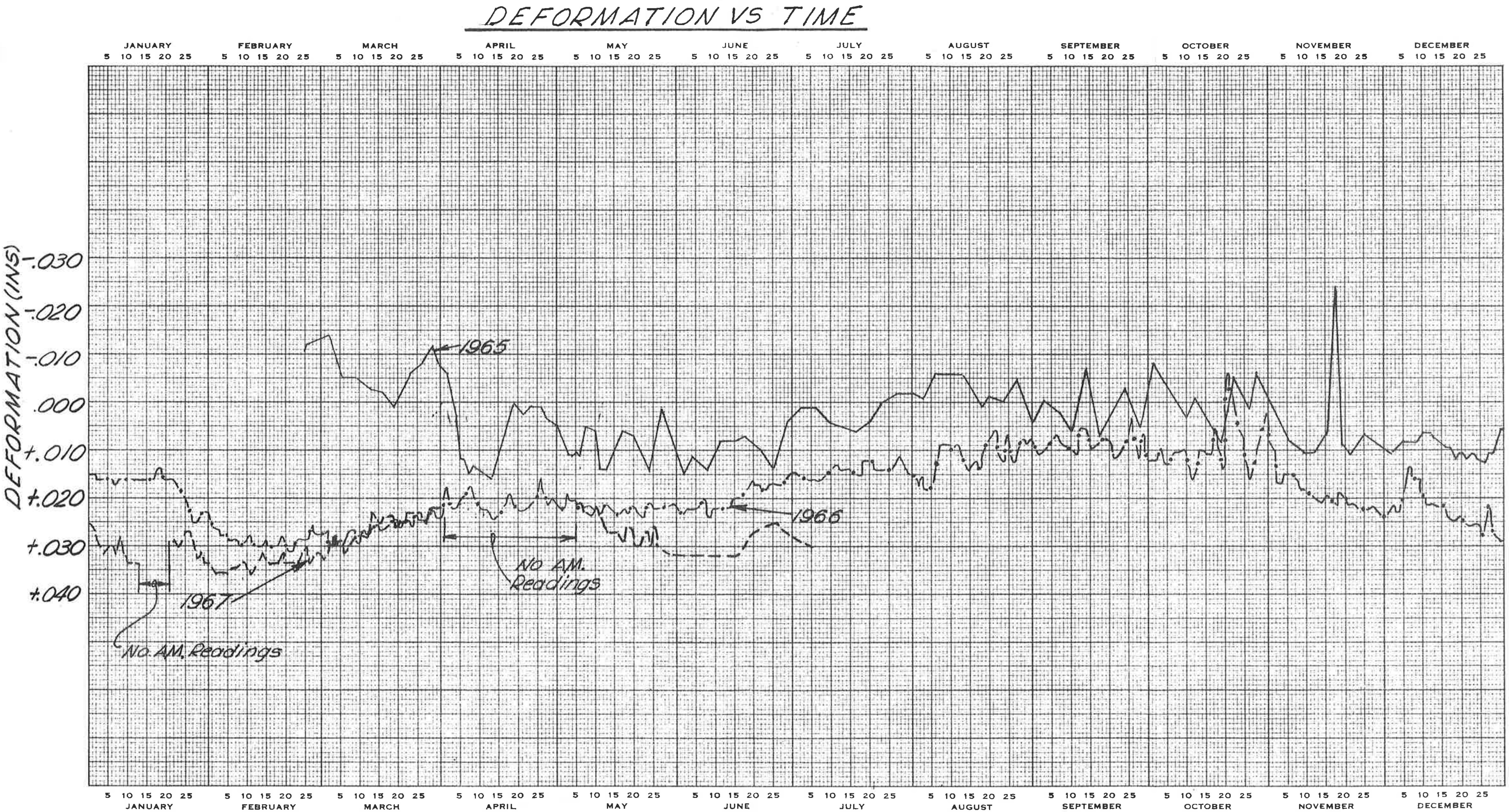
MATILIJA DAM
Abutment Yield Measurements



