REQUEST FOR PROPOSALS (RFP)

2024
Probable Maximum Flood Study

November 2024

Calaveras Public Utility District
506 W St Charles Street
San Andreas, CA 95249
Phone (209) 754-9442

Proposals Due By:

November 7, 2024
by 3:00 p.m.

Introduction:

Calaveras Public Utility District (CPUD) is seeking Proposals from qualified consultants to conduct a Probable Maximum Flood Study (PMF Study) for the Middle Fork Dam. The intent of the Study is developing a probable maximum flood consistent with the current Federal Energy Regulatory Commission (FERC) and Division of Safety of Dams (DSOD) guidelines. The PMF study will be documented in report format and reviewed by CPUD and the appropriate regulatory agencies. The selected consultant shall provide a full range of services to develop the Study that addresses the Scope of Work described below.

Background:

CPUD (San Andreas) is located approximately 60 miles southeast of Sacramento on Highway 49. The district was established on January 19, 1934, as a publicly owned utility that provides domestic and irrigation water services to the residential communities of Railroad Flat, Glencoe, Paloma, Mokelumne Hill, and San Andreas, California. CPUD has a service area population of roughly 6,350 people, as well as its commercial businesses.

The Project is in Calaveras County on the Middle Fork Mokelumne River, approximately five miles east of West Point, California. The Project is owned and operated by CPUD and is classified as high hazard. The Project structures include from left to right (looking downstream): a 400-ft-long main earth embankment, a 500-ft-long earth dike, and a reinforced concrete chute spillway. Within the embankment there is a 12-inch diameter HLO pipe (near the left abutment) and a 24-inch diameter main LLO pipe (near the maximum section of main dam). Between the embankment and the chute spillway is a 30-inch diameter siphon steel penstock that leads to a powerhouse containing three turbines powering two generators. The reservoir, known as Schaad's Reservoir, has a capacity of 1,650 acre-feet at elevation 3,020 ft. The Project was constructed between 1939 and 1940 for the purpose of water storage. In the winter of 1940, the original concrete chute spillway failed, and the present concrete chute spillway was then constructed in 1941. The hydropower facility was added in the mid-1980's, and the Project underwent a major improvement in 1989 which involved construction of a downstream buttress on the embankment, construction of an upstream and downstream buttress on the dike, raising the embankment and dike crest elevation by four feet, installing groin drains and a blanket drain between the previous and modified toes of the embankment, and adding training walls to the chute spillway.

Middle Fork Dam was originally constructed in the late 1930's to an initial height of about 90 ft. The dam was subsequently raised in 1940, 1975, and 1989 to its current height of 112 ft to a design crest elevation of 3,040 ft. The embankment crest width is 16-ft width. The upstream face is sloped approximately 3 horizontals (H) to 1 vertical (V) whereas the downstream face is sloped approximately 2.5H to 1V. Both the upstream and downstream faces are covered with grass for erosion protection.

Request for Proposal:

This Request for Proposals (RFP) is to update the Probable Maximum Flood (PMF) study for Middle Fork Dam based on *Hydrometeorological Reports No. 58 and 59*, the National Oceanic and Atmospheric Administration's (NOAA) most current analysis guidance. A technical report will be required following Federal Energy Regulatory Commission (FERC) and Division of Safety of Dams (DSOD) Guidelines that describes the data, methodology, findings, and recommendations of the PMF study for Middle Fork Dam (DSOD Dam No. 82-02 and FERC Project 7506-CA) in Calaveras County, California.

Scoping assumptions are outlined below:

1) Develop a Probable Maximum Flood (PMF) consistent with the current Federal Energy Regulatory Commission (FERC) and Division of Safety of Dams (DSOD) Guidelines.

2) The PMF rainfall runoff transformation will be estimated using the latest U.S. Army Corps of Engineers Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software.

3) PMF Hydrographs will be developed based on a HEC-HMS model calibrated to historic flood data. However, if historical data is found to be inadequate to estimate credible parameters, synthetic runoff parameters will be developed, as recommended in the FERC and DSOD Guidelines.

4) The Probable Maximum Precipitation (PMP) will be developed based on NOAA's HMR 58/59. There are several types of PMP outlined in HMR 58/59, including all-season PMP, monthly general-storm PMP, and local-storm PMP. These relate to seasonal or monthly weather variations and other watershed conditions to produce related PMFs. The objective of the present study is to determine the PMF that produces the highest peak reservoir water surface elevation for the reservoir. The scope is limited to determining the controlling maximum stage PMF for the reservoir.

5) The PMF will be routed through the reservoir, starting from the maximum normal storage elevation, using the reservoir storage-elevation relationship and outflow rating curve. The outflow rating curve should be reviewed and confirmed for accuracy using best practices outlined in USBR's Design of Small Dams. The outflow rating curve should include embankment over topping flows if reservoir stage exceeds dam crest during the PMF. This includes calculating spillway rating curve.

6) Development of more frequent recurrence (10-year, 50-year, 100-year, 200-year, and 500-year) 3-day storm events can optionally be developed at CPUD's discretion. NOAA Atlas 14 precipitation frequency estimates should be used to develop these more frequent recurrence events. Please include optional subtasks for this analysis.

7) Modeled sub-basins will be delineated to increase hydrometeorological precision and credibility. The number of sub-basins, however, will be realistically limited to prevent unwarranted increases in calculation and modeling complexity, and to facilitate model calibration and verification using available empirical data.

8) The PMF study will be documented in report format and reviewed by CPUD and appropriate regulatory agencies.

The scoping assumptions listed the types of services that the consultant will need to perform; however, if additional services are needed, the consultant needs to indicate them in the proposal.

All inquiries regarding the proposal should be directed to Travis Small, General Manager, by telephone at (209) 754-9442, or preferably by email at info@cpud.org.

Attachments:

The following is a list of attachments included with this RFP.

1. Supporting Technical Information Document
2. Detailed Spillway Assessment
3. Standard Professional Agreement

Interpretations and Addenda

No interpretation made to any respondent as to the meaning of the RFP shall be binding on the District unless repeated in writing and distributed as an addendum by CPUD. Interpretations and/or clarification shall be requested in writing. Questions regarding the RFP can be submitted to info@cpud.org with a deadline of 10/10/2024 at 3pm. The questions and answers to the questions will be posted on the District's website by the close of business on 10/17/2024.

Proposal Format and Content:

The proposal shall be brief, precise, in an acceptable font format, and shall not include unnecessary promotional material. The proposal shall not exceed 25 single sided pages, excluding resumes. The proposal should contain the following elements in the exact order and segmentation listed below:

1. *Cover Letter* – Describe your firm or team's interest and commitment in providing Consultant Services to the District. The letter shall be signed by a person authorized to negotiate a contract with the District.

