File FC-1002.6

MATILIJA DAM

F. E. BONNER 5/25/49

MEMORANDUM RE

MATILIJA DAM - VENTURA COUNTY

INDEX

PLATES

Plate No.

F. E. Bonner
Consulting Engineer
San Francisco, California

May 25, 1949

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MEMORANDUM RE
MATILIJA DAM - VENTURA COUNTY

1. INTRODUCTION:

Under date of October 23, 1948, a preliminary report was submitted covering the results of my studies up to that time regarding the adequacy of the Matilija Dam and the remedial measures which might be desirable to assure its safety. Further investigation was accomplished subsequently but, in coordination with the geologists studying the foundation, conclusions have been deferred in expectation that runoff of the 1948-49 season would provide enough storage so that the behavior of the dam under load could be observed carefully. Since it is now evident that such opportunity will not be available before next winter, the results of my studies are summarized and presented herewith.

As indicated by the preliminary report of October 23, 1948, the paramount question of the capability of the foundation to support the structure which has been built upon it is being determined by the eminent geologists employed by the County, and therefore my analysis has been restricted largely to two secondary features; first, the adequacy of the arch structure to withstand the water load that will be imposed upon it and second, the sufficiency of provisions for passing large flood flows with safety.

2. ADEQUACY OF ARCH STRUCTURE:

With the improvement of the technique of making high strength concrete in recent years, the tendency has been toward the allowance of greater stress limits in the design of arch dams. In the design of the Matilija arch full advantage was taken of this tendency and the resulting structure has more slender proportions than usual. A general check of the arch stresses shows rather high values over large sectors but they can hardly be rated as dangerously high for good quality concrete. Plate 1 herewith shows the stresses to be expected under full water load. It will be observed that the crown stresses at the extrados range slightly above 800 p.s.i. up to elevation 1070 and more than 900 p.s.i. may develop at the intrados of the abutments. In current conservative practice a maximum working stress of 800 p.s.i. is customary. On the other hand, the record of the Smith Emery Laboratory tests of the concrete placed in the arch indicates average 28 day compressive strength of approximately 4500 p.s.i. This is better quality than usual and serves to compensate for the high unit working stress of the design. The factor of safety of 4 to 5 which is provided is about the margin considered necessary to allow for indeterminate internal stresses arising from other forces than the water load. One feature not to be overlooked is the added load which may be placed on the arch by the accumulation of silt in the reservoir. It is known that the tributary watershed rates high in silt production. No investigations have been made in this specific basin, but conditions are similar to the upper Santa Ynez basin of 218 square miles immediately to the north where reliable silt accumulation figures are available. Forest Service studies show that during the period 1920 to 1948 the silt volume accumulated in Gibraltar reservoir, together with the Caliente, Mono and Juncal debris storage basins above, amounted to 8363 acre feet. Thus the average was 300 acre feet a year or 1.37 acre feet per square mile. Application of this rate to the 55 square miles of Matilija basin indicates annual average of 75.35 acre feet. By the end of 20 years therefore, it is possible that silt deposition against the dam will reach elevation 1060. (See capacity curve - Plate 2). Technical investigations have determined that to resist the added load of such depositions, debris dams should be designed with a horizontal pressure component of 90# per square foot instead of 62.5# for the water load alone. This increase of 44% in load up to the silt level could result in direct stresses at the abutments of the arch in excess of 1100 p.s.i. at the lower elevations with silt accumulation to el. 1060. Practice of erosion control on the watershed and sluicing at the dam may be effective in postponing and reducing the rate of silt deposition.

To summarize my opinion regarding the adequacy of the arch it may be stated that while conservative practice would require a structure of more ample proportions, particularly in view of the precarious foundation and the proposal to pass large flood flows by free overfall, the excellent character of the concrete construction compensates for the slimness of the cross section in such degree that the arch is safe against compression failure. This is on the assumption, of course, that the abutment and foundation rock as now fortified through grouting operations proves to be tight and unyielding. That question is in the hands of the geologists.

3. SPILLWAY FACILITIES:

The project plan contemplates the passage of floods up to a maximum of 60,000 sec. ft. over the crest of the dam. Admittedly a flood of such size would be very rare and of short duration. Runoff measurements of Matilija Creek have been maintained only from October, 1927. The erratic character of the discharge is pictured by the condensed summary of the record provided by Plate 3 herewith. The annual yield ranges from the low of 3380 A.F. in 1929 to 125,900 A.F. in 1941. The 20-year mean is only 26,500 A.F. (35.3 s.f.). Most of the time the stream is nearly dry but occasionally the heavy precipitation of violent winter storms produces sudden large flood flows. Twice within twenty years the peaks of such floods have reached 15,000 sec. ft. The brief duration of such high discharge is evidenced by the fact that the mean daily discharge of March 2, 1938 flood, which peaked at 15,900 s.f., was only 4850 s.f. and the mean daily of the Jan. 22, 1943 flood, cresting at 15,000 s.f., was only 3320 s.f. On the other hand the large volume of flow in February and March, 1941, which yielded 74,230 A.F., peaked at only 4290 s.f. By high water marks and other indications it has been estimated by E. E. Everett of Ventura that the flood of February 20, 1914 peaked at 22,900 s.f.

From this evidence it is fair to conclude that a 15,000 s.f. flood peak may be expected with average frequency of once in ten years. The frequency of larger floods is mere conjecture, but obviously over a 100-year period one or more floods of the magnitude of 25,000 or 30,000 s.f. are probable. The 60,000 s. f. flood adopted as the maximum by the designers of the dam appears to be within the range of possibilities from this watershed. It is likely that such an extraordinary flood would take the form of a sharp momentary peak and that the duration of discharge in excess of 15,000 s.f. would hardly be more than a few hours. Hydrograph of the March 2, 1938 flood is presented by Plate 4 herewith. The preceding month of February produced runoff of 16,260 A.F., and with the reservoir operated on a cyclic basis, primarily for water conservation rather than flood control, it is probable that little or no capacity would have been available to alter the dimensions of the flood peak.

The duration of the flow at different stages was as follows:

Over	1000	s.f.	96 hrs.
"	2000	"	30 "
"	3000	"	15 "
"	4000	"	9 "
"	5000	"	7 "
"	10000	"	3.5 "
"	12000	"	2 "

A secondary flood followed on March 12th and 13th and streamflow was maintained at more than 100 s.f. until April 12th. This is pertinent to the matter of repair of possible flood damage at the toe of the dam which will be discussed later.

The dimensions and elevations of the spillway bays on the crest of the dam are shown by Plate 5, which is a developed profile of the dam along the axis. A curve showing the spillway discharge with the reservoir water surface at different levels is given by Plate 6. It will be seen that for the maximum discharge of 60,000 s.f. the reservoir level would be at elevation 1137 or 12 ft. above the crest of the six main spillway bays at the center of the dam. The major part of the spillway discharge falls directly to the stilling pool at the toe of the dam, but the portion passed by the five spillway bays between sta. 0 and sta. 2+17 falls to the inclined channel formed by the concrete paving and training wall along the left abutment. Approximately 20% of the total spill would be discharged to the stilling pool by the inclined channel with floods ranging from 15,000 to 60,000 sec. ft. The effect of this high velocity lateral increment complicates calculation of the effect of the pool in partially dissipating the energy of the free overfall.

In order to portray the problem relating to flood discharge a series of diagrams have been prepared. Details of the maximum overpour at the crest are shown by Plate 7, while Plate 8 charts the theoretical positions of the upp er and lower surfaces of the falling water sheet down to el. 1040. Plate 9 presents a cross section of the dam in combination with the nappe and stilling pool. Plate 10 shows the computed profiles of various flood discharges in the half mile of stream channel immediately below the dam based on the detailed topographic survey executed by the County Engineers' office in October,1948.

The main question is whether the stilling pool at the base of the dam provides adequate protection against undermining of the dam at the downstream toe. A concrete slab 6 ft. thick has been laid on the bottom of the pool area to resist the erosive power of the flood discharge. To appreciate the ramifications of the problem it is necessary to clearly visualize the enormous physical forces exerted by the falling water of a large flood. With the free vertical fall of some 130 feet the velocity at the point of impact on the pool will be approximately 90 ft. per second. The discharge of 15,000 sec. ft. represents a falling mass of 28,000 tons per minute. In terms of energy generated by the 130 ft. fall this is equivalent to over 7 billion foot-pounds per minute, or approximately 225,000 horse power. The quantities would be four times as much for the 60,000 s.f. flood.