DH-IL
(Morning Readings)
+ Expansion
− Compression

DEFORMATION
VS
TIME
DH-IL
FIG. 3.11

MATILIJA DAM
Abutment Yield Measurements



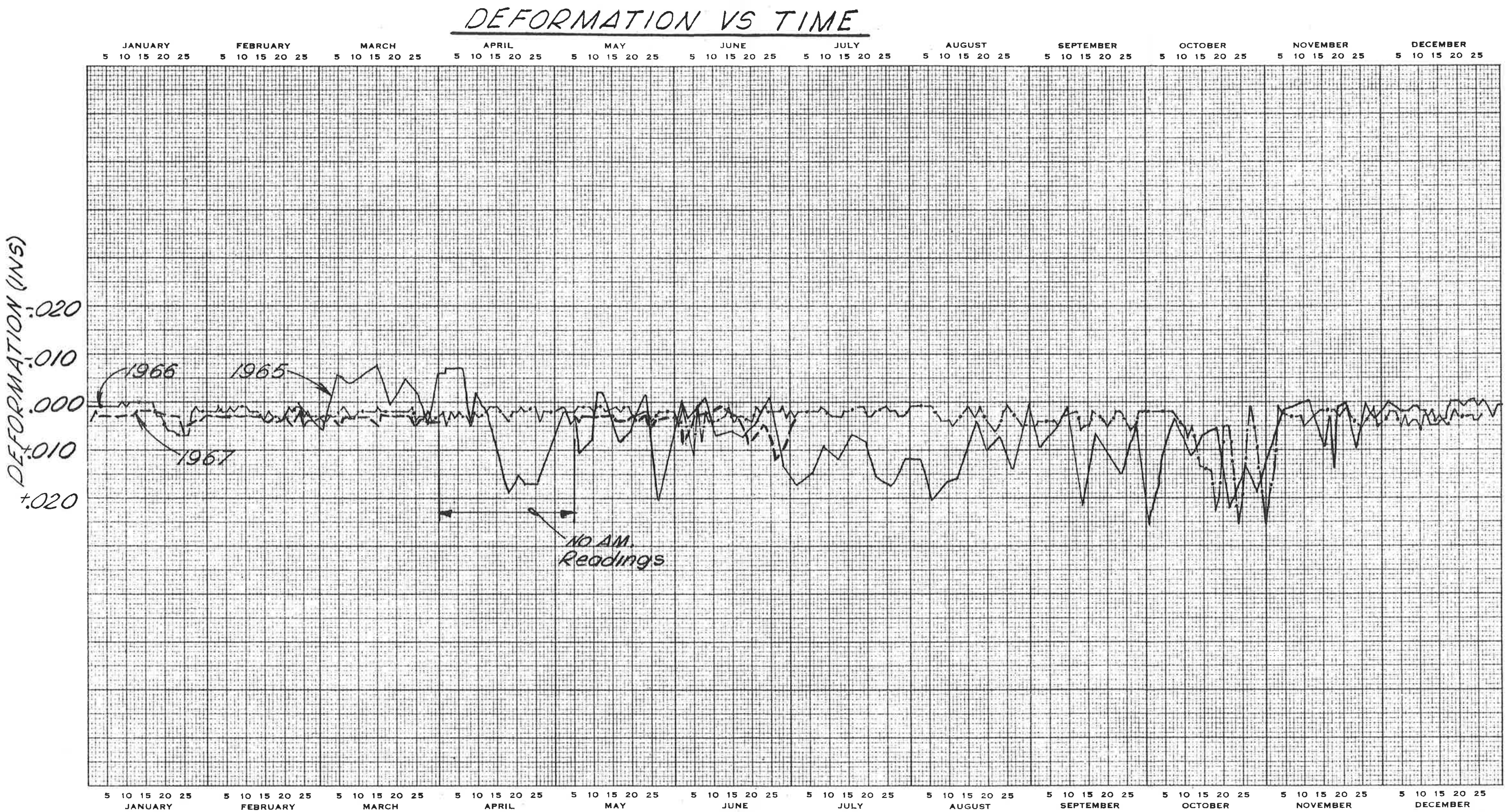
DH-2L

(Morning Readings)
+ Expansion
- Compression

DEFORMATION VS TIME DH-2L

FIG. 3.12

MATILIJA DAM
Abutment Yield Measurements

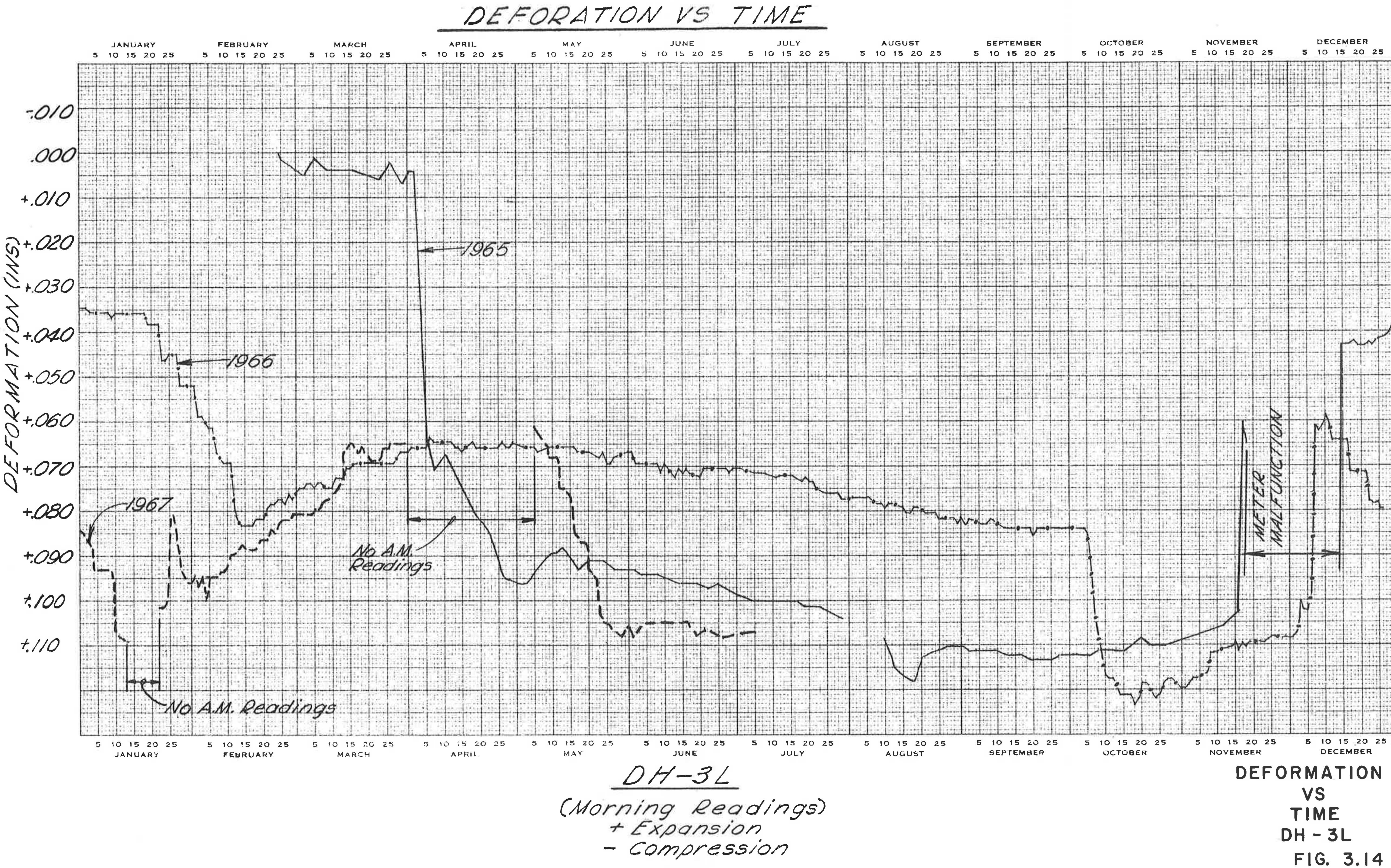


DH-2aL
(Morning Readings)
+Expansion
-Compression

DEFORMATION VS TIME DH-2aL

FIG. 3.13

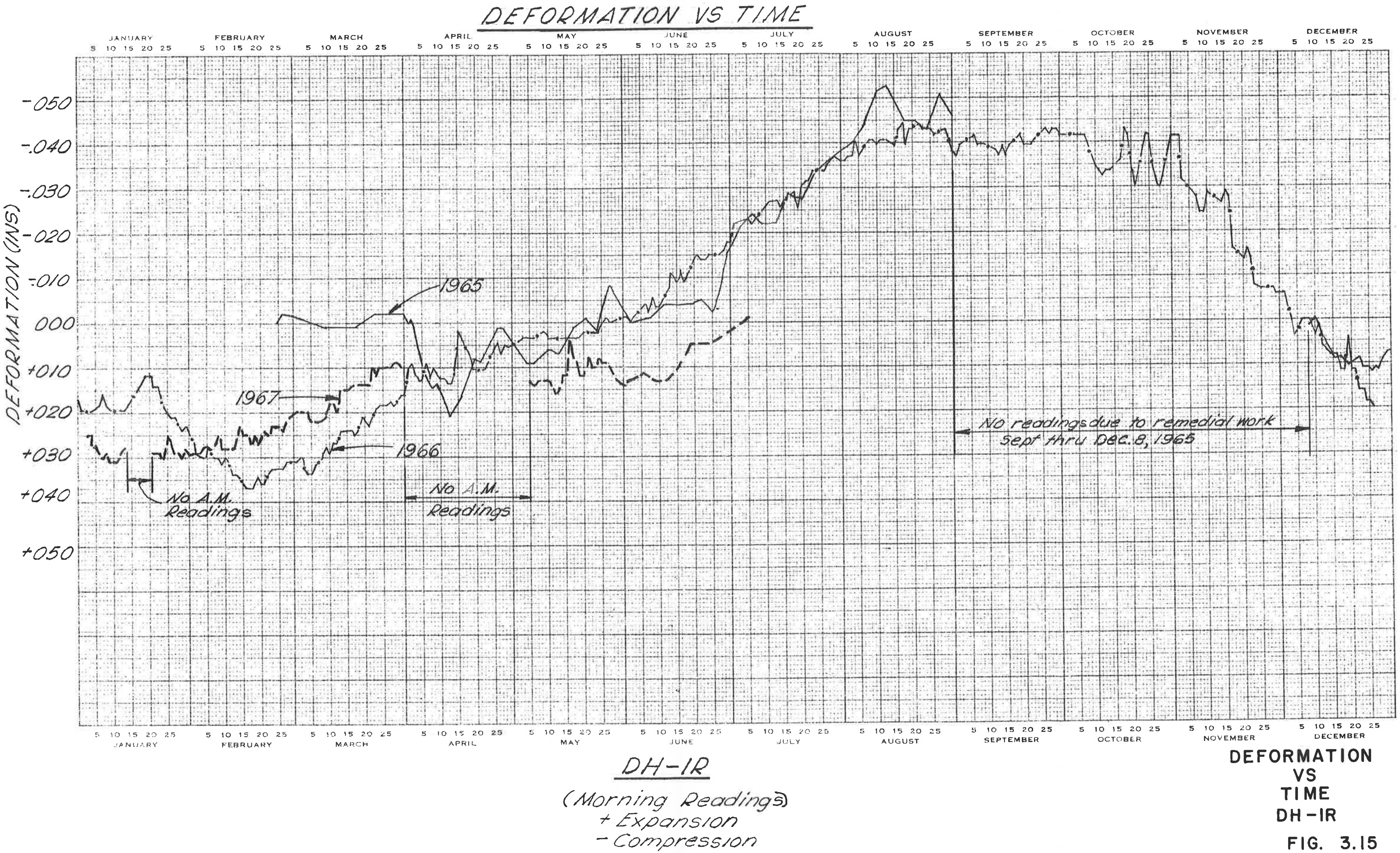
MATILIJA DAM
Abutment Yield Measurements