2. S*taffing, Team Experience and Understanding of Project & Objectives* - Describe the qualifications and experience of the team members expected to be assigned to this project. The description shall include previous experience with similar projects. Include an organization chart and provide a matrix including which projects team members have worked together in the past. A discussion demonstrating the proposer's understanding of the project, the goals, the services to be provided, and their significance to the overall District goals.

3. *Work Plan Approach and Schedule* – Discuss your firm's understanding of the scope of work to be performed and the level of effort expected to be performed by each resource. Include an itemized table of estimated person hours by professional classification (or team member) to quantify the level of effort. Describe the method that will be used for scheduling, coordination, management of overall project costs, quality assurance/quality control, and list key or potential issues/risk you may deem critical to this project.

4. *Resumes* – Include single page resumes of the key personnel and sub-consultants (if any) to be assigned to the project. It is expected that designated key staff will remain for the duration of the project.
5. *References* – Provide at least three references (name, agency, title, address and telephone number) for recent similar or related work.

6. *Other Relevant Information & Exceptions* – Provide additional relevant information that may be helpful in the selection process including any exceptions taken to the District's standard agreement.

7. *Proposal Cost* – In a separate document, please include the total and the line-item cost estimates for the proposal.

Evaluation and Selection Process:

Qualifications will be screened by staff and evaluated by a committee. The qualifications for the top candidates will be verified and references will be checked. The top candidates may be invited to meet with staff and the board of directors to present the proposals, the District's committee will carefully weigh: (100 Points)

- Consultant's understanding of the District's desires and general approach to completing the work – 30 POINTS
- Consultant's experience with contracts of similar complexity and magnitude – 25 POINTS
- Demonstrated ability of the Consultant to perform high quality work, to control costs and to meet time schedules – 20 POINTS
- Ability to work effectively with District staff – 15 POINTS
- Total Proposal Price – 10 Points (Lowest Price 10 PTS, and 2nd Lowest Price 5 points) Proposal Price will not be reviewed until after the proposal is evaluated.
- In the event of a tie, the lower priced proposal will be the tie breaker.

Submittal Requirements:

Consultant to submit an electronic copy of their proposal via email to Calaveras Public Utility District at info@cpud.org **by** November 7, 2024, by 3:00 p.m.
Optional: Hard copy submissions will be accepted at the District Office at 506 W. St. Charles Street, San Andreas, CA 95249 (Please provide 5 copies for Evaluation Team). USPS Mail Delivery will not be accepted at this address.

Any changes made by the District to the requirements in this RFP will be made by written addenda. Any written addenda issued to this RFP shall be incorporated into the terms and conditions of any resulting Agreement. The District will not be bound by any modifications to or deviations from the requirements set forth in this RFP as the result of oral instructions. The District reserves the right to revise or withdraw this RFP at any time and for any reason.

Proposals received after the above date and time will be considered late and will not be accepted. Any late proposals will be returned unopened to the firm. Responses will be evaluated objectively based on the firm's responses to the RFP.

The District will not pay costs incurred in the proposal preparation including the costs for printing, mailing, etc. All costs for the preparation of the proposal shall be borne by the proposing firm.

Right to Reject Proposals:

The District reserves the right to reject any and all proposals or any part of any proposals, to waive minor defects or technicalities, or to solicit new proposals on the same project or on a modified project that may include portions of the originally proposed project as the District may deem necessary in its best interest. The District also reserves the right to negotiate with any firm, all or part of any proposal that is in the best interest of the District.

Project Schedule: (Subject to Change)

Issue Request for Proposal	9/12/2024
RFP Questions Due	10/10/2024
Answers to Questions Due	10/17/2024
Receive Proposals by	11/7/2024
Review Proposals	11/12/2024
Select Consultant (Board Meeting)	11/19/2024
Notice to Proceed	12/1/2024

Goal: Complete PMF Study by 6/30/2025

Award of Contract:

A cost proposal will be requested from the selected consultant.

The contract will be between CPUD Board of Directors and a consultant. The District General Manager will be responsible for, and will be the sole point of contact for, all contractual matters.

Services for the PMF study shall be professional services not subject to Labor Compliance requirements such as prevailing wage, apprentices, and payroll submittal.

The final contract including Scope of Services will be negotiated. If contract negotiations with the first selected firm are unsuccessful, CPUD will begin negotiations with the second selected firm, and so on.

ATTACHMENT 1

Supporting Technical Information Document

Section 6.0 Hydrology and Appendix H

5.2.3 Site MCE

Kleinfelder concluded that the Maximum Credible Earthquake that could be expected at the site was a Richter magnitude 6.5 occurring on the Melones Fault approximately 17 miles west of the site. The associated bedrock acceleration was estimated to be 0.19g. An independent review of the Kleinfelder report was made by Wallace-Van Alstine Geotechnical Engineering in 1985. Wallace-Van Alstine concurred with Kleinfelder's faulting and seismicity evaluation.[7]

5.2.4 Time history of adopted earthquakes

No information is available.

5.2.5 Response spectrum

A discussion of the response spectrum used for an embankment deformation analysis is provided in letter from James C. Hanson to FERC dated June 1, 1989, a copy of which is provided in Appendix G.

5.2.6 Historic earthquake centers map

The epicenter map from the 1984 DSOD Safety Review Report is provided in Appendix F.

6.0 HYDROLOGY AND HYDRAULICS

6.1 Hydrology

Hydrologic analyses performed by Norman S. Braithwaite, Inc., in 1989 confirmed that the PMF was the appropriate design flood for Middle Fork Dam.[21, 22] A design flood hydrologic analysis of the Middle Fork Dam watershed was performed by James C. Hanson, Consulting Civil Engineer in connection with the Middle Fork Dam Improvement Project of 1989. A hydrologic model of the watershed was developed using the U.S. Army Corps of Engineers' HEC-1 computer program.

6.1.1 Hydrometeorology report used

The Probable Maximum Precipitation was developed using the method described in the National Weather Service's "Hydrometeorological Report No. 36 - Probable Maximum Precipitation in California." The PMF analysis included consideration of snow melt.

6.1.2 Probable Maximum Precipitation (PMP)

The 72-hour PMP was determined to be 31.67 inches

6.1.3 Drainage basin conditions

Basin area = 28.48 miles. Basin terrain is mountainous with elevation ranging from about Elev. 3,000 at the dam site to about Elev. 7,200 at the highest elevations. Ground cover is generally heavy brush and dense forest.

6.1.4 Antecedent conditions

The inflow design flood was based on a 72-hour PMF. In addition, a snow melt allowance was included in the analysis.