Obviously a portion of this great volume of energy will be dissipated in the tumultuous churning of the stilling pool and in some circumstances a further portion would be used up in forming the well known phenomena termed the hydraulic jump. In this case, however, due to the depth of stilling pool, and to the depth and velocity of outpour from the pool, the hydraulic jump tendency will be submerged and characterized merely by a standing wave or series of waves below the fall. Many attempts have been made by mathematical treatment and model experiments to derive general laws which may apply to stilling pool behavior, but they have contributed but little that can be applied to different situations. The present case is complicated also by the lateral discharge from the inclined channel from the left abutment and the unstable character of the spoil bank downstream from the pool. From study of the hydraulics of the channel discharge below the pool it appears reasonable to assume that not more than 1/4th of the energy will be consumed in providing velocity head and that 3/4ths must be dissipated in the pool. As shown by Plate 9 the falling water sheet will impinge near the downstream edge of the concrete apron and maximum velocities at the bottom of the pool would occur at that point. Reliable quantitative estimates of such velocities are not practicable, but even with a 40 ft. pool depth I am convinced they would be sufficient to possess great erosive power. The Board of Geologists state that the concrete apron rests upon crushed and leached sandstone that will erode easily. Casual field inspection readily discloses such weak material in the exposed formation at the downstream edge of the apron. The stilling pool affords enough depth to minimize the erosive effect of moderate spill, but with the raging agitation accompanying the impact of large flood overpour it is practically certain that undercutting of the apron will begin at the downstream edge and cause progressive failure which might well reach and undermine the dam itself.

It has been suggested that damage of this kind from a single flood would not be so extensive as to prevent repairs to be made in advance of further heavy spill. Such repair would have to wait until the discharge from the dam diminished to less than the capacity of the confined outlet. As shown heretofore streamflow was sustained above 100 sec. ft. for 6 weeks after the 1938 flood, and in connection with the successive floods of February and March, 1941, no opportunity would have been afforded for inspection and repair of damage until May. It is evident, therefore, that prompt repair of damage cannot be relied upon, and moreover, there can be no assurance that the extent of apron failure from a single flood will not jeopardize the safety of the dam.

Attention should also be directed to possible damage to the apron from uplift forces. Even with the low reservoir level which has so far existed, considerable water is flowing from the relief holes. It is not certain whether drainage facilities under the apron are sufficient and the situation should be watched carefully to make sure that hydrostatic uplift arising from higher reservoir levels is adequately relieved.

4. IMPROVEMENTS NEEDED:

Various alternatives have been considered for eliminating the risk of failure of the dam through flood erosion. First thought naturally turned to extending the apron and thickly paving the stream channel downstream far enough to effect a cutoff wall connection to a sound rock formation. The difficulty of this course is to find such a formation within a reasonable depth and distance. The geological investigations fail to indicate promise of foundation material in the downstream channel any better than that now in evidence at the edge of the apron. Also considering the large amount of excavation and concrete that may be found necessary to accomplish such an undertaking in a satisfactory manner, it is evident that the construction cost would be substantial.

A wholly independent spillway as a substitute for the flood discharge over the dam would have numerous advantages.

The capacity would have to be large enough to take all ordinary flood water in order that spill over the crest of the dam would occur only for a brief interval during extreme floods. If the separate spillway was made capable of by-passing 16,000 sec. ft. before crest spill began it would be adequate. It was first considered that such a spillway might be developed by a tunnel through the narrow ridge separating the reservoir from the North Fork at a point one-fourth mile north of the dam. Survey of this area by the County Engineer in November, 1948 disclosed that the available drop is insufficient. Three concrete lined tunnels each 15 ft. in diameter and 600 ft. long would be needed to provide the required capacity.

Further study demonstrated that the most favorable prospect for a separate spillway comprises a tunnel around the right abutment of the dam. A preliminary plan is presented by Plate 11 herewith. Plate 12 shows the design in profile. The concrete entrance weir, with crest at elevation 1115 would consist of three bays, each 45 ft. wide and discharging to steep inclined tunnels of 10 ft. diameter. At the bottom of the 45° slope the three tunnels would be merged by transition section into a single concrete lined tunnel of 16.5 ft. diameter extending 650 ft. on 7.5% grade to a junction with Matilija Creek just above the Springs. With reservoir water level at elevation 1125 this spillway would discharge about 16,000 sec. ft. In the opinion of the District's geologists the sandstone ledge formations, through which the tunnel would penetrate at right angles, is entirely suitable for this type of construction. The cost of construction is estimated at $670,000 without crest gate control. The effective capacity of the reservoir would be reduced 1200 acre feet. In the event crest gates were added to avoid this loss of storage capacity, the cost would be increased about $60,000. Map data upon which the plan was based is subject to question and reliable surveys are needed before detailed layout, quantity calculations and cost estimates may be undertaken. This is particularly true at the site of the weir.

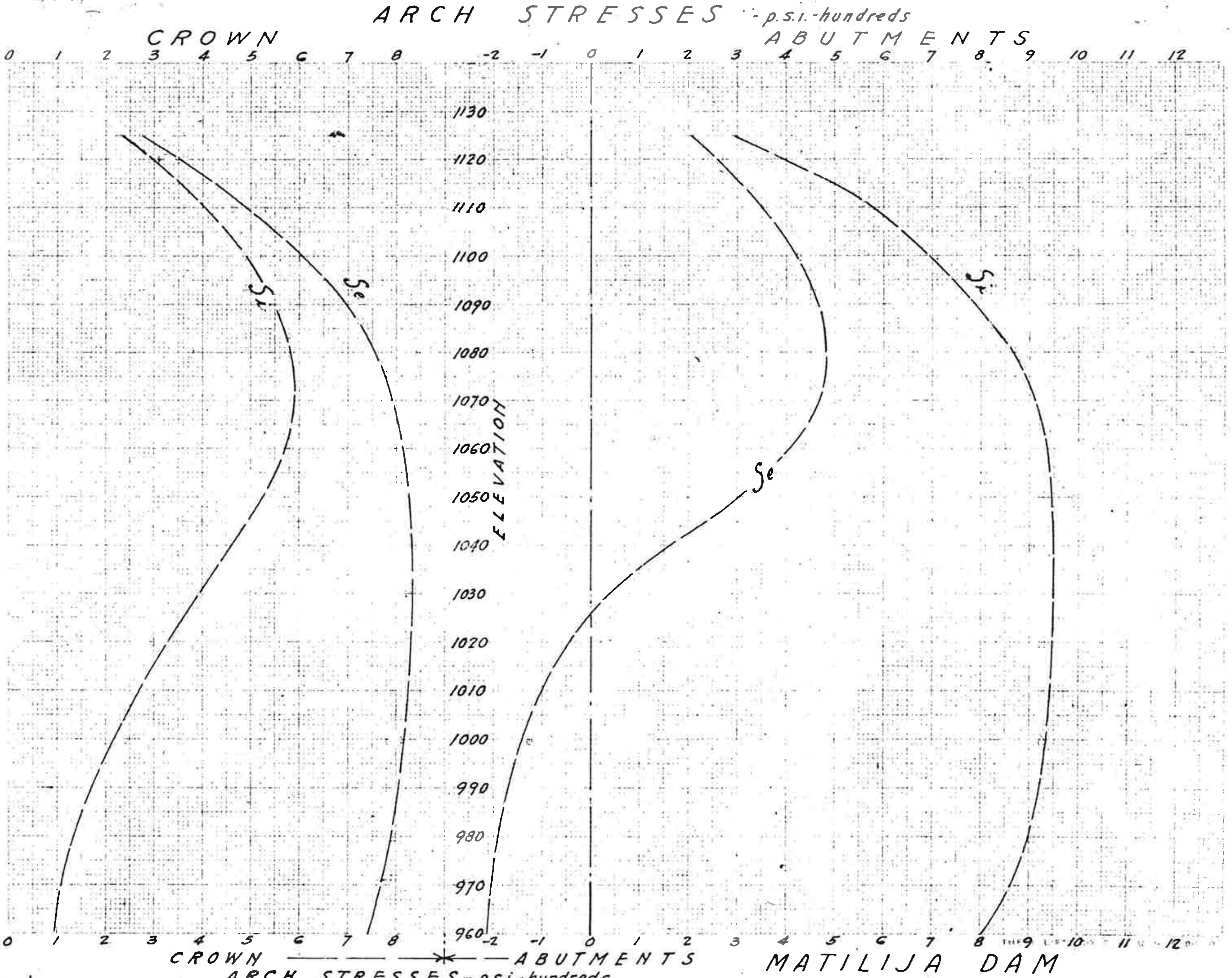
5. RECOMMENDATIONS:

In harmony with the foregoing discussion and as the first step toward early improvement of the spillway facilities of the dam, it is recommended that:

(a) Investigation by core drilling and other appropriate means be undertaken to determine suitability of the rock formation of the stream channel (for a distance of 500 ft. below the dam) to support a competent concrete lined flood channel;

(b) Topographic survey and geological investigation be made of the area involved in the suggested tunnel spillway around the right abutment of the dam to serve as a basis for detailed plans and estimates.