DEFORMATION VS TIME DH - 3L

FIG. 3.14

MATILIJA DAM
Abutment Yield Measurements



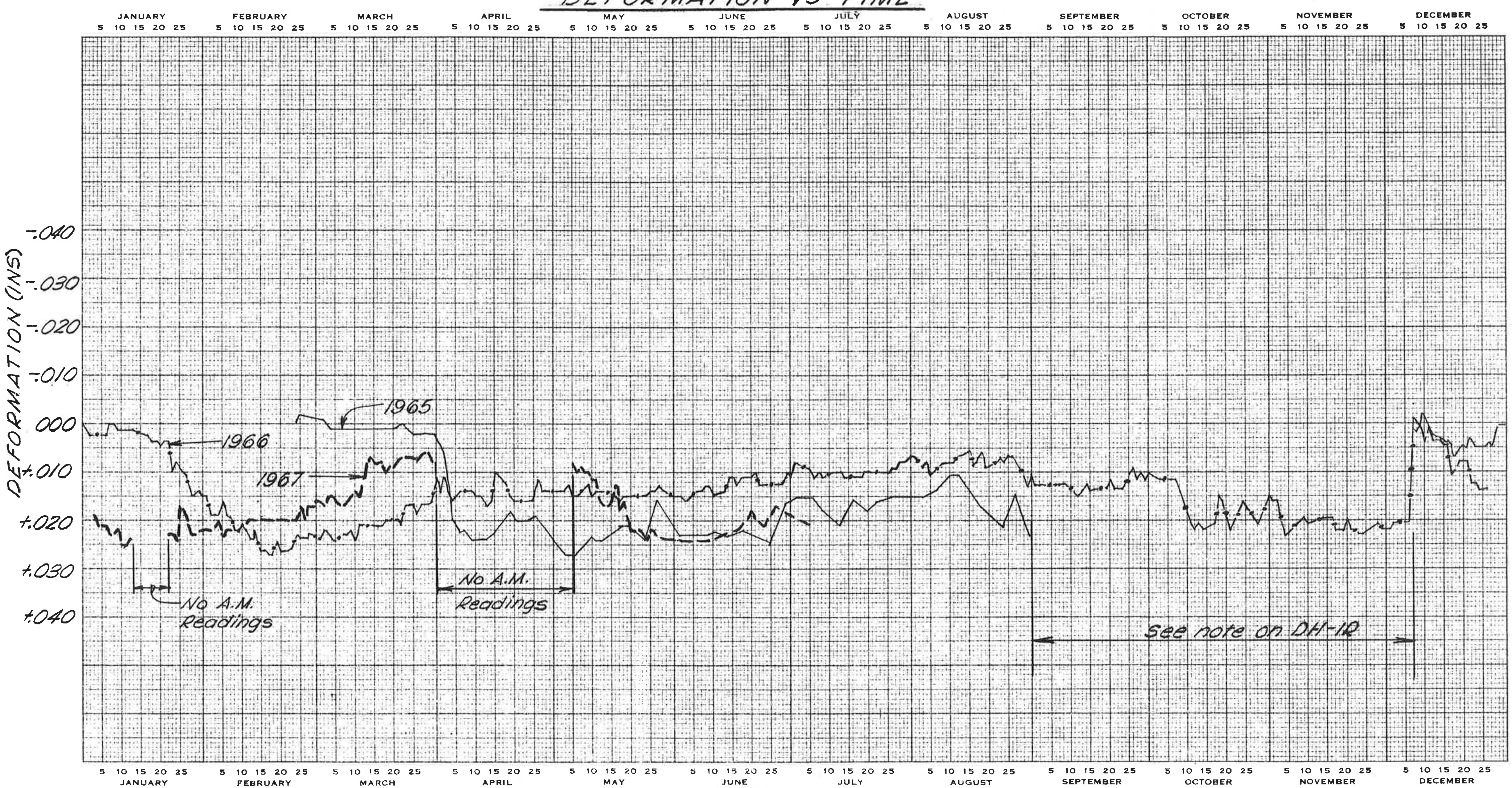
DEFORMATION VS TIME DH-IR

FIG. 3.15

MATILIJA DAM

Abutment Yield Measurements

DEFORMATION VS TIME



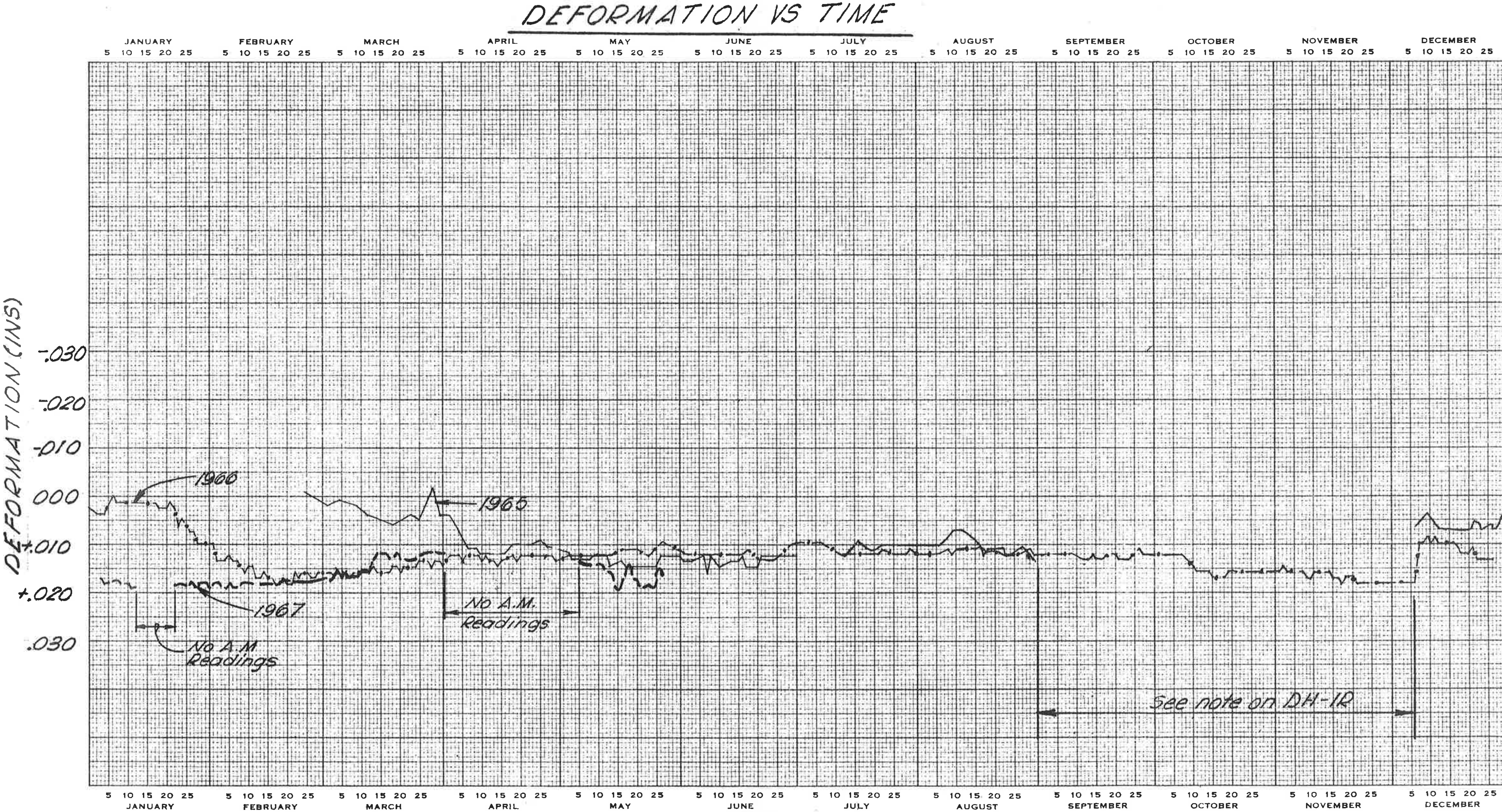
DH-2R

(Morning Readings)
+ Expansion
- Compression

DEFORMATION VS TIME DH-2R

FIG. 3.16

MATILIJA DAM
Abutment Yield Measurements



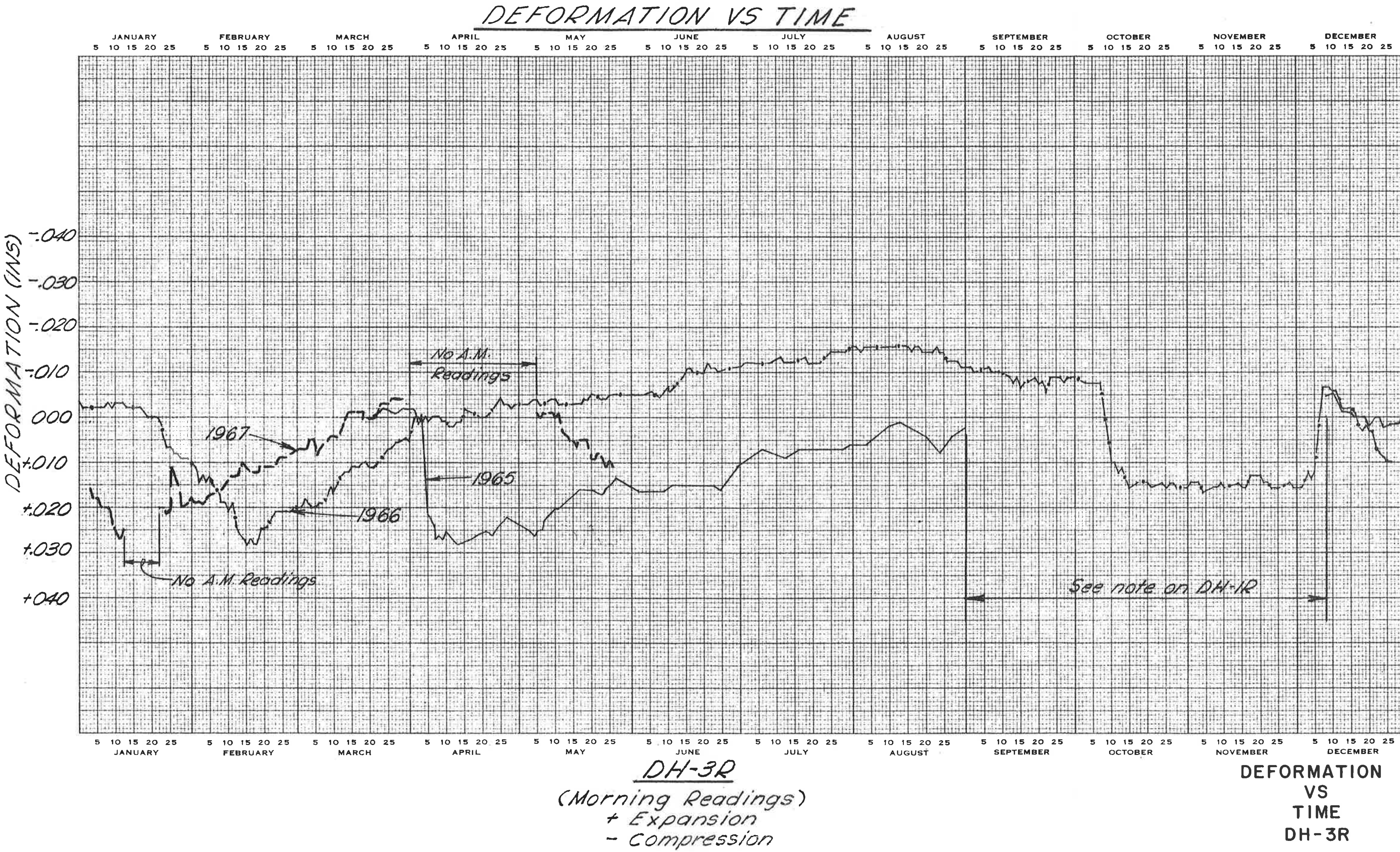
DH-2aR

(Morning Readings)
+ Expansion
- Compression

DEFORMATION VS TIME DH-2aR

FIG. 3.17

MATILIJA DAM
Abutment Yield Measurements

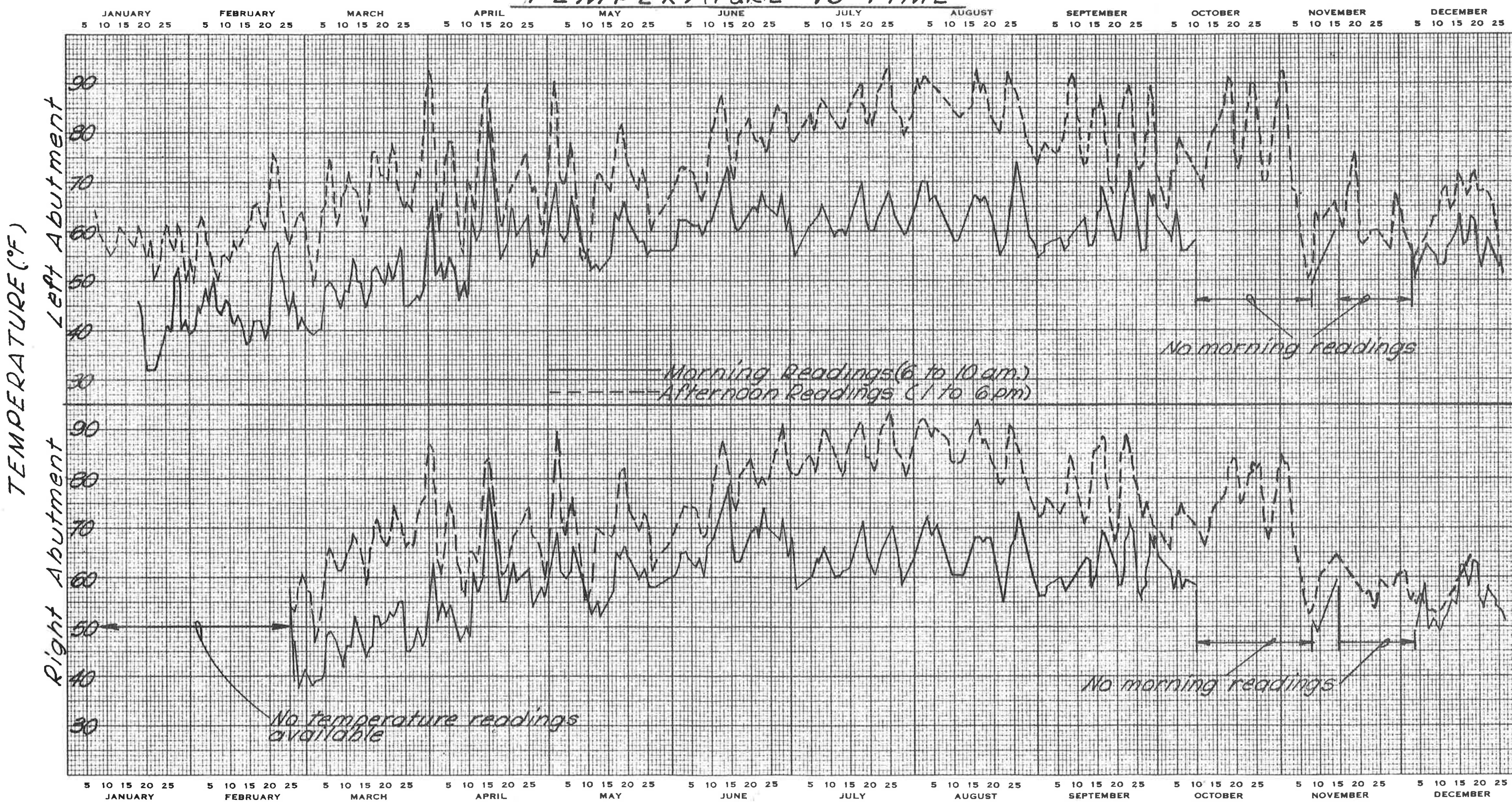


DEFORMATION
VS
TIME
DH-3R

FIG. 3.18

MATILIJA DAM
Abutment Yield Measurements

1966
TEMPERATURE VS TIME



TEMPERATURE
VS
TIME
(1966)

FIG. 3.19

4. MEASUREMENT OF DAM MOVEMENT

4.1 Description of the Measurement System.

At the time of completion of Matilija Dam in 1948 a comprehensive system of survey markers was built into it, and a control network of survey stations and targets was established surrounding the dam.

A total of 15 markers (referred to also as "plates"), were installed initially along the length of the crest of the dam and on the upstream and downstream faces. Markers on the dam are brass or enameled plates, suitably incised, and embedded in the concrete. Survey stations are brass and lead plugs, set into rock. Movement of a particular marker is determined by measuring the perpendicular distance of the marker from a fixed reference line between a station and a target located on the abutments. Complete triangulation surveys of the system are not made.

The points were first surveyed in 1948. They were re-surveyed at least monthly from 1948 to 1950, and once every three months from 1950 to present.

In 1965 the dam was modified by removing the upper lifts of reactive concrete, as described in Section 1. At that time markers near the center of the crest of the dam were removed; new markers have since been installed to replace them.

Only 6 of the original 15 points have been surveyed continuously to the present time (or until they were destroyed in 1965), and hence give a continuous record of the behavior of the dam since 1948. The locations of these points are shown on Drawing 5716-67-3. Plots of the movements of the points are shown on Drawings 5716-67-4, 5 and 6. On the other 9 markers, which are mostly on the upstream and downstream faces, there have been few, sometimes no readings, and they contribute little information on the behavioral pattern of the dam.

4.2 Data and Analyses.

4.2.1 The Left Abutment. Movement at the left abutment as recorded by the measurements is shown on Drawing 5716-67-4. Movement tangential to the curve of the arch is measured at Plates 1 and 5 and movement in a radial direction at Plate 5 only. Plate 5 is located in the deck of the walkway immediately adjacent to the thrust block, but Plate 1 is embedded on the top of the concrete wall of the walkway, and could therefore show movements of the wall which are not indicative of the movements of the dam as a whole.

The plots show that during the first complete filling of the reservoir in January 1952 there was compression of the abutment rock, with the main component of movement about 0.75 inches tangentially to the curve of the dam, into the left abutment, and a component of about 0.25 inch radially downstream. This movement would have been partly elastic deformation, but partly a permanent inelastic deformation which is generally to be expected during the first filling of a reservoir behind an arch dam, and is the result of the water loading closing any systems of cracks and fissures within the rock. The order of magnitude of the movement shown here is in the range shown by other arch dams of similar size. In 1967 the reservoir level was completely drawn down, but as there was no noticeable recovery of the earlier deformation it is concluded that the movement in 1952 was almost completely inelastic. Since 1952 there have been movements measured which reflect the yearly changes in temperature. However, readings were not taken frequently or accurately enough to show the complete annual cyclic movement.

The tangential movements as recorded at Plate 1 and Plate 5 should be equal. Until 1963, within the limits of accuracy of the measurements, this was so. Since 1963, however, the plots of the movements of the points have separated by about 0.50 inch, probably due to the renewed cracking caused by the alkali-aggregate reaction in the concrete.

Since the initial inelastic deformation of the rock during the first filling there has been no sudden movement of the abutment which might indicate local slippage or yield of the abutment rock.

4.2.2 The Right Abutment. Movement at the right abutment as recorded by the measurements is shown on Drawing 5716-67-5. Movement tangential to the curve of the arch is measured at Plate 3 and movement in a radial direction at Plate 7. Plate 7 is located in the deck of the walkway, which is the top lift of the dam itself at the abutments; Plate 3 is in the top of the walkway wall and could therefore give readings which are not indicative of the movements of the dam itself.

The plots show that during the first complete filling of the reservoir, January 1952, there was movement tangentially into the right abutment of about 0.50 inch. There was no component of deformation in the radial direction. Again, as at the left abutment, this movement was partly elastic and partly a permanent inelastic movement, and of normal magnitude. In 1967 when the reservoir was drawn down there was no noticeable recovery, showing that the movement in 1952 was almost wholly inelastic.

Since the initial deformation of the rock there have been movements recorded which reflect the temperature changes throughout the year, but no sudden movement which might indicate a local slippage or yield of the right abutment.

4.2.3 The Crown of the Arch. Movement at the crown of the arch as recorded by the measurements is shown on Drawing 5716-67-6. Movement in a radial direction, upstream and downstream, was measured at Plates 2 and 6, until they were destroyed when the dam was modified in 1965. Plate 2 was embedded in the walkway wall and Plate 6 in the floor slab of the walkway. Since the walkway at the center of the dam was a bridge on piers spanning over the spillway, a component part of the movement recorded may have occurred only in the bridge and not in the dam itself. Since both plates showed radial movement at the same location, it might be expected that they would give equal readings. Differences between them could be due partly to a rotation of the walkway wall relative to the floor slab and partly reflect the limit of accuracy of the method of measurement.

The plots show that during the first complete filling of the reservoir in January 1952 there was a movement downstream at the crest of about 1.20 inches. This would be a total movement, resulting from the elastic movement of the arch itself, about 0.6 inch, plus the elastic and inelastic movements at the abutments.

From the first filling in 1952, until 1958 the crown showed cyclic temperature movements with an amplitude of ± 0.50 inch, but with a constant mean position. However, in 1959 it began to exhibit steady net movement upstream which was continuing at an accelerating rate when the markers were destroyed by constructing the new spillway in 1965. By that date there had been a net movement upstream of 2.5 inches in 7 years, but 50% of this had occurred in the last two years of that period.

New markers have been installed at the new spillway crest, 30 feet below the old crest, but to date insufficient readings have been taken to establish whether the dam is continuing to move upstream, and if so, at what rate.

Three other markers, Plates 13, 14 and 15, were installed at different elevations on the upstream face of the dam at the crown of the arch when the dam was constructed; however, regular readings were not taken on any of these markers. Now all three have been destroyed. (Plates 13 and 14 were removed during the modifications in 1965 and Plate 15 has rusted out.) There are now no points remaining which would show what the total net movement has been at the crown of the arch between 1965 and 1967.

4.2.4 The Base of the Dam. Movement of the arch immediately above the slip-joint, Elevation 960, relative to the concrete block below the slip-joint, is measured at Plates 9, 10 and 11. Plate 10 is at the crown of the arch and Plates 9 and 11 are at the quarter points towards the right and left abutments. The locations of these plates are shown on Drawing 5716-67-3.

Movements at all of these plates are measured by suspending a plumb-line from a fixed pin embedded in the concrete above the slip-joint, then measuring the position of the plumb-bob relative to two inscribed lines on a fixed plate embedded in the foundation block.

All three plates are usually inaccessible due to a pool of water at the downstream toe of the dam, and they have only been read at infrequent intervals. Movements recorded at these plates are tabulated on Table 4.1.

A year after the first filling of the reservoir in January, 1952, the dam was found to have moved downstream and out toward both abutments. These movements were partly permanent set and partly elastic deformation. Plate 10 at the crown showed maximum downstream movement of 1.0 inch. Plates 9 and 11 showed components of movement towards the right and left abutments, with 0.5 inch more movement towards the left. This is compatible with the known greater weakness of the left as compared to the right abutment.