6.1.5 Loss rates

The following loss rate parameters were derived from a previous evaluation of the entire Mokelumne River watershed by East Bay Municipal Utility District (EBMUD), FERC Project No. 2916, using the HEC exponential loss rate function in HEC 1:

Initial loss rate (STRKR) = 0.34 in/hr
Initial loss during which loss coefficient is increased (DLKTR) = 0.95 in
Rate of change of loss-rate (RTIOL) = 2.21
Exponent of loss rate function (ERAIN) = 0.51
Percent of basin impervious (RTIMP) = 5 percent

6.1.6 Basin precipitation/runoff model

The HEC 1 analysis was run using a 60-minute time step. Precipitation input was based on hourly increments obtained from the PMP evaluation. Snowmelt was modeled based on the EBMUD Mokelumne River evaluation using the following HEC 1 snowmelt parameters:

Initial snow melt loss rate (STRKS) = 0.48 in/hr
Rate of change of snow melt loss-rate (RTIOK) = 2.23

The watershed was modeled as single basin and inflow was routed through the reservoir based on the rating curve developed for the spillway improvements made in 1989 (see Section 6.2.1).

6.1.7 Unit hydrograph

The following unit hydrograph parameters were derived from the EBMUD evaluation:

Clark's TC = 6.35 hours
Clark's Storage Coefficient = 6.35

6.1.8 Reservoir inflow and outflow hydrographs for PMF event

Output for the HEC1 PMF analysis showing the input file and output data and hydrographs is provided in Appendix H. As shown the peak inflow is computed to be about 13,300 cfs, and the routed peak outflow is about 13;200 cfs.

6.1.9 Floods of record

There are no measuring devices at Middle Fork Dam to measure flows, therefore, no data is available concerning specific historical maximum flows at the dam. The closest gaging station is located on the Middle Fork of the Mokelumne River about 5 miles downstream of Middle Fork Dam (USGS #11317000 Middle Fork Mokelumne River At West Point). The period of record for this gaging station is Water Years 1911 to 2004.[20] The maximum discharge for this gaging station was recorded to be 5,040 cfs on January 2, 1997. Based on a simple drainage area ratio, the corresponding maximum historical flow at Middle Fork Dam is estimated to have been about 3,000 cfs.

6.2 Hydraulics

6.2.1 Discharge rating curves

Discharge of storm flows is made by way of the ungated spillway chute on the right dam abutment. The spillway rating curve used for hydrologic studies, is provided as Appendix I.

6.2.2 Tailwater rating curve

Not applicable.

6.2.3 Normal and IDF freeboard

Normal freeboard is 20 feet. The minimum residual freeboard under PMF conditions was determined to be 0.2 feet.

6.2.4 Zero freeboard flood capacity

This parameter is estimated to be about 13,500 cfs.

6.2.5 Inflow Design Flood (based on dam break)

See Section 6.1 above.

6.2.6 Reservoir Probable Maximum and Inflow Design Flood outflow hydrograph and corresponding reservoir levels

The HEC1 input file and output data and hydrographs for the PMF analysis for this project are provided in Appendix H.

6.2.7 Freeboard for general and thunderstorm events

Total freeboard is 20 feet. Thunderstorm events have not been evaluated.

6.2.8 Stilling basin or plunge pool design flood flow

No studies have been prepared regarding the stilling basin at the spillway outlet.

6.2.9 Operating rule curve

Not applicable. There are no operating rules or restrictions for reservoir storage levels.

6.3 Hydraulics – Water Conveyance Systems

Not applicable. There are no water conveyance systems at this project apart from the releases facilities described in Section 2.0 above.

7.0 SURVEILLANCE AND MONITORING PLAN

7.1 Plans, Sections and Details of Active Instrumentation

APPENDIX H

Output for HEC 1 Analysis for PMF

```
TYPE MPM.OUT
1
                                   ****
             FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
 U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
                                   ****


THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3

1                                  HEC-1 INPUT                                            PAGE  1

 LINE       ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

    1       ID     MIDDLE FORK MOKELUMNE RIVER HYDROLOGY, PROBABLE MAXIMUM FLOOD STUDY
    2       IT      60  1JAN90     000      72
    3       IO       3       2

    4       KK   MIDFK
    5       BA   28.48
    6       BF      50     200     1.0
    7       LE     .34     .95    2.21     .51     .05
    8       LM     .48    2.23
    9       UC    6.35    6.35
   10       PB
   11       PI     .29     .29     .29     .29     .29     .29     .38     .38     .38     .38
   12       PI     .38     .38     .43     .43     .43     .43     .43     .43     .33     .33
   13       PI     .33     .33     .33     .33     .50     .50     .50     .50     .50     .50
   14       PI     .73     .73     .73     .73     .73     .73     .82     .83    1.16    2.23
   15       PI    1.17     .83     .59     .59     .59     .59     .59     .59     .19     .19
   16       PI     .19     .19     .19     .19     .23     .23     .23     .23     .23     .23
   17       PI     .26     .26     .26     .26     .26     .26     .21     .21     .21     .21
   18       PI     .21     .21
   19       MA     .18       0      40
   20       MA    5.31       3      45
   21       MA   11.12      13      50
   22       MA    8.46      24      57
   23       MA    3.07      34      61
   24       MA     .34      45      64
   25       MC     2.9     .07      32
   26       IN     360  1JAN90     000
   27       MT    47.2    47.8    48.2    47.6    50.4    51.6    52.4    51.0    48.6    49.4
   28       MT    49.8    49.0    49.4

   29       KK  SCHAAD
   30       RS      .1    ELEV    3020
   31       SV    1700    1780    1880    1970    2060    2150    2235    2325    2425    2525
   32       SQ       0     250     675    1500    2450    3600    5000    6750    8750   13450
   33       SE    3020    3022    3024    3026    3028    3030    3032    3034    3036    3040
   34       ZZ
1                                  ****
             FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
 U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
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  3 IO         OUTPUT CONTROL VARIABLES
                     IPRNT          3  PRINT CONTROL
                     IPLOT          2  PLOT CONTROL
                     QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT          HYDROGRAPH TIME DATA
                      NMIN         60  MINUTES IN COMPUTATION INTERVAL
                     IDATE     1JAN90  STARTING DATE
                     ITIME       0000  STARTING TIME
                        NQ         72  NUMBER OF HYDROGRAPH ORDINATES
                    NDDATE     3JAN90  ENDING DATE
                    NDTIME       2300  ENDING TIME

                COMPUTATION INTERVAL    1.00 HOURS
                     TOTAL TIME BASE   71.00 HOURS

       ENGLISH UNITS


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

          **************
          *            *
  4 KK    *    MIDFK   *
          *            *
          **************

 26 IN         TIME DATA FOR INPUT TIME SERIES
                     JXMIN        360  TIME INTERVAL IN MINUTES
                    JXDATE     1JAN90  STARTING DATE
                    JXTIME          0  STARTING TIME

            SUBBASIN RUNOFF DATA

  5 BA         SUBBASIN CHARACTERISTICS
                     TAREA      28.48  SUBBASIN AREA

  6 BF         BASE FLOW CHARACTERISTICS
                     STRTQ      50.00  INITIAL FLOW
                     QRCSN     200.00  BEGIN BASE FLOW RECESSION
                     RTIOR    1.00000  RECESSION CONSTANT

               PRECIPITATION DATA

 10 PB               STORM      31.67  BASIN TOTAL PRECIPITATION

 11 PI          INCREMENTAL PRECIPITATION PATTERN
                     .29       .29       .29       .29       .29       .29       .38       .38       .38       .38
                     .38       .38       .43       .43       .43       .43       .43       .43       .33       .33
                     .33       .33       .33       .33       .50       .50       .50       .50       .50       .50
                     .73       .73       .73       .73       .73       .73       .82       .83      1.16      2.23
                    1.17       .83       .59       .59       .59       .59       .59       .59       .19       .19
                     .19       .19       .19       .19       .23       .23       .23       .23       .23       .23
                     .26       .26       .26       .26       .26       .26       .21       .21       .21       .21
                     .21

   MC          SNOWMELT DATA
                     TLAPS       2.90  TEMPERATURE LAPSE RATE
                      COEF        .07  SNOWMELT COEFFICIENT
                     FRZTP      32.00  MELT TEMPERATURE
```