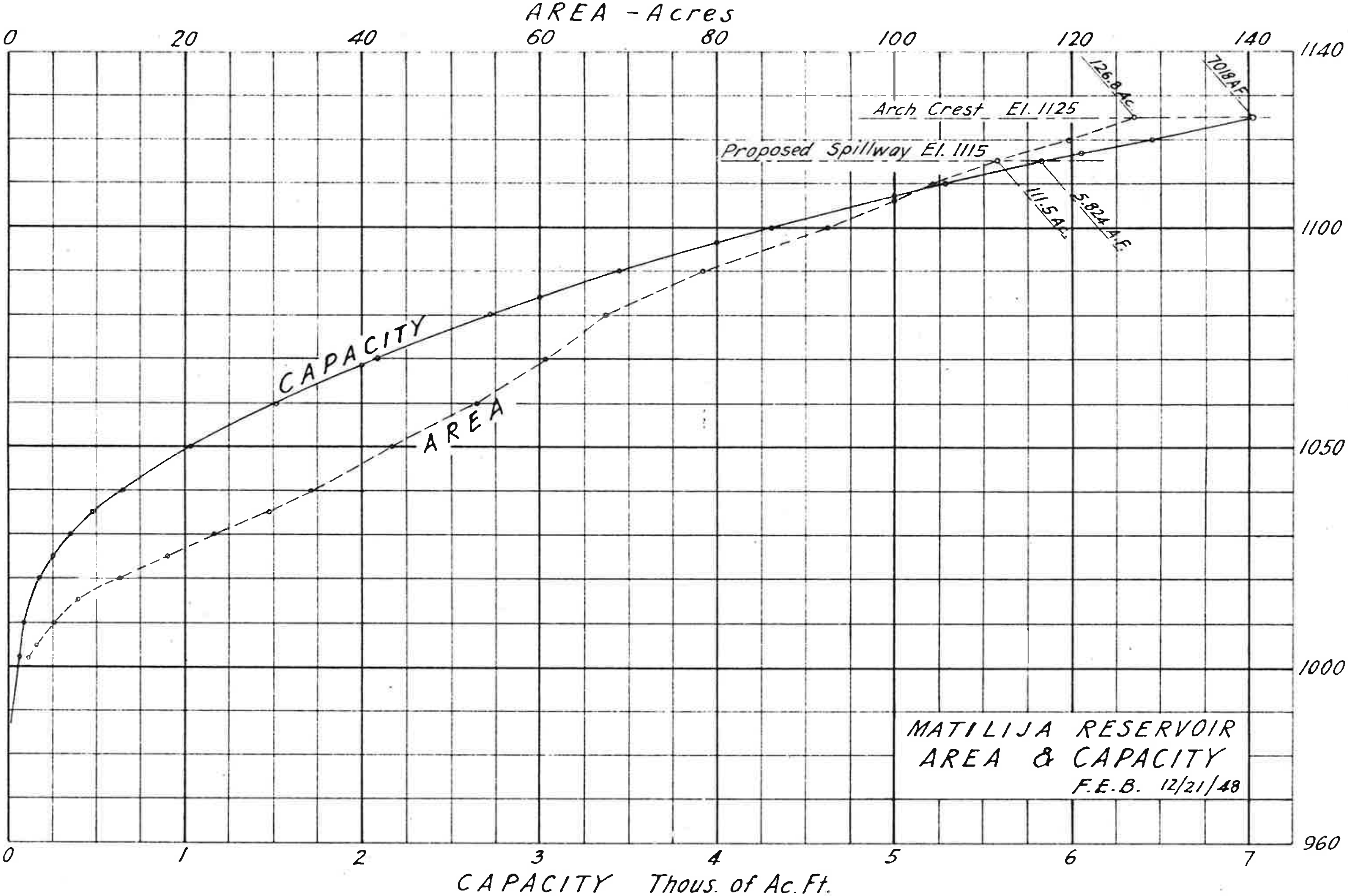
F. E. Bonner
Consulting Engineer
San Francisco, California