In the 12 years following the first filling there was a continuing movement downstream, totalling about 0.25 inch.

Readings taken a year after the notch was cut showed that the dam had recovered a little due to the reduced water load, and had moved upstream about 0.2 inch.

4.3 Conclusions on the Dam Movement.

The movement at the crest of the dam as recorded by the surveys shows the following characteristics:

TABLE 4.1

MOVEMENTS OF DAM AT SLIP-JOINT

All Movements in Inches

Date	Water Surface Elevation	PLATE 9		PLATE 10		PLATE 11	
		Movement Downstream	Movement to Left Abutment	Movement Downstream	Movement to Right Abutment	Movement Downstream	Movement to Right Abutmnet
Initial Construction	1948	0.0	0.0	0.0	0.0	0.0	0.0
First Filling of Reservoir, January 1952							
16 December 1952	1120.12	0.59	0.22	1.05	0.02	0.73	0.73
18 October 1960	1093.38	0.70	0.30	1.20	0.05	0.90	0.85
28 October 1964	1066.98	0.80	0.30	1.30	0.10	1.20	0.85
New Spillway Cut, August - November 1965							
28 September 1966	1054.65	0.72	0.30	1.11	0.06	0.84	0.78

4.3.1 At both abutments, there were permanent inelastic deformations of the rock during the first complete filling of the reservoir, caused by the closure of any systems of fissures or cracks in the rock. These were about 0.75 inch at the left abutment and 0.50 inch at the right abutment, both values of the order that could be expected for an arch dam of this type. Since that time there have been cyclic movements due to temperature changes.

At neither abutment has there been any sudden movement of a magnitude that would indicate local slippage or yield of the abutment rock.

4.3.2 At the crown of the arch the recorded movement was normal until about the beginning of 1959. At that time the crown began to move upstream against the water load, at an increasing rate, and by 1965 had moved upstream 2.5 inches from its normal equilibrium position (its mean downstream position).

This movement was undoubtedly caused by an overall expansion of the dam as a result of the alkali-aggregate reactivity. It is known that the reaction is still continuing in the concrete in what remains of the upper lifts of the dam, as shown by the increased cracking discussed in Section 5. It is not known what the movement upstream was during 1965-1967, nor is there sufficient survey data presently available to determine if the movement upstream is continuing.

4.3.3 At the slip-joint at the base of the dam there was a total downstream movement of 1.0 inch after the first filling, followed by a further small movement in subsequent years. After the notch was cut reducing the water load, the dam moved upstream 0.2 inch.

All the movements recorded at the slip-joint are normal for a structure of this type.

5. STATE OF THE CONCRETE

5.1 General

In Bechtel's report "Review of Matilija Dam", February 1965, the condition of the dam concrete was investigated by means of compression and density tests on cored samples, petrographic examination of specimens from selected cores, and soniscope testing of the concrete in the dam itself. At that time greater concern was felt about the concrete in upper lifts of the dam, since it was observed to be severely cracked in certain areas. Because of this most of the testing was concentrated on the upper portions of the dam.

Now, however, the top elevation of that part of the dam which functions as an arch has been lowered by 30 feet to Elevation 1095, and the structural integrity of the arch depends on the quality of the concrete below this elevation.

An evaluation of the condition of this concrete has been made, based on data from concrete volume change tests, soniscope testing, strength tests, and visual inspections of the dam. No petrographic examination has been made on any of the concrete remaining in the dam as it is modified.

5.2 Volume Change Tests.

Two concrete samples from the cores drilled in Matilija Dam between October 1964 to January 1965 have been tested in Bechtel's Geology and Rock Mechanics Laboratory for volume change behavior.

Both were taken from Core B, which was drilled vertically into the crest of the dam at Station 1+49, through Lifts 34, 33, 32, 31 and into Lift 30. Sample 1 was taken between depths 4.6 feet to 5.6 feet, i.e. in Lift 33 (El. 1120 to 1125), Sample 2 was taken between depths 10.5 feet and 11.8 feet, i.e. in Lift 32 (El. 1115 to 1120). The location of Core B is shown on Drawing 5716-67-3.

Three pairs of gage plugs each with a gage length of 10 inches were fitted into each specimen. These were spaced equally around the sample so that readings could be averaged to eliminate the effect of warping. All measurements were made with a Whittemore strain gage, accurate to 0.0001 inch in the 10-inch gage length. Installation of the plugs, and the extensometer used, were in accordance with the applicable parts of ASTM.C341 - "Volume Change of Concrete Products".

The cores were stored in sealed containers, standing above, but not in, water, and at a constant temperature of 100°F. The containers, the conditions of storage, and the method of measurement all conform to the relevant parts of ASTM.C227 - "Potential Alkali Reactivity of Cement-Aggregate Combinations".

The lengths of the samples were measured, initially at intervals of 14 days, then increasing to 90 days. They have now been under observation for 800 days. No silica gel can be observed, and earlier patches have reduced to hard, dry deposits. No system of fractures can be seen.

Plots of linear strain with time for both samples are shown on Drawing 5716-67-7. This shows that Sample 1 (Lift 33) exhibited no growth, while Sample 2 (Lift 32) has shown an expansive strain of 1100×10^{-6} millionths in/in, or 0.11%, a growth undoubtedly caused by an alkali-aggregate reaction.

Drawing 5716-67-7 also shows for comparison purposes the results of volume change tests which had previously been made on concrete cores from another concrete arch dam which also contained reactive minerals in the aggregates. This structure, referred to as Dam X, also developed systems of cracks varying from open cracks to micro-cracks and showed movement upstream against the water load.

The drawing shows the result of volume change tests made on samples taken from Dam X at 4 years, 11½ years and 27 years after construction. The first sample at 4 years showed very little growth; but the second sample taken at 11½ years expanded over 0.12% during the test period. The final sample taken at 27 years showed no growth and in fact contracted slightly. These tests, correlated with other observations and measurements on the dam, suggest that the reaction had continued at a high rate for a number of years, then slowed down either because the reactive minerals were all used up, or because an equilibrium condition had been reached with the restraining forces, so that no further expansion occurred.

There are far too many factors involved for it to be suggested that Matilija will follow this same pattern exactly. However, the behavior of Sample 2 in the Matilija tests show that it still has a high potential for growth, and it has definitely not reached a dormant condition.

The lack of movement in Sample 1 suggests that reactive minerals in sufficient quantities to cause disruptive growth were unevenly distributed in the dam. However, it is concluded that where reactive minerals do occur they would still have a potential for expansion, causing movement and cracking. This expansion could occur when moisture is available within the concrete and the conditions of restraint allow the expansion to occur.

5.3 Soniscope Tests

A description of the procedure for assessing the quality of concrete by soniscope testing, including the apparatus employed, its method of use, and the interpretation of results, is given in Bechtel's report "Review of Matilija Dam", February 1965.

Soniscope tests have been made on Matilija Dam on two separate occasions. Firstly, tests were made under the supervision of Bechtel consultant Professor David Pirtz of the University of California, Berkeley, in November and December 1964. A second set of tests was made on the dam by representatives of the Portland Cement Association in June 1965. When the tests were made by the PCA the reservoir level was much lower and readings were taken farther down on the face of the dam. The results of both series of tests are tabulated in Table 5.1, omitting those readings made on parts of the concrete which have since been removed during the dam modification.

These two independent examinations are generally in good agreement, and give conflicting results only for Block M, Lifts 31 and 32. In this case Bechtel took readings in five locations and the PCA in two; also Lift 31 was visibly deteriorated and had a system of wide horizontal cracks, so Bechtel's appraisal of this concrete as being deteriorated and cracked was probably more accurate.

TABLE 5.1

RESULTS OF SONISCOPE TESTING

Test Made By	Station	Section	Lift	Elevation	Pulse Velocity	Remarks
Bechtel	0+15	0	33	1122.5	12,800	Concrete is good in all lifts
			32	1117.5	12,500	
			31	1112.5	12,900	
			30	1107.5	12,900	
PCA	0+20	0	34	1127.5	14,000	Concrete in all these lifts is good to excellent.
			33	1122.5	13,900	
			32	1117.5	14,200	
			31	1112.5	14,000	
			30	1107.5	13,800	
			29	1102.5	14,400	
			28	1097.5	15,200	
			27	1092.5	15,100	
Bechtel	1+10	M	33	1122.5	12,000	Concrete questionable
				1122.5	11,800	
			32	1117.5	-	Concrete deteriorated & cracked
				1117.5	-	
				1117.5	-	
			31	1112.5	-	Concrete poor.
				1112.5	11,500	
PCA	1+20	M	34	1129.0	13,700	Concrete in good condition
				1128.0	13,700	
				1127.0	13,200	
				1126.0	12,800	
			33	1124.0	9,300	Concrete poor, and deteriorated at top of lift
				1123.0	10,800	
				1122.0	11,800	
				1121.0	12,800	
			32	1117.5	13,600	Concrete in all these lifts is good to excellent
			31	1112.5	13,400	
			30	1107.5	13,800	
			29	1102.5	13,900	
			28	1097.5	14,000	
			27	1092.5	14,200	
			26	1087.5	14,100	
			25	1082.5	14,200	
			24	1077.5	14,200	
			23	1072.5	14,600	
			22	1067.5	14,500	
			21	1062.5	14,200	

TABLE 5.1 (Continued)

Test Made By	Station	Section	Lift	Elevation	Pulse Velocity	Remarks
Bechtel	1+37	L	31	1114.5	-	Concrete deteriorated and cracked in both lifts
				1114.5	-	
			30	1109.5	-	
				1109.5	-	
Bechtel	1+65	L	32	1117.5	12,000	Concrete is good
				1117.5	13,100	
			31	1112.5	-	Concrete deteriorated and cracked in all these lifts
				1112.5	-	
			30	1107.5	-	
				1107.5	-	
			29	1102.5	-	
				1102.5	-	
				1102.5	-	
PCA	3+03	H	27	1092.5	13,400	Concrete is good in all these lifts
			25	1082.5	13,100	
			23	1072.5	13,300	
			21	1062.5	13,400	
PCA	4+85	D	33	1122.5	13,200	Concrete is good in all these lifts
			32	1117.5	13,600	
			31	1112.5	13,300	
			30	1107.5	13,200	
			29	1102.5	13,400	
			27	1092.5	12,600	

All the readings taken below Elevation 1095, the elevation of the new spillway crest, gave velocities which indicated the concrete was good or excellent, and no symptoms of cracking or deterioration could be detected by this method of testing.

In the concrete remaining above Elevation 1095, the tests show there are areas of badly cracked and deteriorated concrete, and other areas where the alkali-aggregate reaction has not caused any detectable damage.

5.4 Concrete Core Tests

Concrete cores were drilled at Matilija Dam between October 1964 and January 1965, at 6 locations, A to F. The results of the laboratory tests on those cores are given in Bechtel's report, "Review of Matilija Dam", February 1965.

Cores A to E were taken in the spillway crest, and Core F near the base of the dam, at Elevation 979.0, Station 5+45 in the downstream face. As the cores were drilled they were placed in water, and within hours they showed exudations of silica gel, a product of the alkali-aggregate reaction within the concrete. Core F from the base of the dam showed this effect to the same degree as the other cores. Compressive strength tests on samples from Core F gave results of 3,500 psi and 2,600 psi.

5.5 Visual Inspections of the Dam

Field inspections of the dam have been made by Bechtel engineers periodically during the past two years to observe the condition of cracking in the concrete.

As anticipated, the concrete in the upper lifts which was cracked before 1965 has continued to deteriorate. This is most noticeable at the left abutment where cracks in the steps and the control house have continued to expand.

The cracking visible on the face of the dam is shown on Drawing 5716-67-3. The worst crack extends almost horizontally through Lift 32, Block M, into Lift 31, Block L, ending at the base of Lift 31. However, a comparison with records made in 1965 shows that no detectable increase in the extent of this crack has occurred in the interim.

Some new cracks have opened up adjacent to the notch. The new spillway is now accessible by ladder and the pattern of cracking on the left face of the notch has been examined. It is worse in Lifts 29 and 31, and if certain of the cracks enlarge they could cause some concrete to spall off above Elevation 1095.

On the right face of the notch the only crack visible is the opening of the horizontal construction joint at Elevation 1125.

A recent thorough inspection of the dam was made along the crest of the spillway and the upstream and downstream sides of the dam and no cracks were observed below Elevation 1095. The reservoir at the time of this recent inspection was at Elevation 1045±.

5.6 Conclusions on the State of the Concrete. Table 5.2 summarizes the data available on concrete in Block L.

TABLE 5.2

SUMMARY OF CONCRETE TESTS ON BLOCK L

Lift	Volume Change Tests	Soniscope Tests	Strength Tests	Visual Examination
33	No expansion	No test	No test	No cracks
32	Vigorous expansion	Good concrete	5000 psi	No cracks
31	No test	Deteriorated and cracked concrete	2500 psi	Badly cracked

It is concluded from the visual examination of the dam:

a. At present there is no visible cracking or deterioration in the concrete below Elevation 1095.

b. There is continued deterioration and cracking in the remaining concrete above Elevation 1095, most noticeably in the left abutment, but this cracking does not at present endanger the dam.