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 MA          ELEVATION ZONE DATA
                ZONE   AREA   SNOWPACK   ANNUAL PRECIP
                  1      0.       .00       40.00
                  2      5.      3.00       45.00
                  3     11.     13.00       50.00
                  4      8.     24.00       57.00
                  5      3.     34.00       61.00
                  6      0.     45.00       64.00

 MT          TEMPERATURE DATA
               47.2      47.3      47.4      47.5      47.6      47.7      47.8      47.9      47.9      48.0
               48.1      48.1      48.2      48.1      48.0      47.9      47.8      47.7      47.6      48.1
               48.5      49.0      49.5      49.9      50.4      50.6      50.8      51.0      51.2      51.4
               51.6      51.7      51.9      52.0      52.1      52.3      52.4      52.2      51.9      51.7
               51.5      51.2      51.0      50.6      50.2      49.8      49.4      49.0      48.6      48.7
               48.9      49.0      49.1      49.3      49.4      49.5      49.5      49.6      49.7      49.7
               49.8      49.7      49.5      49.4      49.3      49.1      49.0      49.1      49.1      49.2
               49.3      49.3

7 LE        EXPONENTIAL LOSS RATE
               STRKR          .34  INITIAL VALUE OF LOSS COEFFICIENT
               DLTKR          .95  INITIAL LOSS
               RTIOL         2.21  LOSS COEFFICIENT RECESSION CONSTANT
               ERAIN          .51  EXPONENT OF PRECIPITATION
               RTIMP          .05  PERCENT IMPERVIOUS AREA

 LM         MELTWATER LOSS RATE
               STRKS          .48  INITIAL VALUE OF LOSS COEFFICIENT
               RTIOK         2.23  LOSS COEFFICIENT RECESSION CONSTANT

9 UC        CLARK UNITGRAPH
                  TC         6.35  TIME OF CONCENTRATION
                  R          6.35  STORAGE COEFFICIENT

            SYNTHETIC ACCUMULATED-AREA VS. TIME CURVE WILL BE USED

                                                       ***

           WEIGHTED AVERAGE ANNUAL PRECIPITATION FOR BASIN IS   52.44

                                          UNIT HYDROGRAPH PARAMETERS
                                     CLARK   TC  6.35 HR        R  6.35 HR
                                    SNYDER   TP  5.76 HR       CP   .57

                                               UNIT HYDROGRAPH
                                          37 END-OF-PERIOD ORDINATES
           119.      437.      870.     1322.     1669.     1828.     1747.     1516.     1295.     1106.
           945.      807.      689.      588.      502.      429.      366.      313.      267.      228.
           195.      166.      142.      121.      104.       89.       76.       65.       55.       47.
            40.       34.       29.       25.       21.       18.       16.

     ***            ***            ***            ***            ***

                   HYDROGRAPH AT STATION    MIDFK

  PEAK FLOW     TIME                       MAXIMUM AVERAGE FLOW
                                 6-HR        24-HR        72-HR      71.00-HR
+   (CFS)       (HR)
                        (CFS)
+   13281.      45.00           12625.        9273.        4937.        4937.
                      (INCHES)  4.121        12.109       19.071       19.071
                       (AC-FT)  6260.        18393.       28967.       28967.