MATILIJA DAM

F. E. B. 9/20/48

PLATE 1



Values from Jamison Table

PLATE 2

PLATE 3

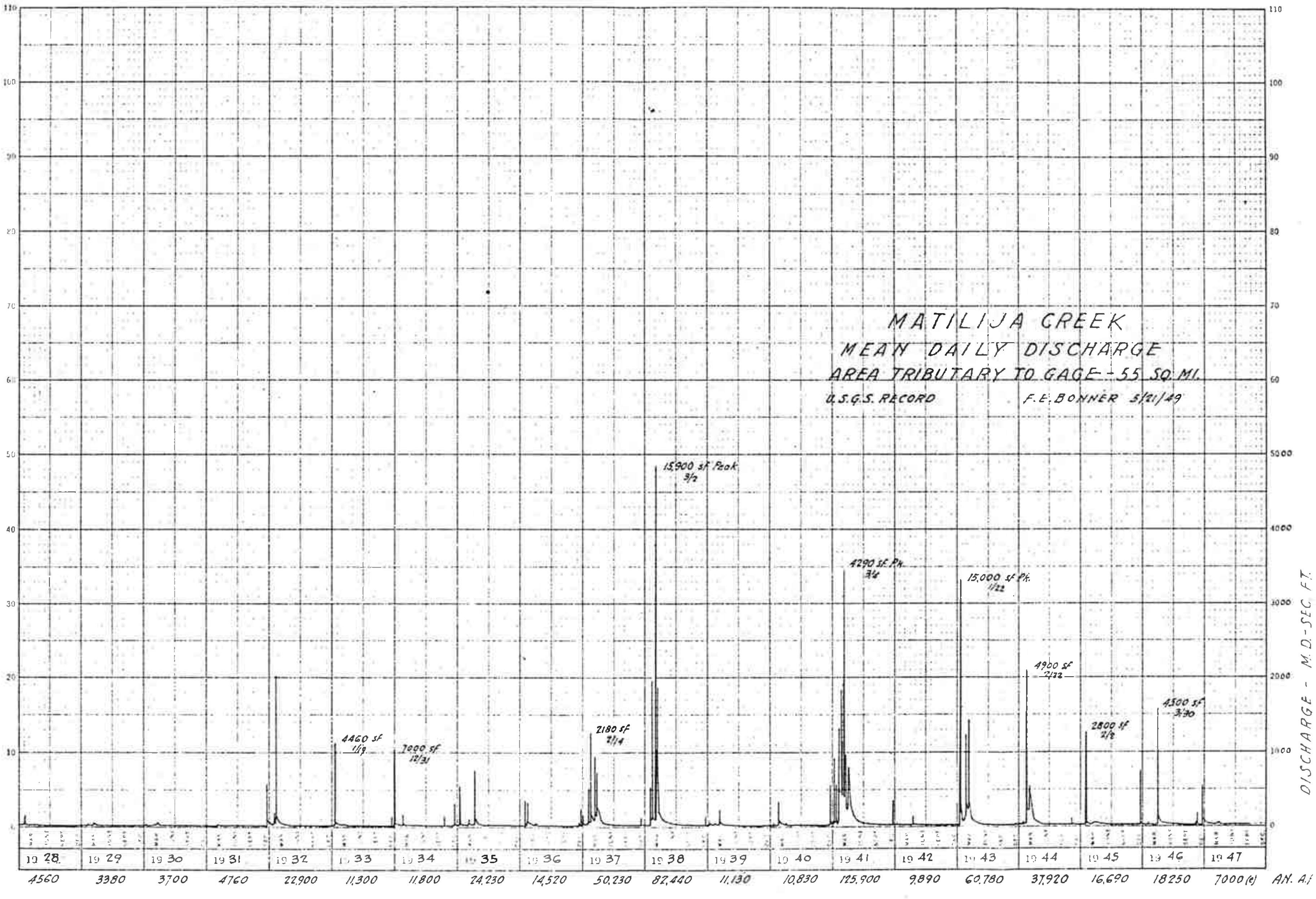
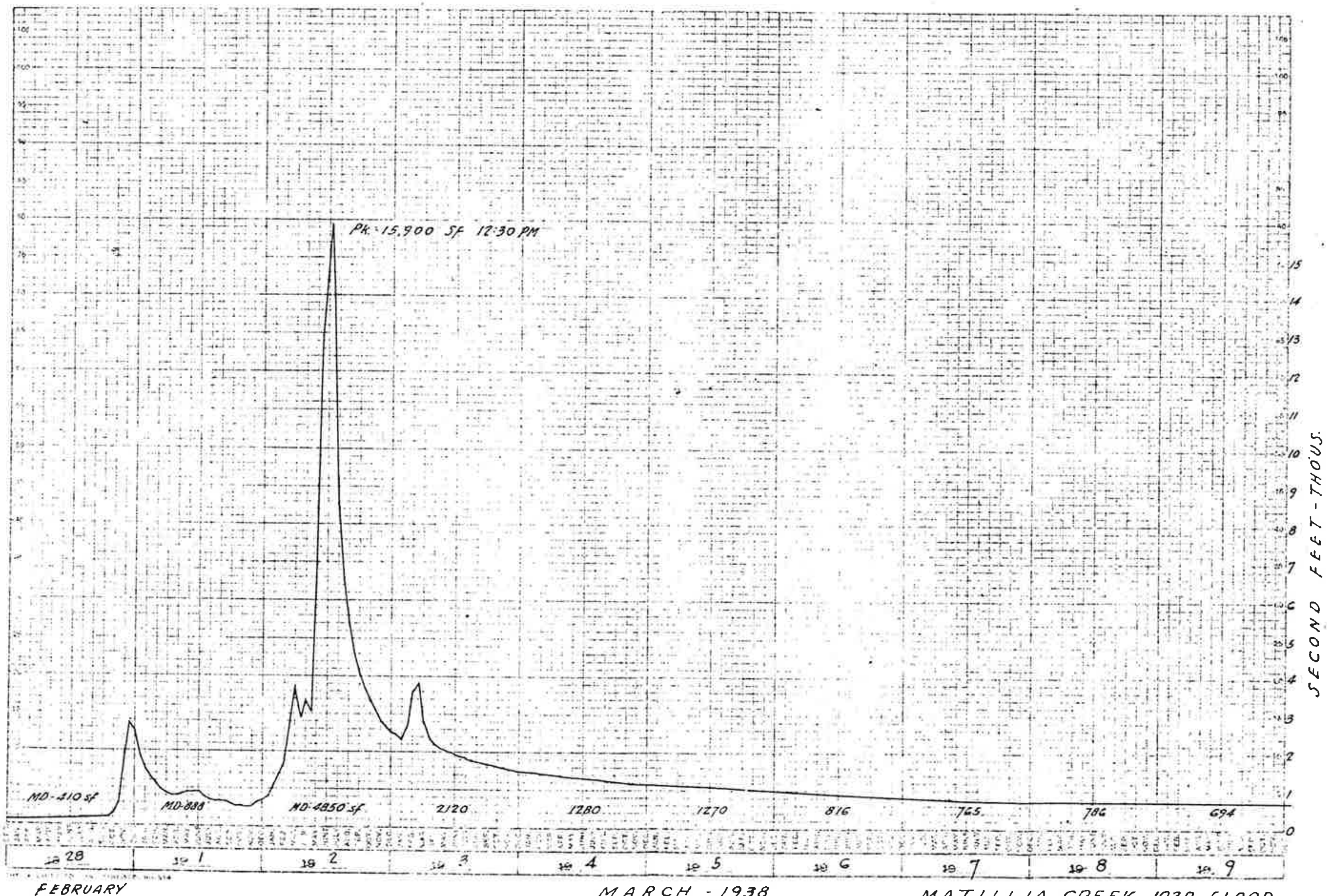
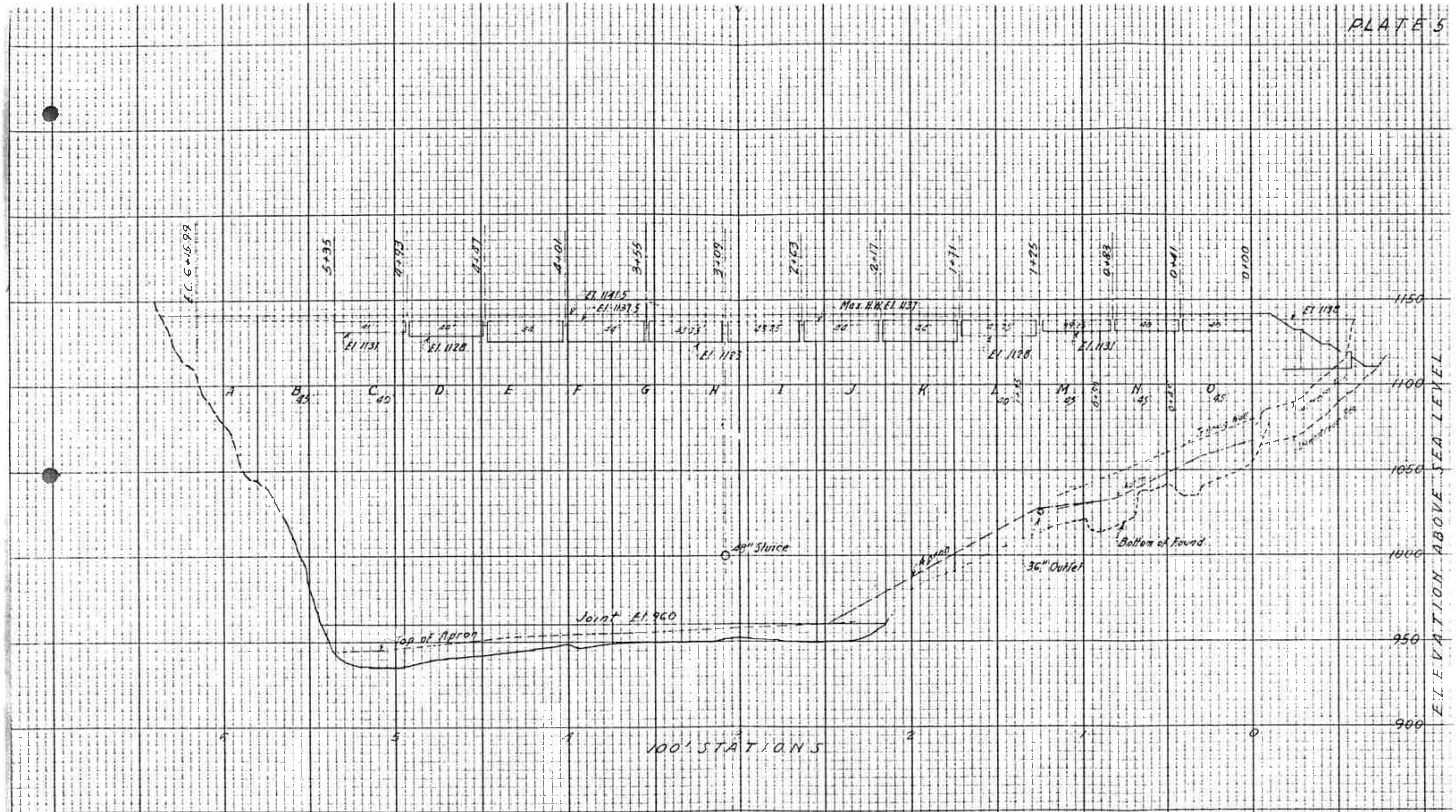

MATILIJA CREEK
MEAN DAILY DISCHARGE
AREA TRIBUTARY TO GAGE - 55 SQ. MI.
U.S.G.S. RECORD
F.E. BONNER 5/21/49
15,900 sf Peak 3/2
4290 sf Pk. 3/4
15,000 sf Pk. 1/22
4900 sf 2/22
4500 sf 3/30
2800 sf 2/2
2180 sf 2/14
4460 sf 1/9
7000 sf 12/31
DISCHARGE - M.D. SEC. FT.
1928
1929
1930
1931
1932
1933
1934
1935
1936
1937
1938
1939
1940
1941
1942
1943
1944
1945
1946
1947
4560
3380
3700
4760
22,900
11,300
11,800
24,230
14,520
50,230
87,440
11,130
10,830
125,900
9,890
60,780
37,920
16,690
18250
7000 (e)
AN. A.F.