It is concluded from the data available on the concrete in Block L, Lift 32, and the observations of Core F:

c. In parts of the dam which now show no signs of deterioration or cracking there may be reactive minerals present, which could possibly cause an expansion in the future, if the right amount of moisture were available and the conditions of constraint would allow it.

6. STRUCTURAL ANALYSIS

6.1 General

This section describes the structural analysis of the concrete arch dam in its present configuration. Since the expanding concrete in the top lifts has been removed and since the arch dam has a foundation sliding joint which prevents cantilever action in the shell, several arches have been analyzed as independent arches under the effect of water load, temperature, flood load, silt load and earthquake load. An attempt was made to find the critical values for crown deflection and abutment yielding. These values could be used as guidelines for comparison with values obtained from future monitoring of the behavior of the dam.

6.2 Description of the Dam

The dam is shown on Drawing 5716-67-2. The crest of the new spillway is at Elevation 1095 and the overflow section extends from Station 1+75 to Station 4+55. Other features of the dam are the same as described in the two previous Bechtel reports.

6.3 Method of Analysis

The independent rings were analyzed by means of the fundamental method of work taking into account the variable thickness, the elasticity of the foundation rock, the abutment yielding and the expansion of the concrete due to chemical reactivity of the aggregate. Four typical arches were selected for the analysis:

a. The crest ring at Elevation 1095.

b. The ring at Elevation 1080.

c. The ring at Elevation 1030.

d. A low ring at Elevation 980.

The effect of abutment yielding was investigated only for the rings at Elevation 1080 and Elevation 980 because at approximately these elevations there are deformation meters installed in the left abutment.

The effect of volume change due to chemical reactivity was investigated for the crest ring at Elevation 1095 because a survey plate was installed at that elevation and the movement is being monitored.

The wingwalls at each side of the new spillway are cracked and they are therefore considered as being incapable of transmitting arch thrust to the abutment. They were analyzed as retaining walls cantilevered off the arch below Elevation 1095 and resisting the water load by their own weight. The condition of these walls does not affect the integrity of the main dam since they no longer act as part of it.

6.4 Design Assumptions

6.4.1 The following assumptions were made for the structural analysis of the dam:

a. The arches were assumed to have radial abutments.

b. Arch stresses were assumed to have a linear variation between the values at the upstream and downstream faces of the dam.

c. The effects of shear deformation were neglected.

6.4.2 The following values were adopted as a basis for the structural analysis:

a. Concrete.

Unit weight	150 lb./cu.ft.
Modulus of elasticity	2,500,000 psi
Coefficient of thermal expansion	5.6×10^{-6} per $1^{o}F$
Compressive strength	3,000 psi

b. Water Load.

Unit weight	62.5 lb./cu.ft.
Maximum normal reservoir water surface	Elevation 1095
Maximum probable flood water surface	Elevation 1113.7

c. Silt Load.

Silt (future assumed)	Elevation 1069

d. Seismic Load.

Horizontal earthquake acceleration	0.10g

e. Temperature Variation. The temperature distribution in the concrete was computed by analytical methods based on the solution of Fourrier's thermal equation.

f. Uplift Load. Uplift was neglected in the analysis of individual arches.

g. Tailwater Load. Tailwater influence was not included in the analysis. It would have a small beneficial effect on the lowest arch.

h. Elastic Constants. A uniform foundation modulus of 2,500,000 psi was adopted for the purpose of stress analysis of the dam. The actual value may be somewhat lower locally but the main region of low modulus was near the top of the dam abutments and since the new spillway was cut down to Elevation 1095 it has no significant influence on the arch stresses of the lower rings.

i. Volume Expansion. The expansion of the concrete due to chemical reactivity of the aggregates was assumed to be uniform over the whole ring and was analyzed as expansion due to temperature rise.

6.4.3 Load Combinations. The following combinations of loadings were considered:

a. Water load at maximum probable flood level, Elevation 1113.7.

b. Water load at maximum probable flood level, Elevation 1113.7, temperature drop and silt load.

c. Water load at maximum normal reservoir level, Elevation 1095, temperature drop, silt load and seismic load.

6.5 Results and Conclusions.

6.5.1 Results. The results of the analysis have shown that the stresses for all loading conditions are acceptable. The highest stressed ring is the one at Elevation 980 and even here the maximum stress due to flood, temperature and silt loads does not exceed 960 psi in compression and 130 psi in tension. Since it can be assumed that the sliding joint is not fully effective and that some shear is actually transmitted at the joint the actual stresses might be even somewhat lower. The yielding of the abutments would produce a stress of ±54 psi in the arch at Elevation 1080 for 1-inch movement in the direction of thrust and ±130 psi in the arch at Elevation 980 for 0.1 inch movement. A volume change in the crest ring resulting in a radial crown deflection of 2 inches in the upstream direction would cause stresses of only ±200 psi in the ring. The results of the analysis are shown in Table 6.1 at the end of this section.

6.5.2 Conclusions from Structural Analysis. The analysis has indicated that at the present time all stresses due to all loading combinations are within acceptable limits. The structure could also be subjected to yielding of an abutment of up to 2 inches at Elevation 1080 and 0.15 inches at Elevation 980 without exceeding the allowable stresses in this type of structure. The actual recorded yielding is many times smaller than the above values (see Section 3). In monitoring the movement of the crown in the future, an upstream movement of 2½ inches would still be acceptable combined with even the most unfavorable assumptions of an empty reservoir and temperature rise.

TABLE 6.1

SUMMARY OF STRESSES

	ARCH RING AT:															
	EL. 1095				EL. 1080				EL. 1030				EL. 980			
	Abutment		Crown		Abutment		Crown		Abutment		Crown		Abutment		Crown	
TYPE OF LOADING	Extr.	Intr.	Extr.	Intr.	Extr.	Intr.	Extr.	Intr.	Extr.	Intr.	Extr.	Intr.	Extr.	Intr.	Extr.	Intr.
Water to El. 1095	0	0	0	0	+120	+210	+180	+135	+81	+488	+494	+208	-17	+559	+516	+90
Flood Water Load El. 1113.7	+215	+318	+290	+234	+270	+472	+405	+303	+104	+628	+636	+268	-20	+650	+600	+105
Temperature Drop	-52	+51	+28	-30	-45	+71	+25	-24	-101	+91	+56	-68	-100	+92	+60	-93
Silt Load to El. 1069	0	0	0	0	0	0	0	0	+24	+146	+148	+62	-7	+220	+200	+35
Seismic Load	+26	+39	+36	+29	+50	+88	+75	+56	+17	+100	+102	+43	-3	+91	+84	+15
Combination a. Flood & Temperature	+163	+369	+318	+204	+225	+543	+430	+279	+3	+719	+692	+200	-120	+742	+660	+12
Combination b. Flood & Temp. & Silt	-	-	-	-	-	-	-	-	+27	+865	+840	+262	-127	+962	+860	+47
Combination c. Water & Temp. & Silt & Earthquake	-26	+90	+64	-1	+125	+369	+280	+167	+21	+825	+800	+245	-127	+962	+860	+47
1" Yielding of One Abutment in Direction of Thrust	-	-	-	-	-54	+54	+38	-38	-	-	-	-	-1298	+1298	+198	-198
Crown Deflection of 1" Due to Volume Change	+103	-101	-55	+59	-	-	-	-	-	-	-	-	-	-	-	-

7. PERTINENT LETTERS

1. Karl V. Taylor to Mr. A. P. Stokes, Director, Department of Public Works, County of Ventura, dated August 11, 1967.

2. Consulting Board to Karl V. Taylor, dated August 14, 1967.

BECHTEL CORPORATION

ENGINEERS-CONSTRUCTORS

TWO TWENTY BUSH STREET · · · · SAN FRANCISCO, CALIF. 94119

August 11, 1967

Mr. A. P. Stokes, Director
Department of Public Works
County of Ventura
Courthouse
Ventura, California 93001

Subject: Review of Matilija Dam - 1967
Job 5716

Dear Mr. Stokes:

We have completed our study of the data gathered during the past two years and can now draw conclusions and make recommendations regarding the present condition and future safety of Matilija Dam. A report entitled "Review of Matilija Dam - 1967" will be issued at the end of this month. This letter summarizes the conclusions and recommendations resulting from our studies which are discussed in detail in the report. The letter was discussed with our consultants, Messrs. Julian Hinds, Roger Rhoades and Dr. R. W. Carlson, prior to its submittal.

Our studies show that there is no reason to believe that the performance of the dam with respect to safety would be unsatisfactory in the foreseeable future. The meters we installed in both abutments and which have been read for two years while filling and emptying the reservoir in controlled cycles, indicate that the abutment rock, within the length of the meters, is adequately stable. The movements measured by the meters are small and can be accounted for.

Our structural analysis of the dam in its present altered condition with a permanently lowered maximum reservoir level shows that movements much larger than those indicated by the meters could be accommodated before stresses would become unacceptably high.

There is evidence of continued cracking and enlarging of existing cracks in the remaining concrete above Elevation 1095, which is the elevation of the new spillway crest. Volume change tests on specimens of the concrete taken in 1965 above Elevation 1095 have also shown that concrete expansion is continuing at a rapid rate due to autogenous growth. We believe, however, that the continued deterioration of the concrete above Elevation 1095 presents no hazard to the integrity of the dam itself. Recent careful visual examination of the concrete below Elevation 1095 shows no evidence of concrete cracking, expansion or deterioration.

At this time it cannot be determined conclusively whether the dam arch is still moving upstream due to reactivity because the survey plates which were located at the crown were perforce removed when the new spillway was constructed, or have rusted away. New plates have been installed recently but sufficient time has not elapsed to indicate whether any movement is still in progress.

Structural analyses of the concrete walls at the ends of the new spillway which support the new footbridge were performed. Based on the present configuration of cracking, the analyses show that the walls are safe even under maximum spillway discharge conditions with the reservoir at Elevation 1113.7. However, observations should be continued to ascertain that any additional cracking which may occur in the future will not affect the integrity of the walls as bridge abutments. It should be emphasized that the condition of the walls does not affect the integrity of the main dam, since these walls no longer act as part of the arch dam itself.

As you know, any assessment of the condition of the dam at present must be based on the past behavior of the dam. Monitoring of the abutment meters and of the survey plates must be continued. Substantial departures in the future from the established pattern, should they occur, would have to be recorded and analyzed immediately. Also periodic testing of the dam concrete is of extreme importance, so that any deterioration, and development of micro-fracturing or the beginning of accelerated autogenous growth will not go unnoticed.

In summary:

1. We recommend that the dam remain in service and be monitored in accord with good practice.

2. In order to properly monitor the future performance of the dam we further recommend that:

a. The abutment yield measuring devices be read, recorded and plots similar to those shown in the final report prepared. The devices should be read at least every two weeks, together with reservoir elevations and temperatures at the time the readings are taken.

b. Horizontal movements of all the survey plates, including the ones recently installed near the spillway crest, be measured and the results plotted. Such measurements should be performed at least once every three months, and immediately following the occurrence of any unusual events such as an earthquake or a major flood.

c. Not later than five years from now and at intervals of five years or less thereafter, strength, petrographic examination and soniscope tests be performed on the dam concrete. Should measurements performed under a. and b. above indicate any significant movements of the dam, the concrete testing noted herein should be initiated immediately.

d. All records, data and results of tests be plotted, evaluated, and any significant deviations from previous patterns be promptly analyzed.

As requested by Mr. Byron D. Edde, a copy of this letter will be sent to the State Division of Dam Safety.

Very truly yours,

Karl V. Taylor

Karl V. Taylor
Manager of Special
Projects Engineering

KVT:NMN:mg

cc: Julian Hinds
Roy W. Carlson
Roger Rhoades
R. B. Jansen, Supervisor, Safety of Dams
State of California
P. O. Box 388
Sacramento, California
B. D. Edde

BECHTEL CORPORATION

ENGINEERS-CONSTRUCTORS

TWO TWENTY BUSH STREET · · · · SAN FRANCISCO, CALIF. 94119

August 14, 1967

Mr. Karl V. Taylor
Manager, Special Projects Engineering
Bechtel Corporation
P. O. Box 3965
San Francisco, California 94119

Dear Mr. Taylor:

At your request, we assembled in San Francisco to consider the studies and recommendations of the Bechtel Corporation pertaining to the Matilija Dam. We examined a considerable volume of data analyzing the results of tests and studies developed over the past two years. These data are to be assembled into a report by Bechtel and submitted to the Director of Public Works of Ventura County. The conclusions and recommendations from this report are contained in a letter from Bechtel Corporation to Mr. A. P. Stokes dated August 11, 1967.

Among the several subjects studied by us were the condition of the concrete, the movement of the dam and its abutments, and analyses to show computed stresses under a variety of assumptions. All evidence indicated the concrete to be of good quality below the elevation of the new spillway crest, but of poor quality at higher elevations.

From survey plate measurements, abutments were shown to have moved from a half to three quarters of an inch with the first filling of the reservoir in 1952. After the first filling, the movements have been much smaller. Since the lowering of the spillway crest, the movements at the abutments with respect to points approximately 50 feet into the abutments have been measured with sensitive strain meters. These movements have been found to be only a few hundredths of an inch and very nearly repetitive.