                       CUMULATIVE AREA =   28.48 SQ MI
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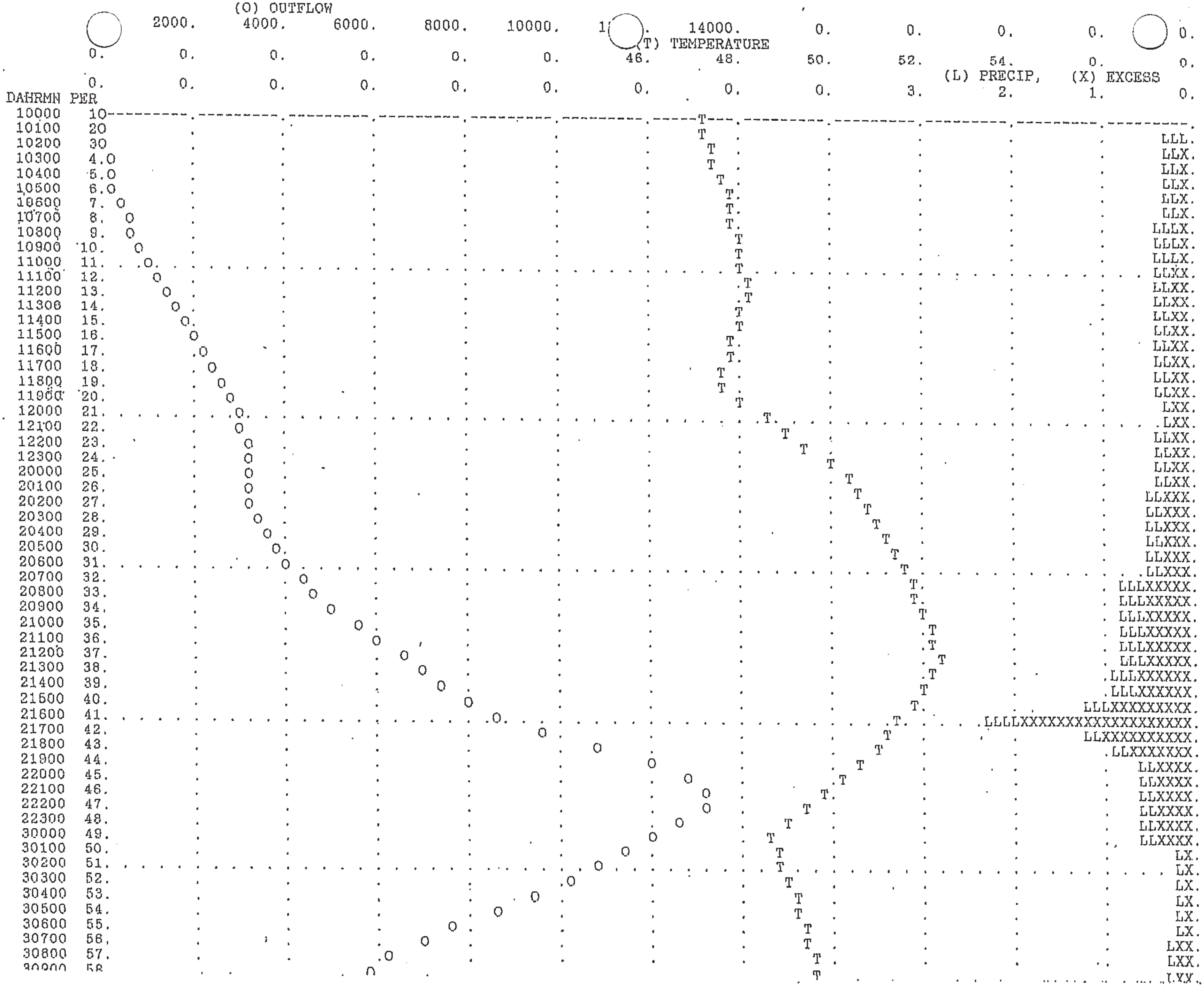
(O) OUTFLOW
(T) TEMPERATURE
(L) PRECIP,
(X) EXCESS
DAHRMN PER
2000.
4000.
6000.
8000.
10000.
14000.
46.
48.
50.
52.
54.
3.
2.
1.
10000 10
10100 20
10200 30
10300 4.
10400 5.
10500 6.
10600 7.
10700 8.
10800 9.
10900 10.
11000 11.
11100 12.
11200 13.
11300 14.
11400 15.
11500 16.
11600 17.
11700 18.
11800 19.
11900 20.
12000 21.
12100 22.
12200 23.
12300 24.
20000 25.
20100 26.
20200 27.
20300 28.
20400 29.
20500 30.
20600 31.
20700 32.
20800 33.
20900 34.
21000 35.
21100 36.
21200 37.
21300 38.
21400 39.
21500 40.
21600 41.
21700 42.
21800 43.
21900 44.
22000 45.
22100 46.
22200 47.
22300 48.
30000 49.
30100 50.
30200 51.
30300 52.
30400 53.
30500 54.
30600 55.
30700 56.
30800 57.

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 31000  59  .         .    O   .          .         .         .         .    T .        .         .         .  XX.
 31100  60  .         .   O    .          .         .         .         .     T.        .         .         .  XX.
 31200  61. . . . . . . . .O. . . . . . . . . . . . . . . . . . . . . . . . .T. . . . . . . . . . . . . . . .LXX.
 31300  62.           .  O     .          .         .         .         .    T .        .         .         . LXX.
 31400  63.           . O      .          .         .         .         .    T .        .         .         . LXX.
 31500  64.           . O      .          .         .         .         .   T  .        .         .         . LXX.
 31600  65.           . O      .          .         .         .         .  T   .        .         .         . LXX.
 31700  66.           .O.      .          .         .         .         .  T   .        .         .         . LXX.
 31800  67.           .O.      .          .         .         .         . T    .        .         .         . LXX.
 31900  68.           .O.      .          .         .         .         . T    .        .         .         .  LX.
 32000  69.           O .      .          .         .         .         .  T   .        .         .         .  LX.
 32100  70.           O .      .          .         .         .         .  T   .        .         .         .  XX.
 32200  71. . . . . . .O . . . . . . . . . . . . . . . . . . . . . . . . .  T . . . . . . . . . . . . . . . . XX.
 32300  72.---------.------O--.---------.---------.---------.---------.---------.-----T--.---------.---------.---------.------XX.
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          **************
          *            *
 29 KK    *   SCHAAD   *
          *            *
          **************

            HYDROGRAPH ROUTING DATA

 30 RS        STORAGE ROUTING
                 NSTPS          1  NUMBER OF SUBREACHES
                  ITYP       ELEV  TYPE OF INITIAL CONDITION
                RSVRIC    3020.00  INITIAL CONDITION
                     X        .00  WORKING R AND D COEFFICIENT

 31 SV          STORAGE    1700.0    1780.0    1880.0    1970.0    2060.0    2150.0    2235.0    2325.0    2425.0    2525.0

 32 SQ        DISCHARGE        0.      250.      675.     1500.     2450.     3600.     5000.     6750.     8750.    13450.

 33 SE        ELEVATION   3020.00   3022.00   3024.00   3026.00   3028.00   3030.00   3032.00   3034.00   3036.00   3040.00

                                                          ***

*** WARNING *** MODIFIED PULS ROUTING MAY BE NUMERICALLY UNSTABLE FOR OUTFLOWS BETWEEN      8750. TO    13450.
                THE ROUTED HYDROGRAPH SHOULD BE EXAMINED FOR OSCILLATIONS OR OUTFLOWS GREATER THAN PEAK INFLOWS.
                THIS CAN BE CORRECTED BY DECREASING THE TIME INTERVAL OR INCREASING STORAGE (USE A LONGER REACH.)

       ***           ***           ***           ***           ***

                      HYDROGRAPH AT STATION   SCHAAD

  PEAK FLOW      TIME                            MAXIMUM AVERAGE FLOW
                                         6-HR        24-HR       72-HR      71.00-HR
+   (CFS)        (HR)
                              (CFS)
+   13216.       46.00                12628.         9264.       4861.        4861.
                           (INCHES)    4.123        12.097      18.779       18.779
                            (AC-FT)    6262.        18375.      28524.       28524.