PLATE 4



MATILIJA CREEK - 1938 FLOOD
WSP 844, p. 272 F.E.B. 12/6/48

PLATE 5



MATILIJA DAM
PROFILE ALONG AXIS

F. E. BONNER 11/13/..

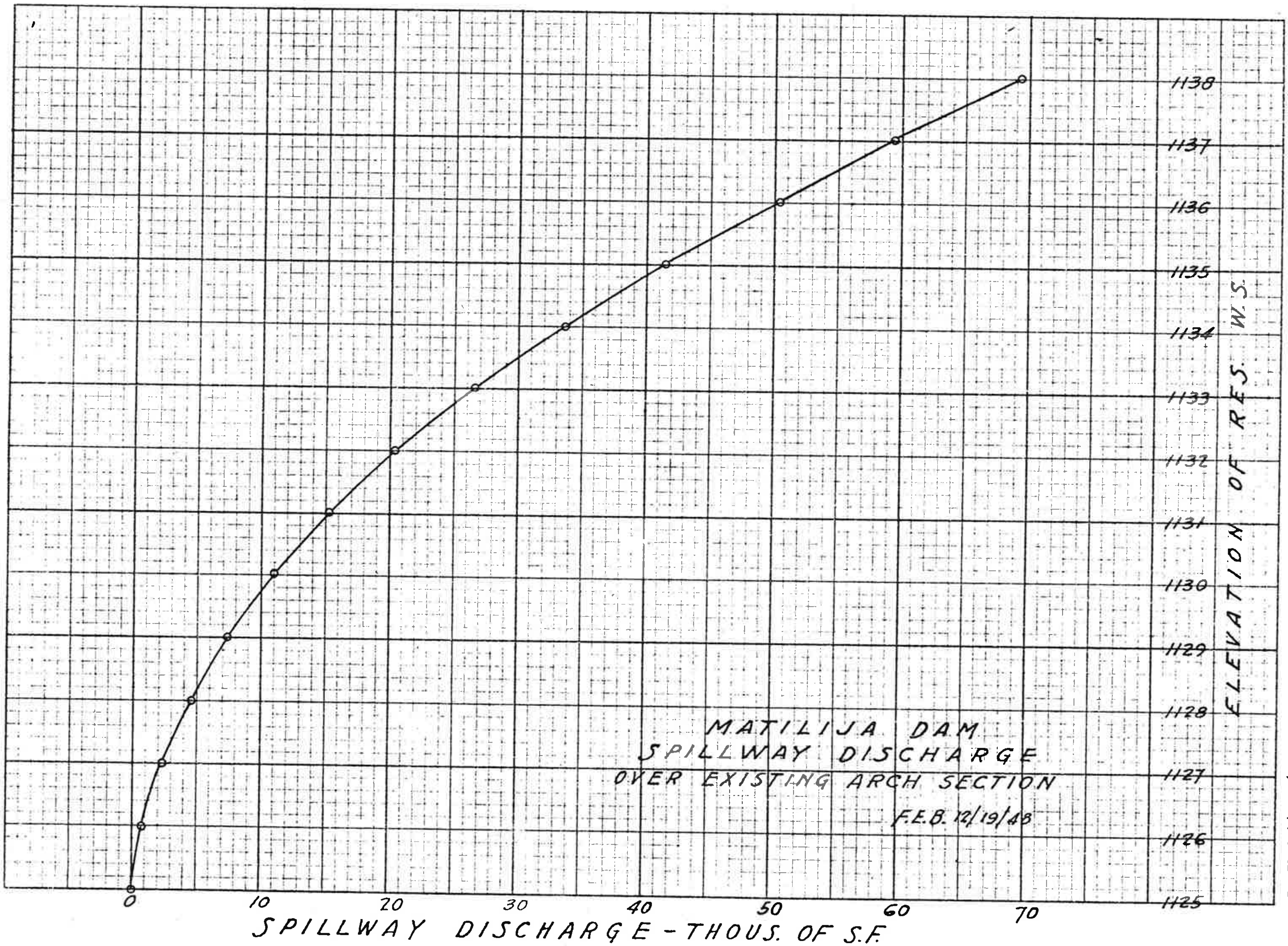
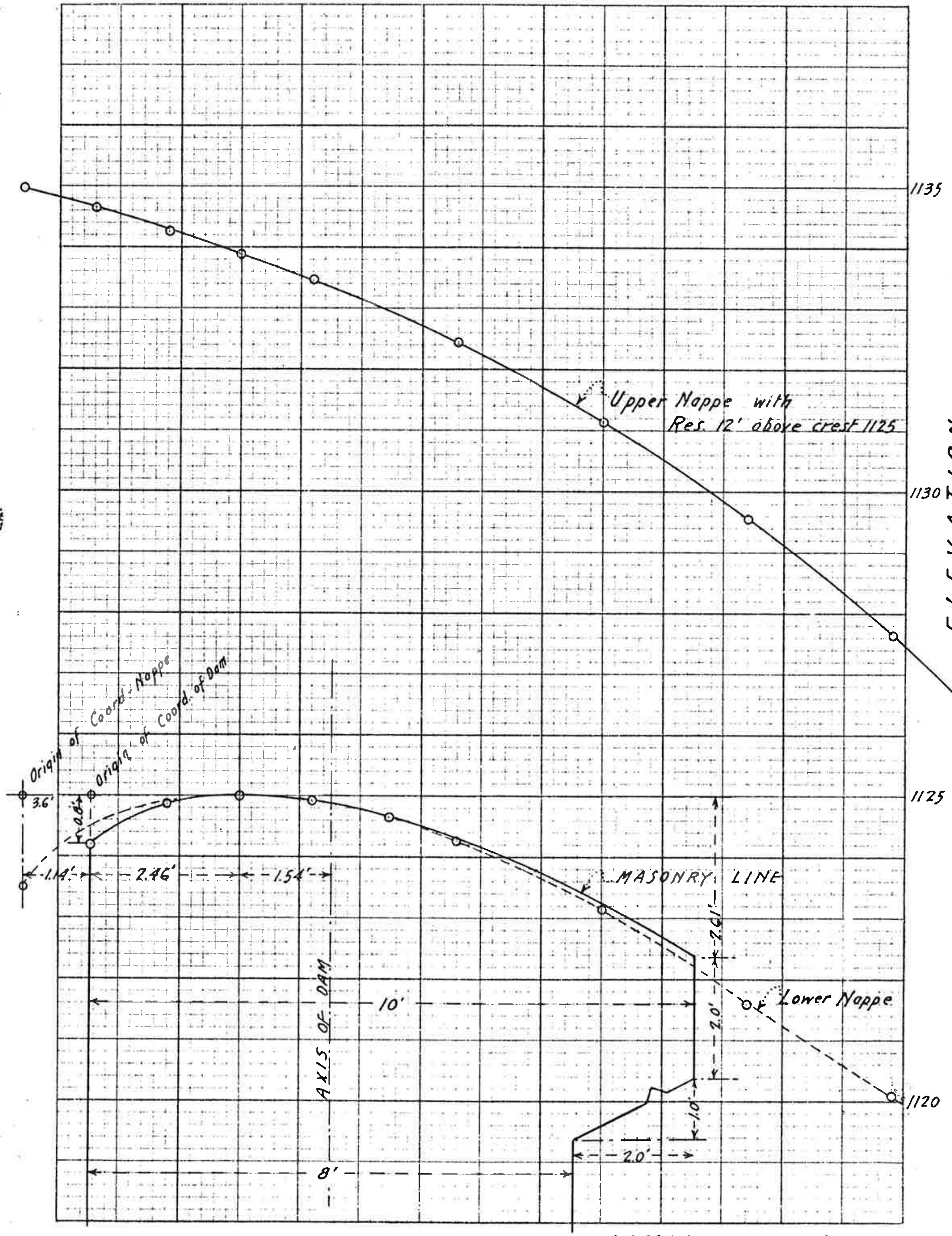

MATILIJA DAM
SPILLWAY DISCHARGE
OVER EXISTING ARCH SECTION
F.E.B. 12/19/48
ELEVATION OF RES. W.S.
1125
1126
1127
1128
1129
1130
1131
1132
1133
1134
1135
1136
1137
1138
0
10
20
30
40
50
60
70
SPILLWAY DISCHARGE - THOUS. OF S.F.

PLATE 6

PLATE 7



MATILIJA DAM
10/18/48 F.E.B.