Stress analyses of the dam with high water at 19 feet above the new spillway crest showed moderate and safe stresses. When each abutment at Elevation 980 was assumed to move one inch tangentially, a bending stress of 1298 psi tension (extrados) and 1298 psi compression (intrados) was indicated at the abutments. Since the observed movements during the past two years have been only a few hundredths of an inch, the actual stresses are indicated to be conservative.

We have reviewed the August 11 letter from Bechtel to Mr. A. P. Stokes and we concur in the recommendations contained in this letter. With proper monitoring, we believe that the dam can remain in service safely for the foreseeable future.

Very truly yours,

Roy W. Carlson

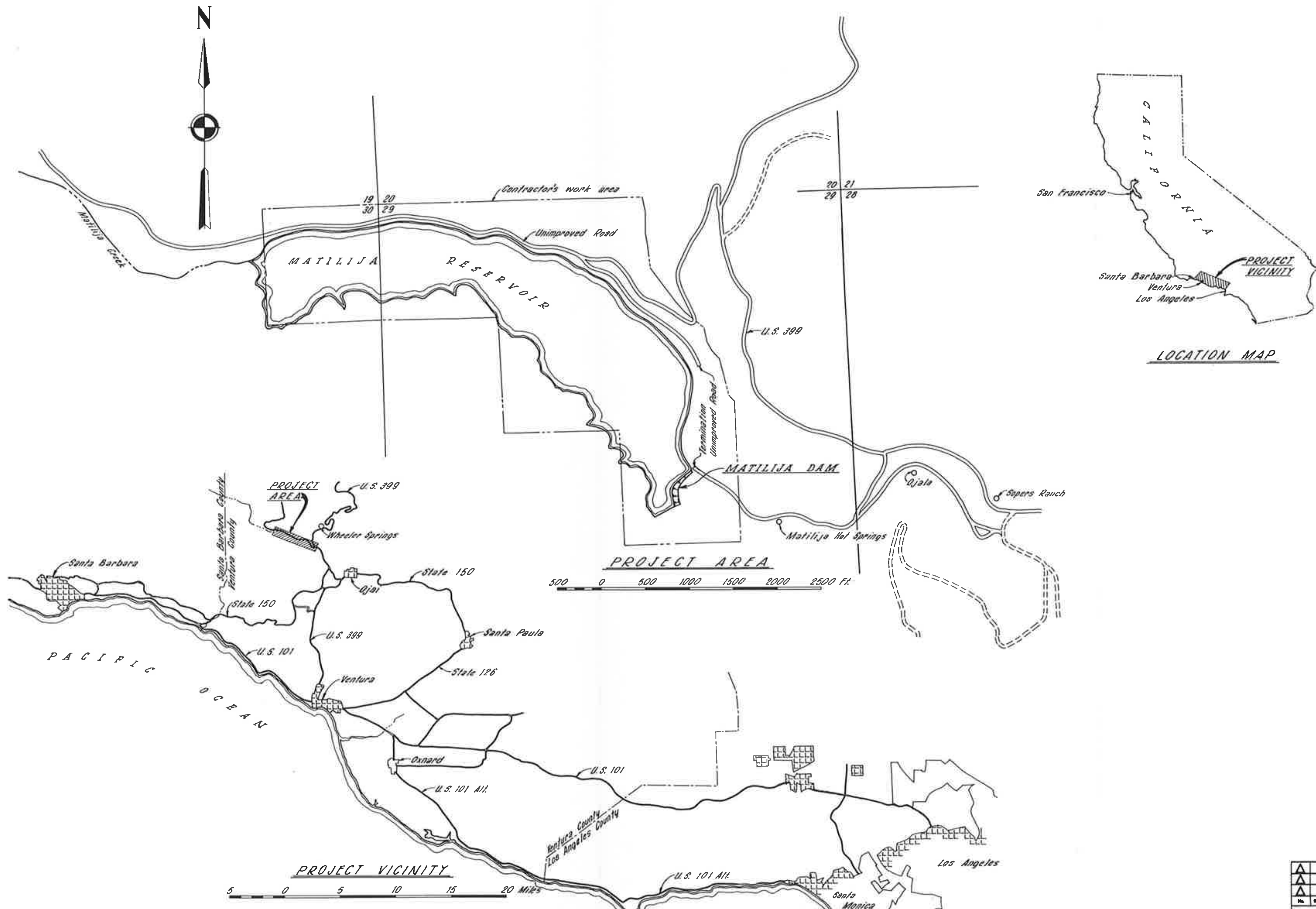
Roy W. Carlson

Julian Hinds

Julian Hinds

Roger Rhoades

Roger Rhoades



No.	DATE	REVISIONS	BY	CH'K	Eng.	Proj. Eng.	MGR.	CLIENT
		Issued for Aug. 1967 Report	SCB					

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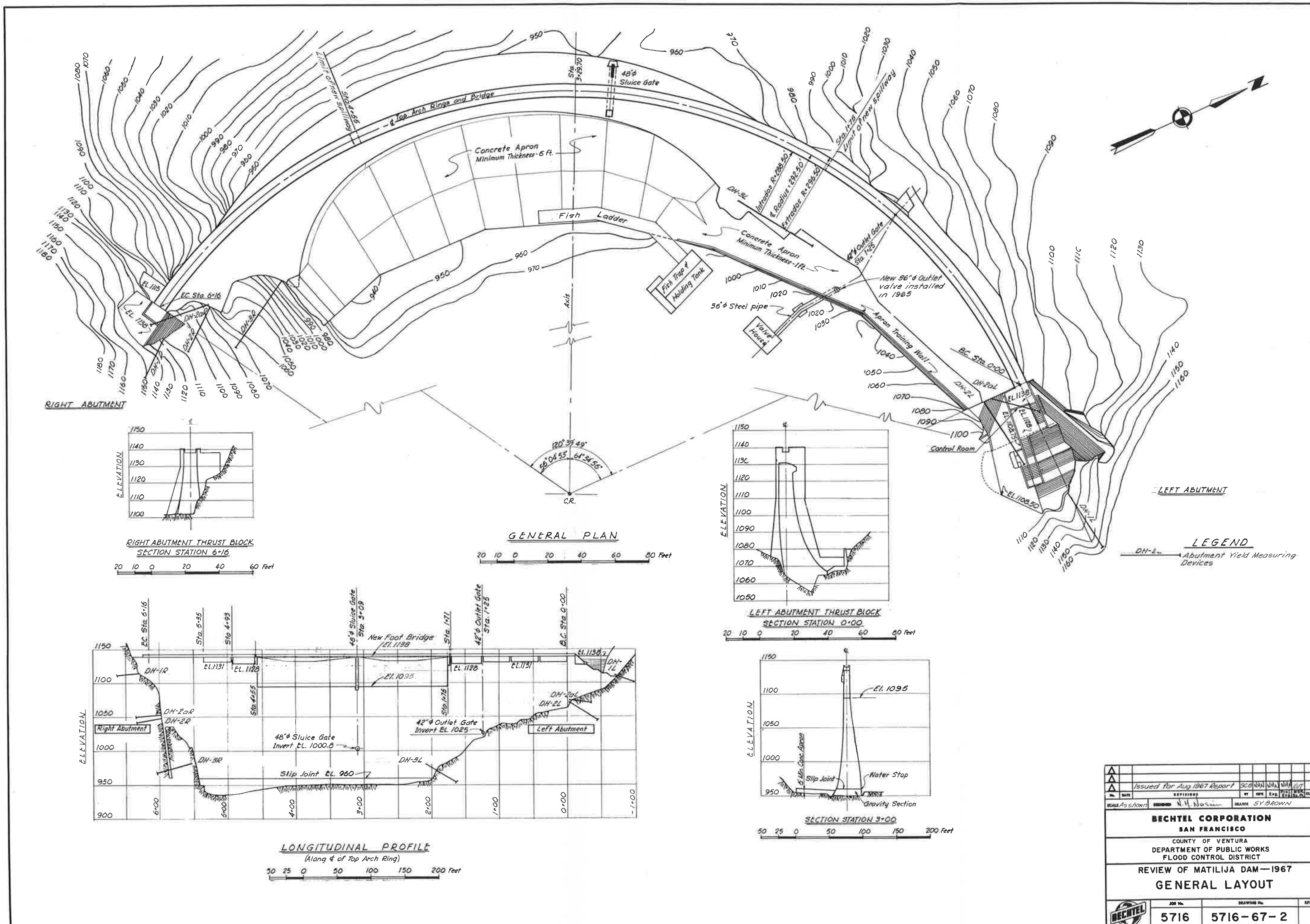
BECHTEL CORPORATION
SAN FRANCISCO

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
FLOOD CONTROL DISTRICT

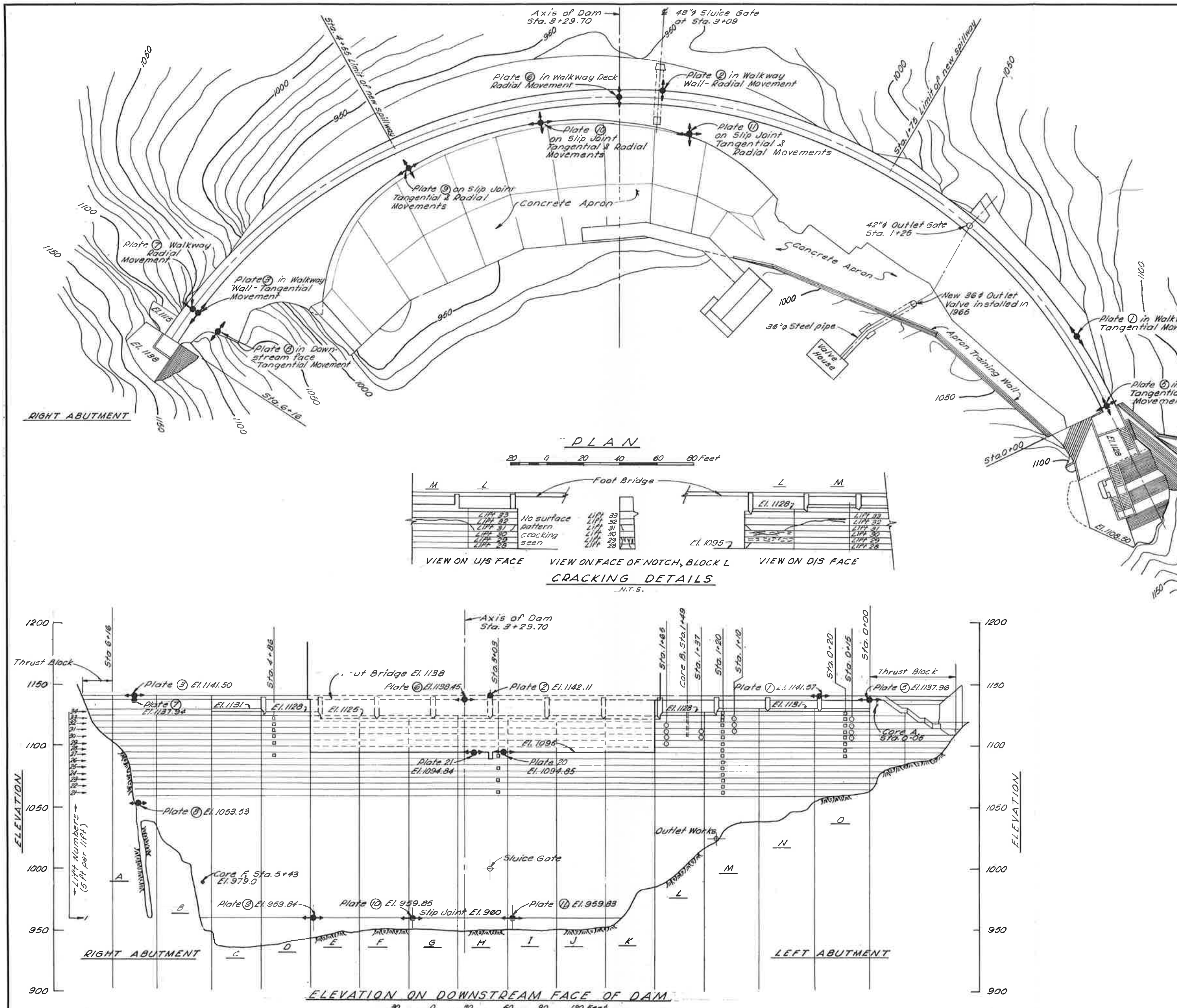
REVIEW OF MATILIJA DAM—1967

GENERAL LOCATION

JOB No.	DRAWING No.	REV.
5716	5716-67-1	



GENERAL PLAN
RIGHT ABUTMENT
LEFT ABUTMENT
Concrete Apron
Minimum Thickness - 6 ft.
Concrete Apron
Minimum Thickness - 1 ft.
Fish Ladder
Fish Trap & Holding Tank
Valve House
36"ϕ Steel pipe
New 96"ϕ Outlet valve installed in 1965
Apron Training Wall
48"ϕ Sluice Gate
42"ϕ Outlet Gate Sta. 1+25
Limit of new Spillway
Limit of new Spillway
℄ Top Arch Rings and Bridge
Intrados R=288.50
℄ Radius 292.50
Extrados R=296.50
Axis
C.R.
120°39'49"
56°04'53"
64°34'56"
EC Sta. 6+16
BC Sta. 0+00
Control Room
EL. 1115
EL. 1138
EL. 1108.50
LEGEND
DH-2L Abutment Yield Measuring Devices
RIGHT ABUTMENT THRUST BLOCK
SECTION STATION 6+16
LEFT ABUTMENT THRUST BLOCK
SECTION STATION 0+00
ELEVATION
LONGITUDINAL PROFILE
(Along ℄ of Top Arch Ring)
New Foot Bridge
El. 1138
El. 1095
48"ϕ Sluice Gate
Invert EL. 1000.8
42"ϕ Outlet Gate
Invert EL. 1025
Slip Joint EL. 960
Right Abutment
Left Abutment
SECTION STATION 3+00
El. 1095
Slip Joint
Water Stop
Gravity Section
Issued for Aug 1967 Report
Scale As shown
Drawn SY BROWN
BECHTEL CORPORATION
SAN FRANCISCO
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
FLOOD CONTROL DISTRICT
REVIEW OF MATILIJA DAM—1967
GENERAL LAYOUT
JOB No. 5716
DRAWING No. 5716-67-2



NOTES

1. The only plates shown on this drawing are those which have been surveyed long enough and frequently enough to yield meaningful data on the behavior of the dam.