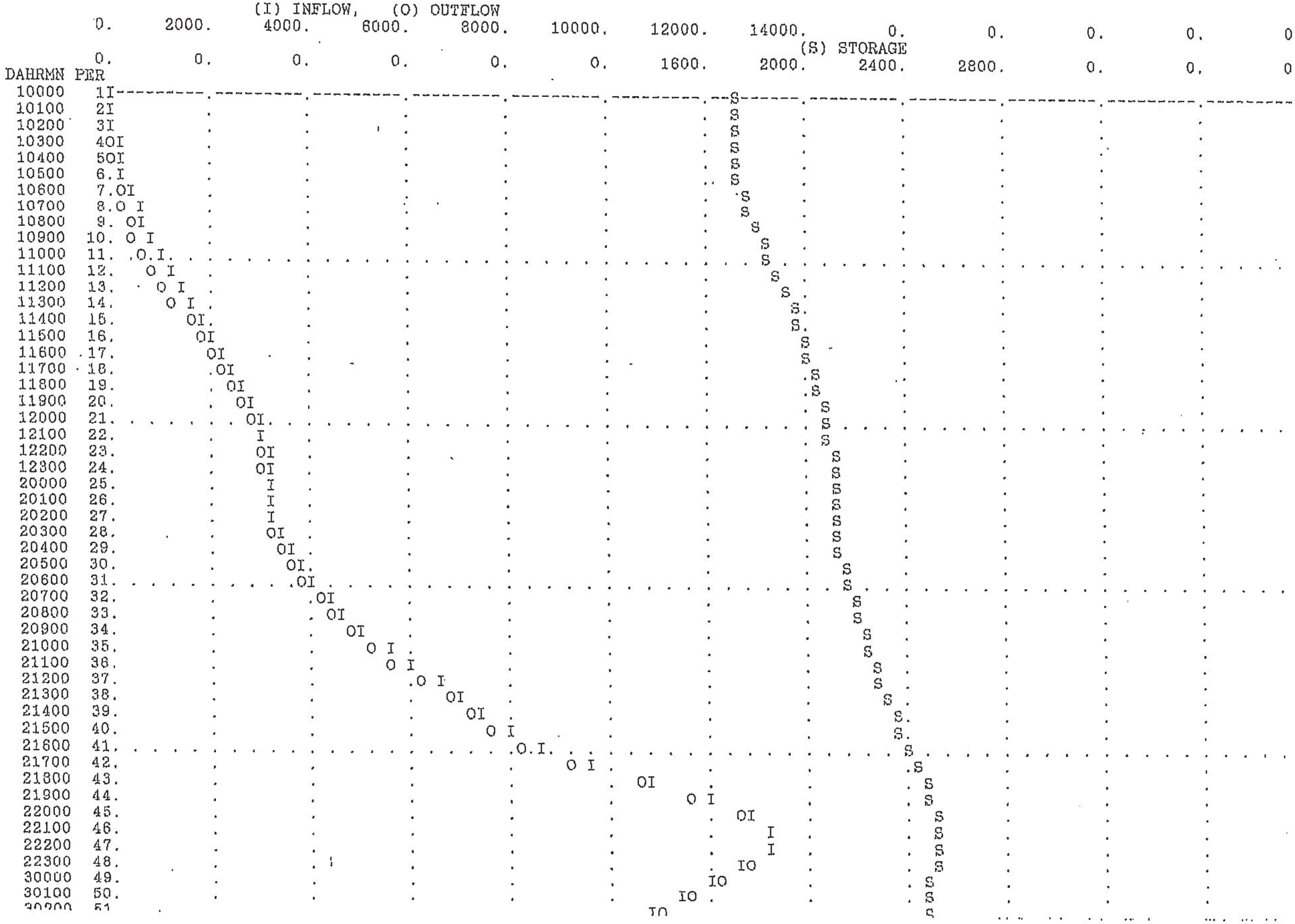
 PEAK STORAGE    TIME                           MAXIMUM AVERAGE STORAGE
                                         6-HR        24-HR       72-HR      71.00-HR
+  (AC-FT)       (HR)
```

PEAK STAGE	TIME		MAXIMUM AVERAGE STAGE		
(FEET)	(HR)	6-HR	24-HR	72-HR	71.00-HR
+ 3039.80	46.00	3039.30	3036.29	3030.49	3030.49

CUMULATIVE AREA = 28.48 SQ MI

1

STATION SCHAAD



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  30200  52.                                                  IO.                        S
  30300  53.                                                .IO                          .S
  30400  53.                                             IO .                             .S
  30500  54.                                          IO                                  .S
  30600  55.                                      I .O                                    S
  30700  56.                                   I O  .                                    S.
  30800  57.                               .I O                                         S .
  30900  58.                              IO                                           S  .
  31000  59.                           I O .                                           S  .
  31100  60.                          IO                                              S   .
  31200  61. . . . . . . . . . . . .IO . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . .
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  31400  63.                      .IO                                                S
  31500  64.                      IO                                                 S
  31600  65.                      I                                                 S
  31700  66.                     IO                                                 S
  31800  67.                     I.                                                 S
  31900  68.                     I.                                                 S
  32000  69.                    IO.                                                 S
  32100  70.                    I .                                                 S
  32200  71. . . . . . . . . . .I . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . .
  32300  72.---------.------IO-.---------.---------.---------.---------.---------.---S-----.---------.---------.---------.---------.
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RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT	MIDFK	13281.	45.00	12625.	9273.	4937.	28.48		
+	ROUTED TO	SCHAAD	13216.	46.00	12628.	9264.	4861.	28.48		
+									3039.80	46.00

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*** NORMAL END OF HEC-1 ***

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ATTACHMENT 2

Detailed Spillway Assessment

DETAILED SPILLWAY ASSESSMENT

Calaveras Public Utility District

Middle Fork Dam
FERC Project 7506
State Dam No. 82-2
NID No. CA83288

Report prepared by:
Mead & Hunt, Inc.
Warren Hayden

November 30, 2018

Table of Contents

Page

Figures

Appendices

1. Introduction

1.1 Background and Purpose

This report presents the findings of a focused engineering assessment of the Middle Fork Dam spillway, located in Calaveras County, California. In a letter dated May 1, 2017, the Federal Energy Regulatory Commission (FERC) requested that Calaveras Public Utility District (CPUD) perform a detailed assessment of the concrete chute spillway at the Middle Fork Dam.

CPUD retained Mead & Hunt, Inc. (Mead & Hunt) to perform a focused spillway assessment comprised of four components: data collection and desktop study; detailed field inspection; focused potential failure modes analysis (PFMA); and reporting.

The desktop study included a review of documents available from CPUD and the State of California Department of Water Resources Division of Safety of Dams (DSOD) that describe characteristics of the spillway such as: general foundation geology and composition; structural design; hydraulic capacity and flow characteristics; provisions for drainage beneath the chute; methods of construction; past modifications or repairs; operational procedures; surveillance and monitoring activities; and previous inspection findings.

The visual inspection addressed the condition of the spillway with an emphasis toward evidence of conditions associated with typical potential failure modes (PFMs) and the PFMA session focused on existing and newly identified PFMs specific to the spillway. The existing PFMs were reviewed, and new PFMs were developed, based on a review of documents available from CPUD, and supplemented by information highlighted from the desktop study and field inspection.