PLATE 8

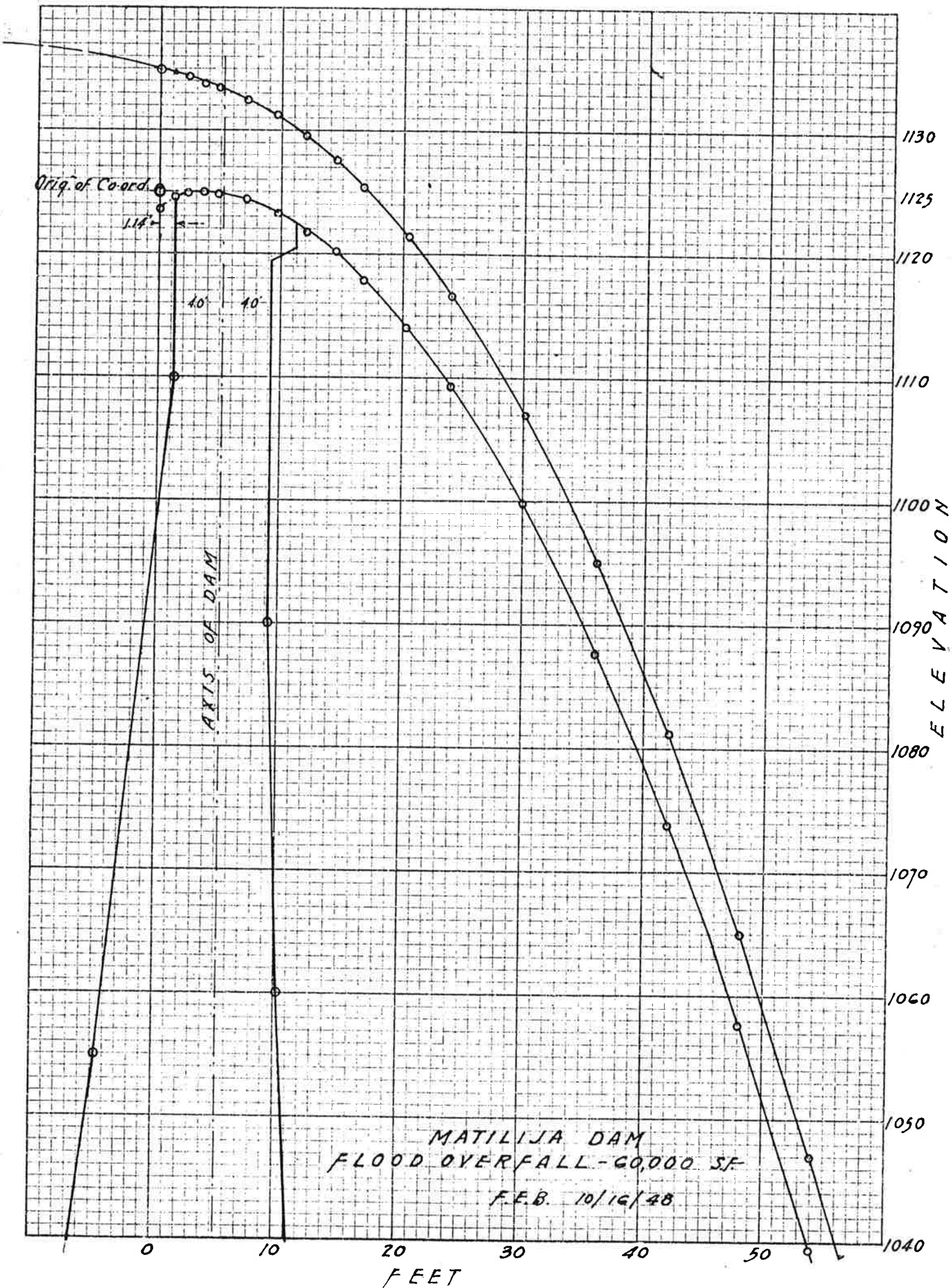

MATILIJA DAM
FLOOD OVERFALL - 60,000 S.F.
F.E.B. 10/16/48
Orig. of Co-ord.
1.14'
40'
40'
AXIS OF DAM
ELEVATION
1130
1125
1120
1110
1100
1090
1080
1070
1060
1050
1040
FEET
0
10
20
30
40
50