LEGEND

- ○ Soniscope Testing performed by Bechtel.
- ▫ Soniscope Testing performed by PCA.

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	8/67	Issued for Aug. 1967 Report			

Scale As shown | Designed M. Morris | Drawn SY Brown

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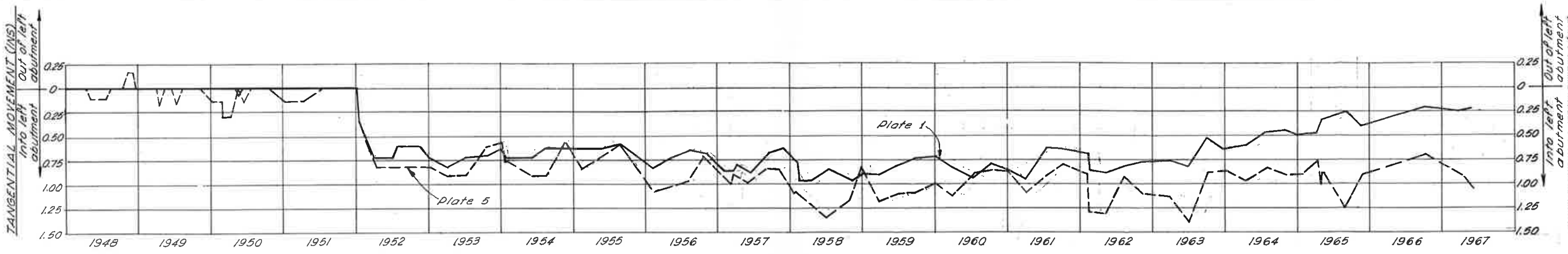
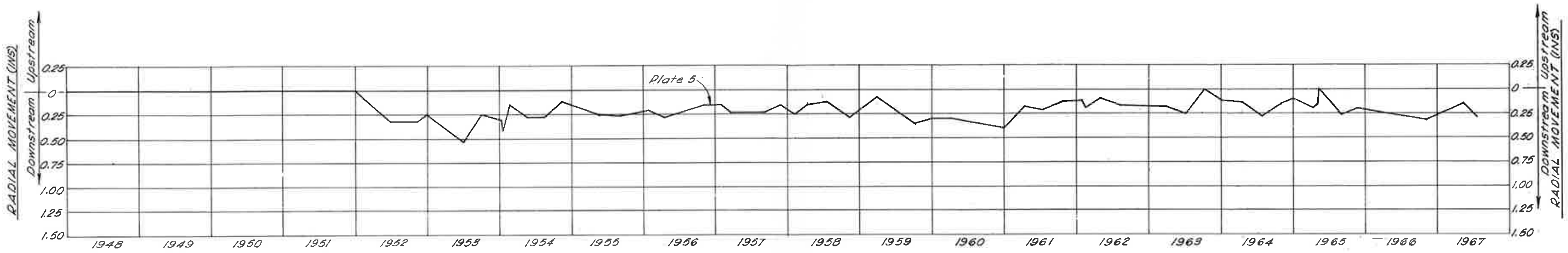
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
FLOOD CONTROL DISTRICT

REVIEW OF MATILIJA DAM—1967
DAM MOVEMENT
PLATE LOCATIONS AND ELEVATIONS

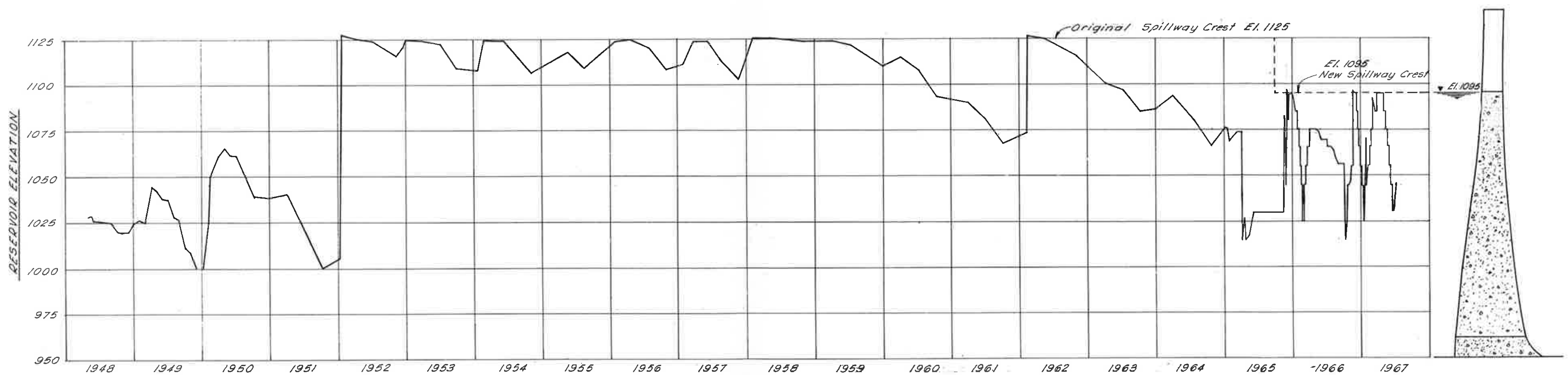
Job No. 5716 | Drawing No. 5716-67-3



TANGENTIAL MOVEMENT—PLATES 1 & 5



RADIAL MOVEMENT—PLATE 5



VARIATION OF RESERVOIR ELEVATION

NOTES

1. From 1948 to 1965 reservoir elevation was recorded on the dates of surveys only. From 1965 reservoir elevations have been recorded daily.

2. For location of plates 1 & 5 see Drawing 5716-67-3
   Plate 1 is embedded in the wall of the walkway at El. 1141.57

   Plate 5 is embedded in the deck of the walkway adjacent to the abutment concrete at El. 1137.96

No.	DATE	REVISIONS	BY	CHK	Eng.	Proj. Eng.	MGR Sp. Pr.	CLIENT
	8/67	Issued for Aug. 1967 Report	SCB	MM	MM	[illegible]	JUT	

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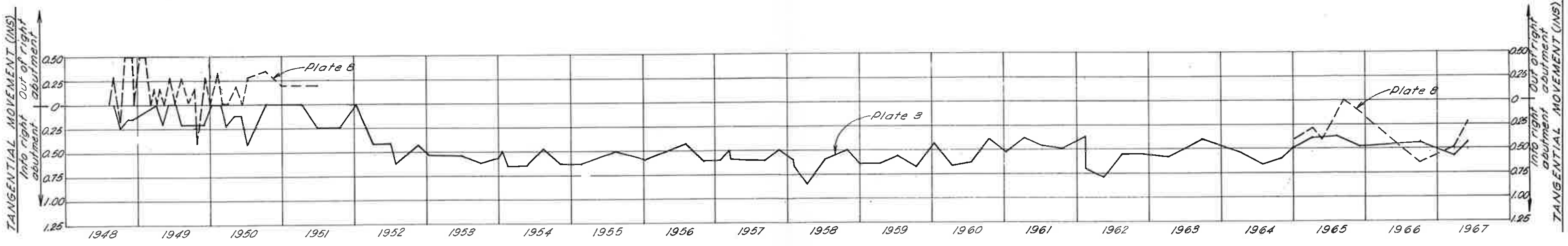
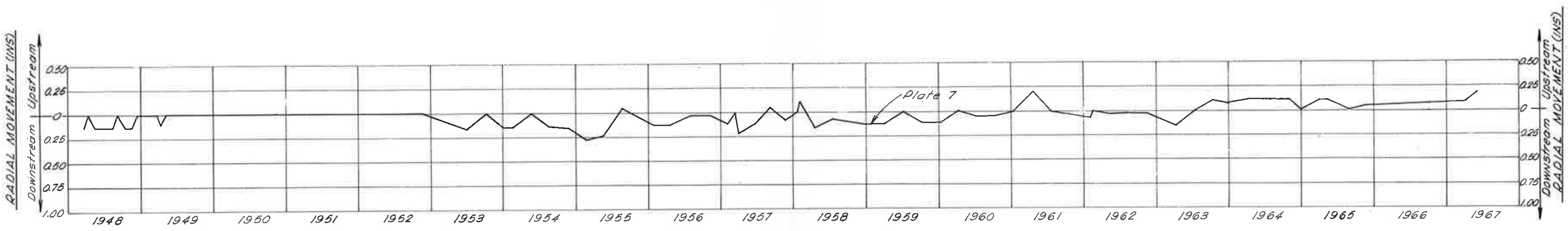
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORK
FLOOD CONTROL DISTRICT

REVIEW OF MATILIJA DAM—1967
DAM MOVEMENT AT LEFT ABUTMENT

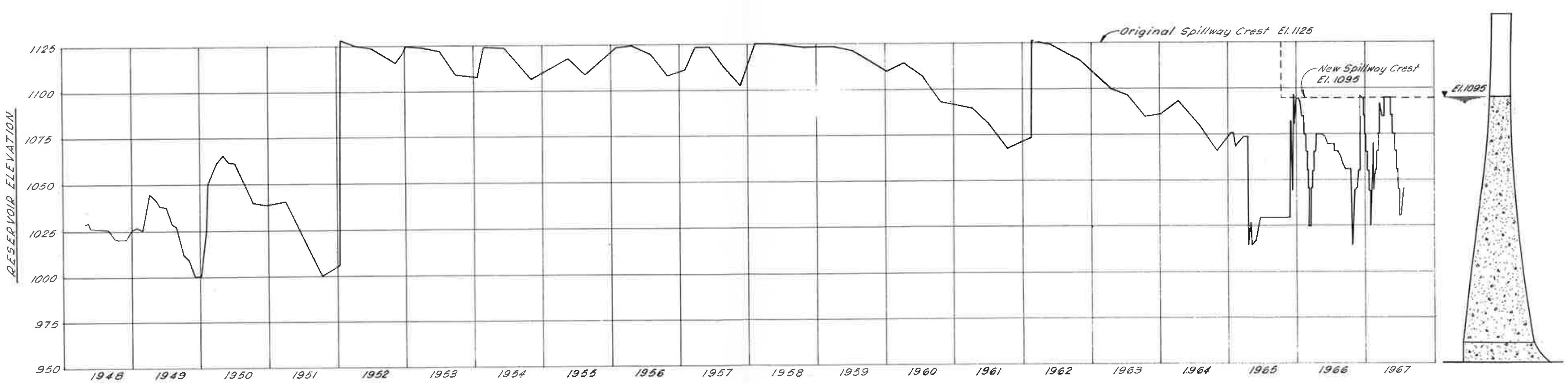
JOB No.	DRAWING No.	REV.
5716	5716-67-4	



TANGENTIAL MOVEMENT PLATES 3 & 8



RADIAL MOVEMENT PLATE 7



VARIATION OF RESERVOIR ELEVATION

NOTES

1. From 1948 to 1965 reservoir elevation was recorded on the dates of surveys only. From 1965 reservoir elevations have been recorded daily.