This report has been prepared by Mead & Hunt to document the findings of the assessment, including conclusions regarding the health and condition of the spillway and recommendations for further investigation, evaluation, and improvements, as necessary. The elements of the assessment, including desktop study and engineering review, field inspection, and focused PFMA session, conform to the requirements outlined in the FERC correspondence with CPUD.

This assessment report presents a project introduction and description of the spillway structure in Section 1, findings from the desktop study in Section 2, findings from the field inspection in Section 3, findings from the PFMA session in Section 4, and lastly in Section 5, conclusions and recommendations derived from an assessment of the information developed in Sections 2 through 4. Appendices A and B present key spillway design and construction documents, and notes from the spillway inspection, respectively.

1.2 Description of Spillway Structure

Middle Fork Dam and its impoundment known as Schaads reservoir, is located in Calaveras County on the Middle Fork Mokelumne River, approximately five miles east of West Point, California (**Figure 1-1**).

The dam has a fixed-crest concrete chute spillway located right of the embankment dam and dike embankment between the embankment and spillway (**Figure 1-2**).[1]

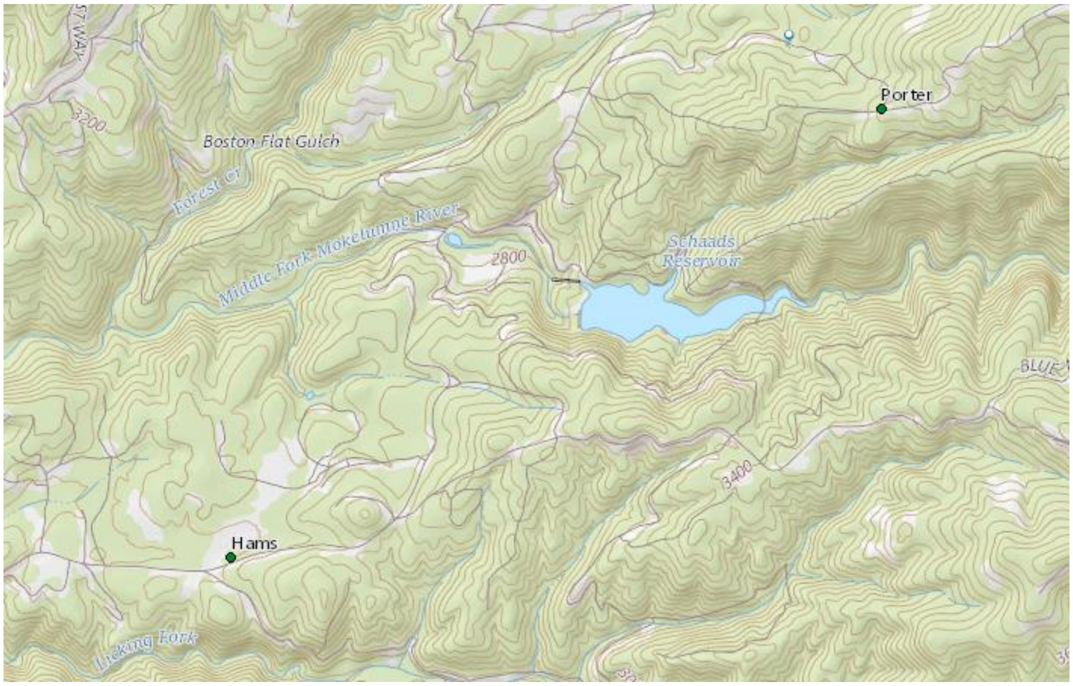

Figure 1-1: Middle Fork Dam location map.

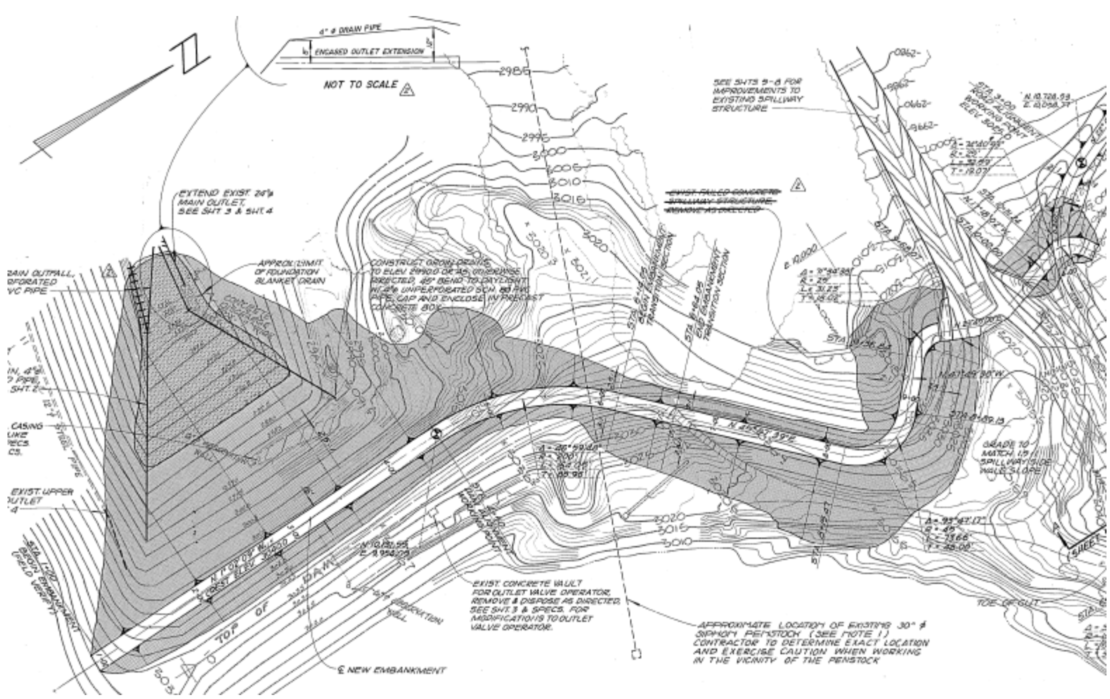

Figure 1-2: Plan of embankment dam, dike embankment, and chute spillway.

[1] The dam components are referred to in this report as left and right relative to an observer standing on the dam and facing downstream.

Mead&Hunt

The spillway structure is a fixed-crest chute of trapezoidal section with vertical training walls. The chute spillway is constructed of reinforced concrete, has a slope that varies from eighteen to thirty-three percent, and extends approximately 475 feet in length over a drop of approximately 107 feet. The crest elevation[2] is 3020.0 feet and the bottom width of the spillway converges from twenty-eight feet at the crest to eight feet in the chute. Details of the spillway are shown on drawings presented in Appendix A. The spillway was apparently founded on weathered, granitic bedrock. There is no stilling basin at the downstream end of the chute and flow discharges onto hard, jointed quartz diorite bedrock. A floating boom located in the approach to the spillway captures debris and prevents its passage down the spillway chute.

2. Desktop Study

2.1 Engineering Design Review

2.1.1 Geotechnical / Geology

Mead & Hunt completed a review of the relevant geotechnical/geologic data from the Middle Fork Dam Supporting Technical Information Document (STID) (Wagner & Bonsignore 2006), Section 5, Geology and Seismicity, and DSOD field reports.

Geology

1. ***Regional Geology***

   Middle Fork Dam is located near the metamorphic-granitic bedrock transition in the western Sierra Nevada geomorphic province. Local bedrock units are predominantly metamorphic rocks and granitic intrusive rocks. Ryolitic tuff of the Valley Springs formation (Tertiary age) overlies the granitic and metamorphic bedrock above the right abutment and downstream of Middle Fork Dam.

2. ***Site Geology***

   According to information in DSOD's 1984 Safety Review Report, the rebuilt spillway was founded on "weathered granitic bedrock." The spillway training walls added in 1989 were founded on firm residual soils, with foundation keys cut 3 feet into stiff in-place native material.

3. ***Groundwater Conditions***

   Several springs were encountered in 1941 during excavation for the lower third of the spillway channel. Open joint drain tiles enveloped in gravel were incorporated into the channel design to drain the groundwater and allow construction of this portion of the chute.

[2] All elevations in this report are given in feet and referenced to a local datum based on the chute spillway crest elevation as 3020.0.

Geologic Hazards

1. Faulting

Based on a review of faulting near the site performed by J.H. Kleinfelder & Associates (Kleinfelder) in 1985, the nearest fault is located within the Melones Fault Zone (part of the Foothills Fault system), located approximately 17 miles west of the site. Other faults identified as possibly having an impact on the seismicity of the site include the San Andreas, Hayward, Midland, Sierra Nevada, and Carson Valley.