PLATE 9

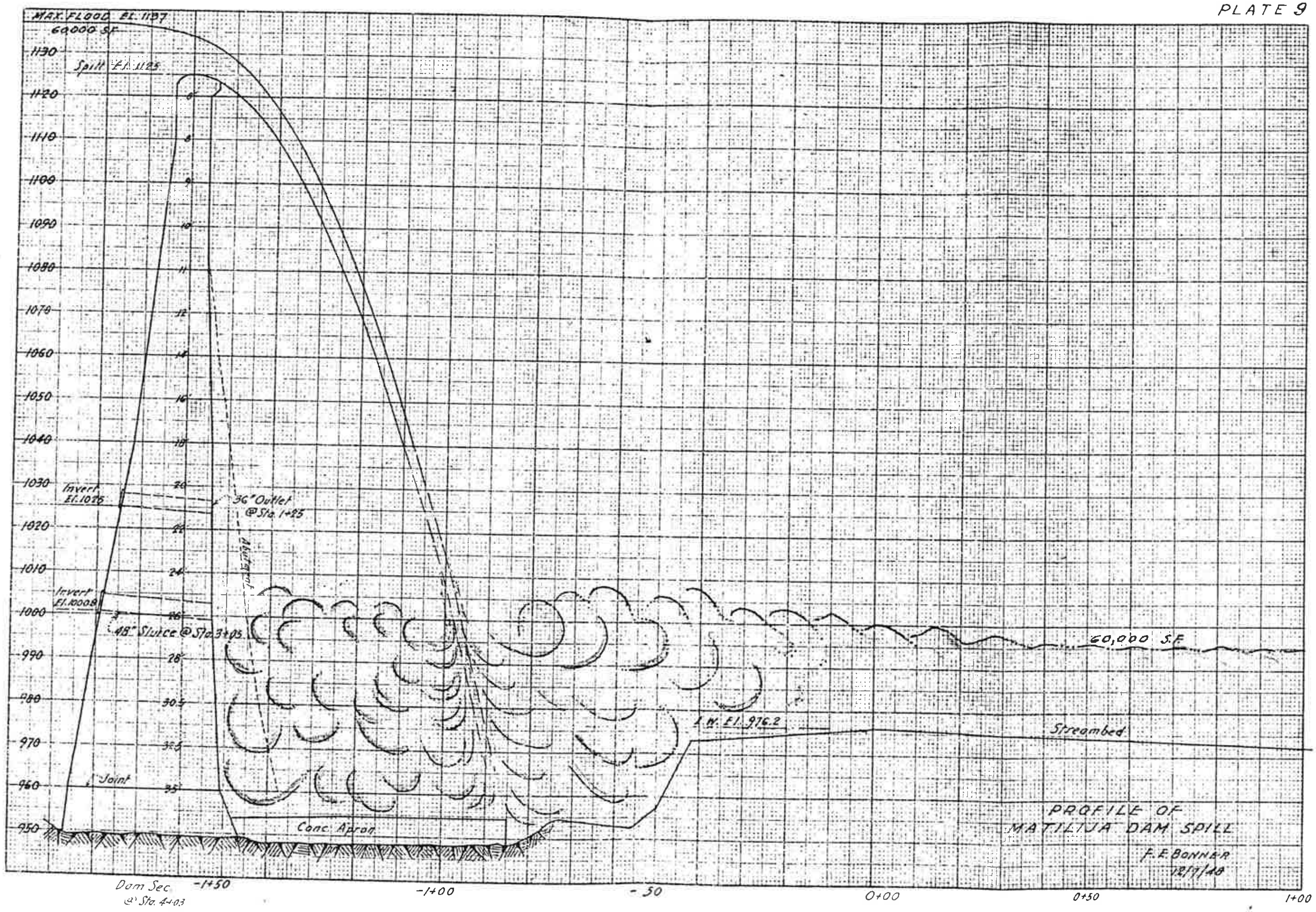
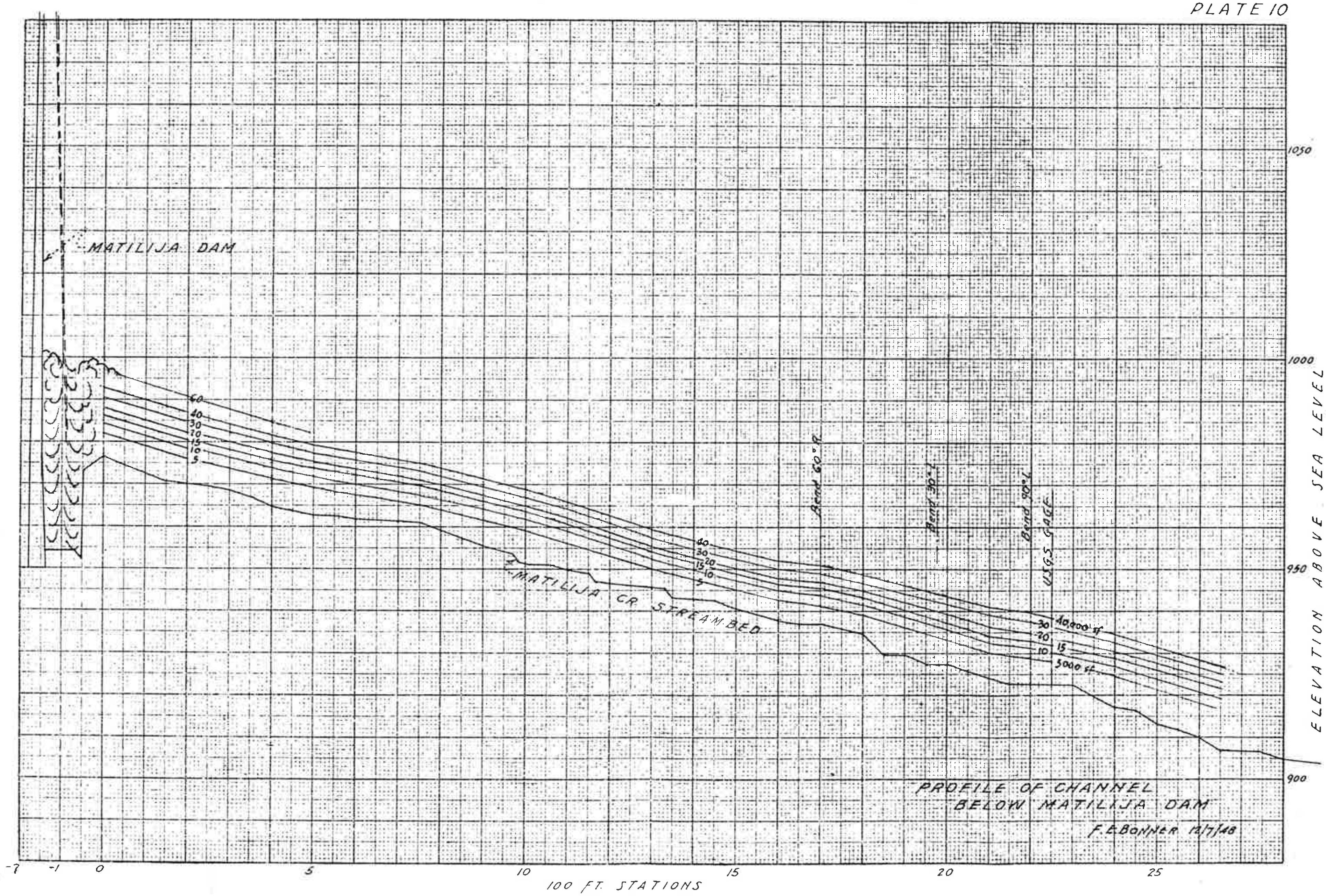
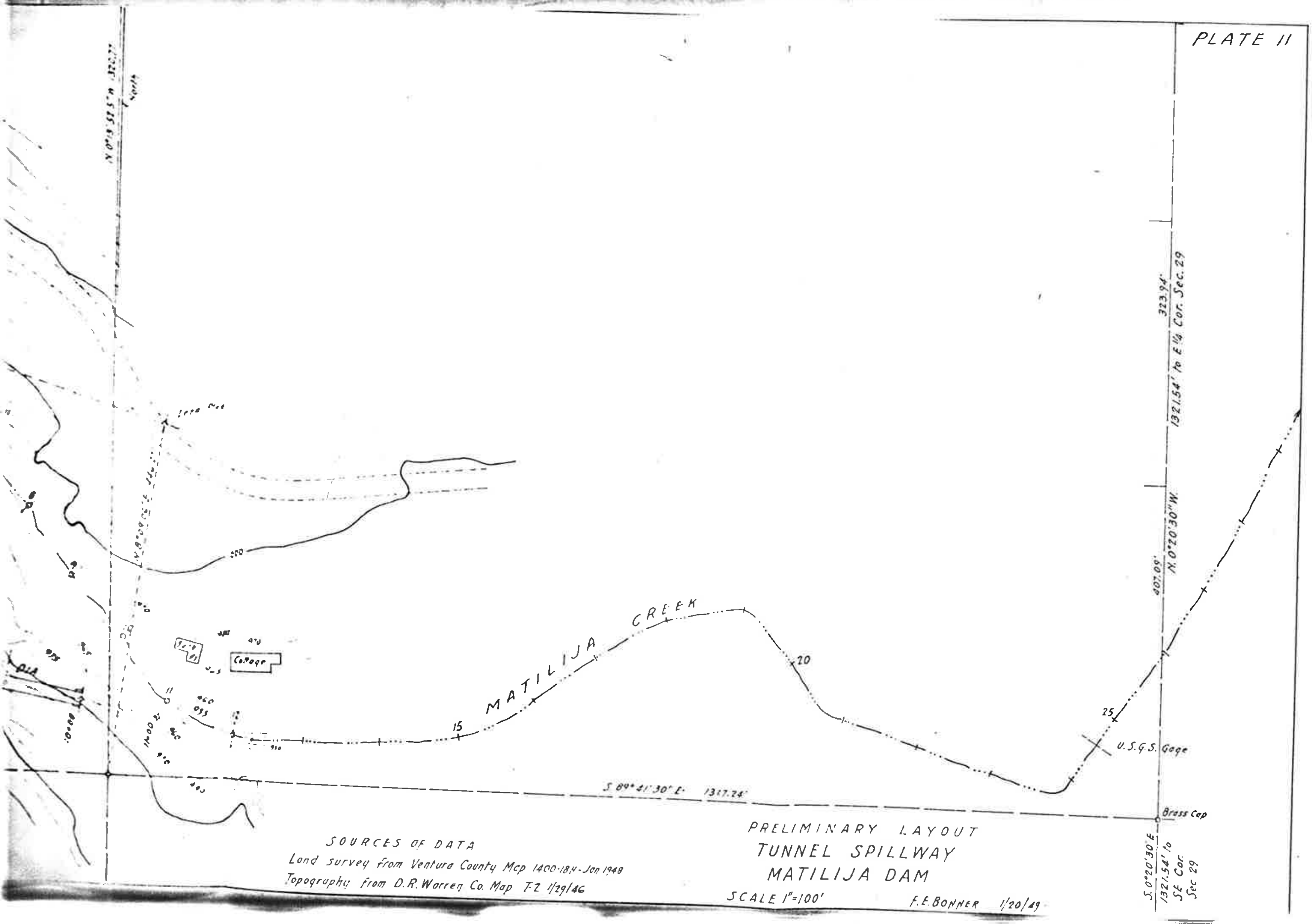

MAX. FLOOD EL. 1137
60,000 S.F.
Spill El. 1125
1130
1120
1110
1100
1090
1080
1070
1060
1050
1040
1030
1020
1010
1000
990
980
970
960
950
Invert El. 1025
36" Outlet @ Sta. 1+25
Invert El. 1000.8
48" Sluice @ Sta. 3+05
Joint
Conc. Apron
60,000 S.F.
L.W. El. 976.2
Streambed
PROFILE OF MATILIJA DAM SPILL
F. F. BONNER
12/7/48
Dam Sec. @ Sta. 4+03
-1+50
-1+00
-50
0+00
0+50
1+00