2. For location of plates 3, 7 & 8 see Drawing 5716-67-3
   Plate 3 is embedded in the wall of the walkway at El. 1141.50

   Plate 7 is embedded in the abutment concrete at El. 1197.94

   Plate 8 is located in the downstream face of the dam at El. 1053.53

No.	DATE	REVISIONS	BY	CH'K	Eng.	Proj. Eng.	Mgr. Sp. Pr.	CLIENT
	8/67	Issued for Aug. 1967 Report	SCB	MM	MM	MM	KVT	

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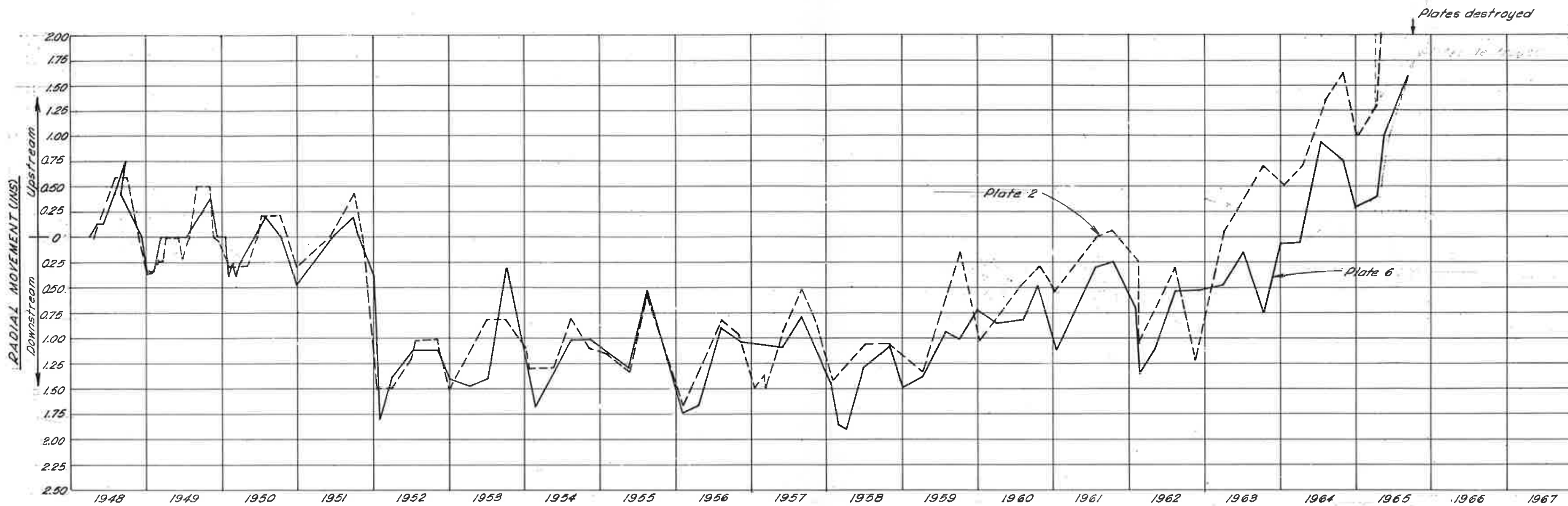
COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
FLOOD CONTROL DISTRICT

REVIEW OF MATILIJA DAM—1967
DAM MOVEMENT AT RIGHT ABUTMENT

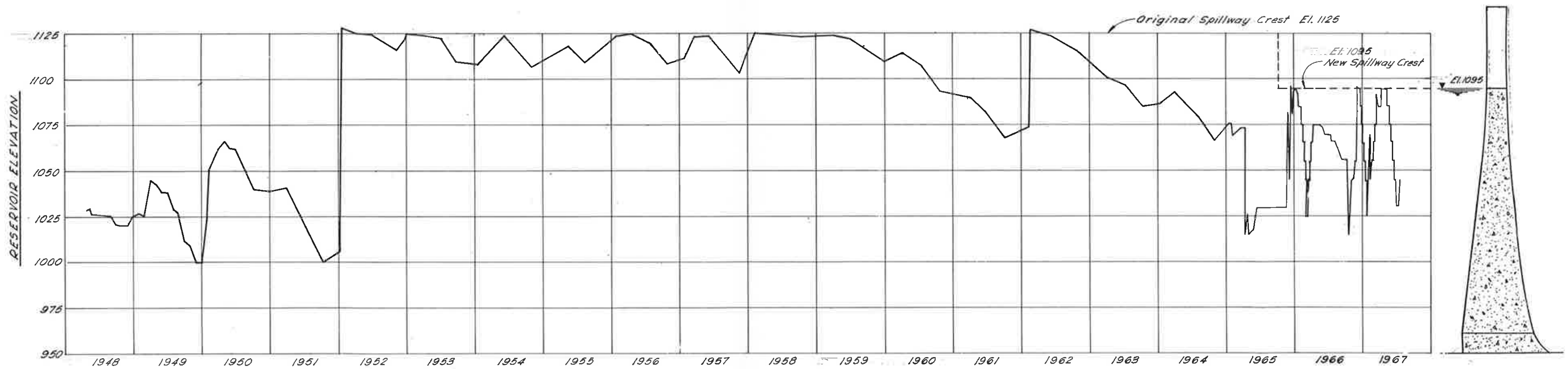
JOB No. 5716 | DRAWING No. 5716-67-5 | REV.



RADIAL MOVEMENT PLATES 2 & 6

NOTES

1. From 1948 to 1965 reservoir elevation was recorded on the dates of surveys only. From 1965 reservoir elevations have been recorded daily.
2. For location of plates 2 & 6 see Drawing 5716-67-8
   Plate 2 was embedded in the wall of the walkway at El. 1142.11
   Plate 6 was embedded in the deck of the walkway at El. 1138.46
   Both plates were destroyed when the notch was cut in August 1965.



VARIATION OF RESERVOIR ELEVATION

No.	DATE	REVISIONS	BY	CHK	Eng.	Proj. Eng.	MGR. Sc. Pl.	CLIENT
	8/67	Issued for Aug. 1967 Report	SCB	M.M.	M.M.	WN	LVT	

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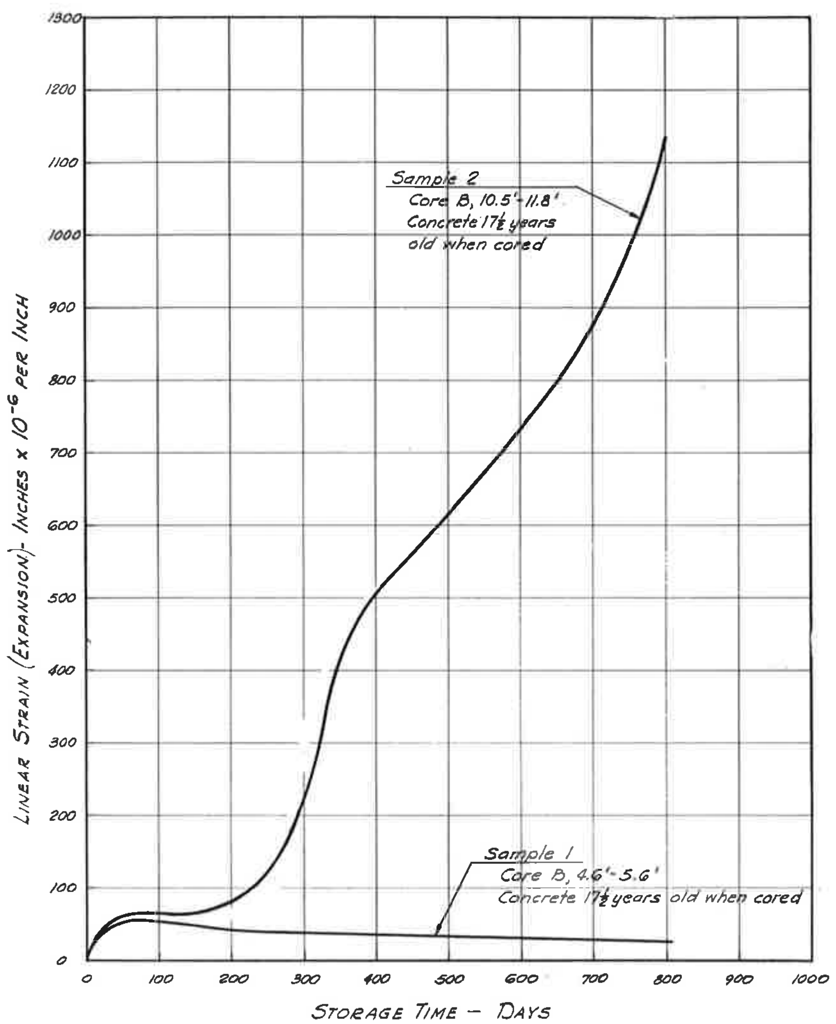
DEPARTMENT OF PUBLIC WORKS
FLOOD CONTROL DISTRICT

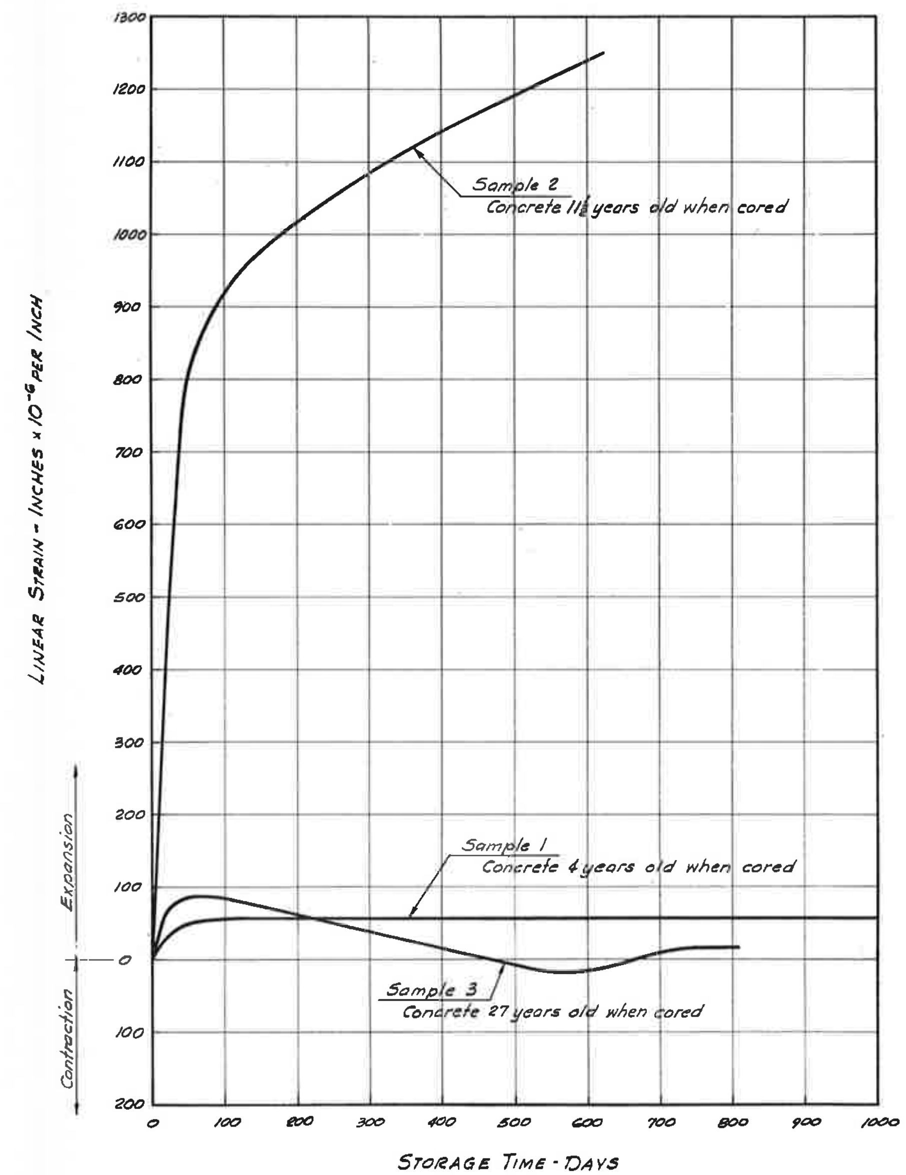
REVIEW OF MATILIJA DAM—1967

DAM MOVEMENT AT CROWN

BECHTEL — JOB No. 5716 — DRAWING No. 5716-67-6 — REV.



EXPANSION OF CONCRETE FROM MATILIJA DAM



EXPANSION OF CONCRETE FROM DAM X
FOR COMPARISON PURPOSES

NOTES:

TESTS ON MATILIJA DAM

Samples 1 & 2
17½ years old when cored.
From 5" diameter core.
Stored in sealed containers at 70°F.

TESTS ON DAM X

Sample 1
4 years old when cored.
From 6" diameter core.
Stored in sealed container at 70°F.

Sample 2
12 years old when cored.
3"×3"×6" bar, cut from core.
Stored in water at 100 F.

Sample 3
27 years old when cored.
From 6" diameter core.
Stored in sealed container at 70°F.

No.	DATE	REVISIONS	BY	CH'K	Eng.	Proj. Eng.	Mgr. Sp. Pr.	CLIENT
	8/67	Issued for Aug. 1967 Report	SCE	MM	MM			

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BECHTEL CORPORATION
SAN FRANCISCO

COUNTY OF VENTURA
DEPARTMENT OF PUBLIC WORKS
FLOOD CONTROL DISTRICT

REVIEW OF MATILIJA DAM—1967
VOLUME CHANGE TESTS

JOB No. 5716 · DRAWING No. 5716-67-7 · REV.