PLATE 10



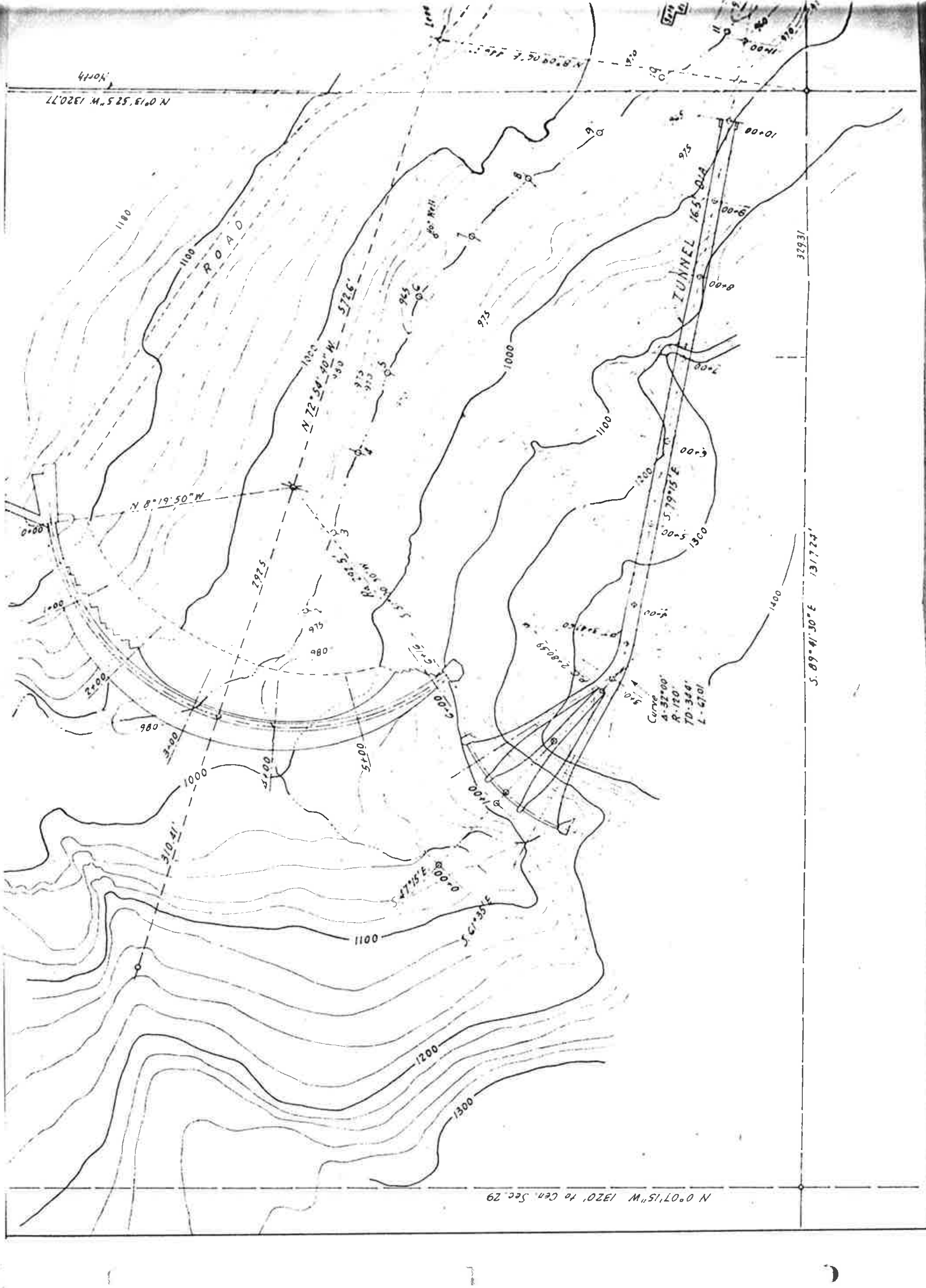
MATILIJA DAM
60
40
30
20
15
10
5
MATILIJA CR. STREAMBED
Bend 60° R.
Bend 30° L.
Bend 90° L.
USGS GAGE
40,000 sf
30
20
15
10
5000 sf
1050
1000
950
900
ELEVATION ABOVE SEA LEVEL
-2
-1
0
5
10
15
20
25
100 FT. STATIONS
PROFILE OF CHANNEL
BELOW MATILIJA DAM
F.E.BONNER 12/7/48



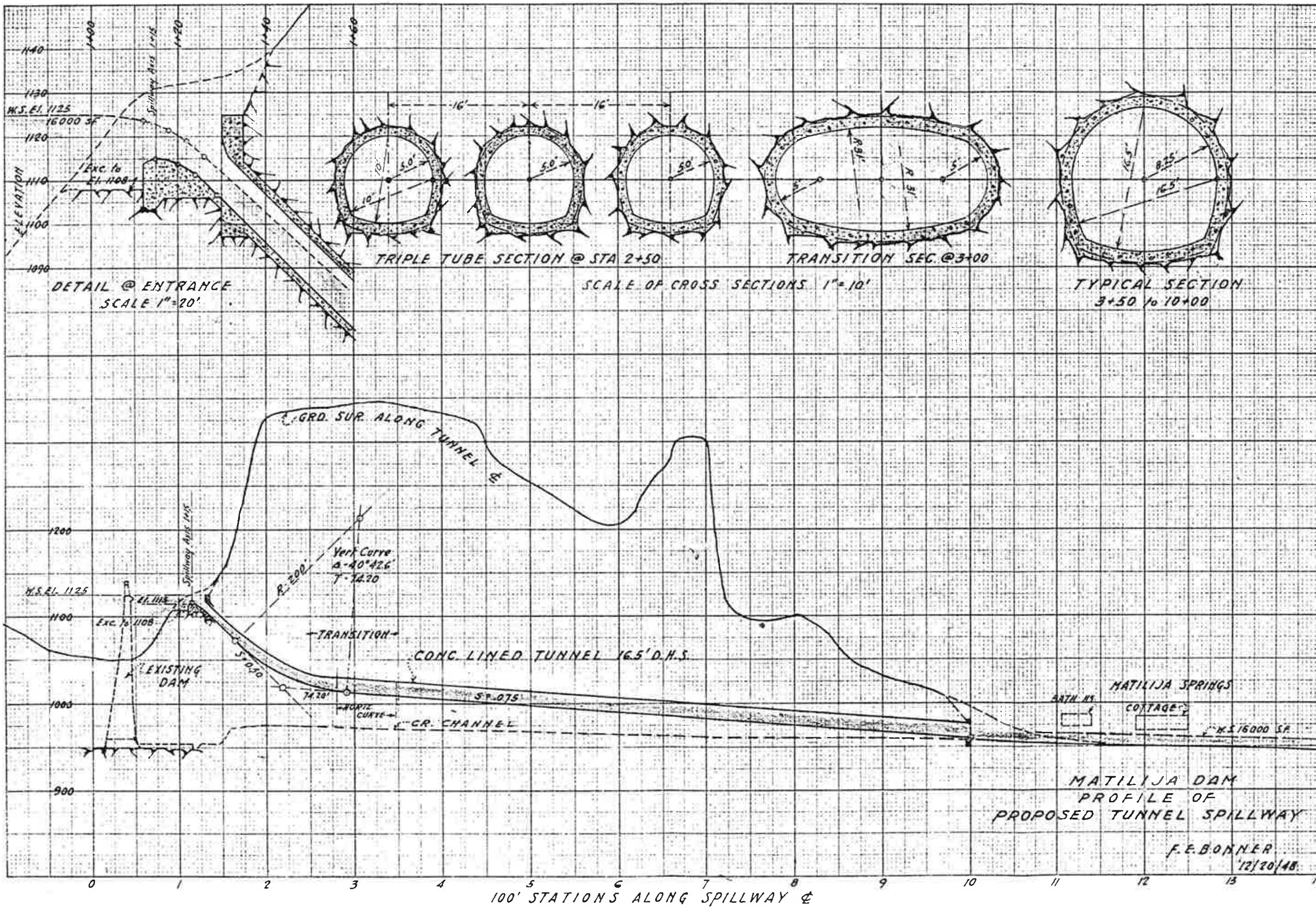
SOURCES OF DATA
Land survey from Ventura County Map 1400-18H-Jan 1948
Topography from D.R.Warren Co. Map T-2 1/29/46

PRELIMINARY LAYOUT
TUNNEL SPILLWAY
MATILIJA DAM
SCALE 1"=100'

F.E.BONNER 1/20/49



ROAD
TUNNEL 16.5' Dia
N 72°54'40" W 5726'
N 8°19'50" W
S 79°15' E
S 47°15' E
S 61°35' E
S 89°41'30" E 1317.27'
N 0°07'15" W 1320' to Cen. Sec. 29
N 0°13'25" W 1320.77
North
Curve
Δ 32°00'
R 120'
TD 34.4'
L 67.01
3293.1'
2925
3104.1'
1000
1100
1200
1300
1400
975
980


DETAIL @ ENTRANCE
SCALE 1" = 20'
TRIPLE TUBE SECTION @ STA 2+50
TRANSITION SEC. @ 3+00
SCALE OF CROSS SECTIONS 1" = 10'
TYPICAL SECTION
3+50 to 10+00
W.S. El. 1125
16000 SF
Exc. to El. 1108
Spillway Axis 1415
ELEVATION
GRD. SUR. ALONG TUNNEL ℄
Vert. Curve
R = 200'
TRANSITION
CONC. LINED TUNNEL 16.5' D.H.S.
EXISTING DAM
S = 0.50
S = .075
HORIZ. CURVE
CR. CHANNEL
MATILIJA SPRINGS
BATH HS.
COTTAGE
W.S. 16000 SF
100' STATIONS ALONG SPILLWAY ℄
MATILIJA DAM
PROFILE OF
PROPOSED TUNNEL SPILLWAY
F. E. BONNER
12/20/48