2. Ground Motions

Kleinfelder concluded that the Maximum Credible Earthquake (MCE) that could be expected at the site was a Richter magnitude 6.5 occurring on the Melones Fault approximately 17 miles west of the site. The associated bedrock acceleration was estimated to be 0.19g.

2.1.2 Structural

Our review of the spillway structural design is based on the following documents:

- Drawings
  - Repair of Spillway of Middle Fork Dam, sheet 1 of 4 sheets showing plan of chute spillway
  - Repair of Spillway of Middle Fork Dam, sheet 2 of 4 sheets showing spillway profile, typical section of channel, details of underdrainage and cutoffs, and schedule of reinforcement
  - Repair of Spillway of Middle Fork Dam, sheet 3 of 4 sheets showing plan and profile of the transition section at the chute entrance
  - Repair of Spillway of Middle Fork Dam, sheet 4 of 4 sheets showing plan, profile, and sections of junction of downstream end of chute with old chute
  - Spillway Modification Plan and Profile, sheet 5 of 9 from James C. Hanson Consulting Civil Engineer, as-built dated 1-18-91, showing plan and profile of training wall addition along length of chute
  - Spillway Crest Details, sheet 6 of 9 from James C. Hanson Consulting Civil Engineer, as-built dated 1-18-91, showing plan and profile of training wall addition along transition section at the chute entrance
  - Spillway Transition Structure, sheet 7 of 9 from James C. Hanson Consulting Civil Engineer, as-built dated 1-18-91, showing plan, elevation, and sections of training wall addition along transition section at the chute entrance
  - Miscellaneous Spillway Sections and Details, sheet 8 of 9 from James C. Hanson Consulting Civil Engineer, as-built dated 1-18-91, showing sections and details of training wall addition along chute
- Field reports with photographs obtained from DSOD files.

See Appendix A for the drawings of the spillway structure listed above.

The chute spillway is comprised of a transition section and a channel section of reinforced concrete. The transition section extends from Station 1+90 at the upstream end of the spillway to Station 2+60. The channel section extends from Station 2+60 to the downstream end of the chute at Station 6+65.

The transition section begins at Station 1+90 with a 30-inch-deep, 16-inch-thick cutoff wall connected to a 6-inch-thick approach slab with a 20-percent adverse slope to the crest elevation of 3020.0 at Station 2+00. The cutoff wall extends up side slopes to the right and left of the approach slab and is connected to 6-inch-thick side slope slabs that transition to 8-inches thick for the upper portions added in 1989. The approach and 6-inch-thick side slabs have a single mat of 5/8” longitudinal bars spaced at 12 inches and 1/2” transverse bars spaced at 18 inches. The 8-inch-thick side slabs have a single mat of 1/2” longitudinal bars spaced at 12 inches and 5/8” transverse bars spaced at 12 inches.

The spillway crest is located at Station 2+00 at an elevation of 3020.0 and has a 28-foot bottom width with sides sloped at 1.5 horizontal to 1 vertical (1.5H:1V) as measured on a radial perpendicular to the toe of slope. There is an approximately 22-foot-deep, 8-inch-thick cutoff wall sloped upstream at 0.25H:1V located immediately upstream of the crest. This cutoff wall extends up the side slopes at a variable depth and has a single mat of 5/8” horizontal and vertical bars spaced at 12 inches each way. The upper portion of the cutoff wall added in 1989 is 12 inches thick with the same reinforcing pattern as the older section.

From Station 2+00 to 2+60 the bottom width converges from 28 feet to 8 feet while maintaining 1.5H:1V side slopes as measured on a radial perpendicular to the toe of slope. The spillway profile slopes at 20 percent from Station 2+03 to 2+20, and thereafter at 18 percent to Station 2+60. The bottom slab varies in thickness from 10.8 inches at Station 2+04.5 to 10 inches at Station 2+10 to 8 inches from Station 2+20 to 2+60. From Station 2+10 to 2+60, the bottom slab is underlain by an 8-inch-thick gravel blanket constructed on top of weathered bedrock. There are transverse construction joints at Stations 2+25 and 2+50 that are located atop reinforced concrete cutoff walls of trapezoidal section that extend 2 feet vertically below the bottom of the slab and laterally at least 2 feet beyond each exposed edge of the bottom slab. Two, 2-inch-diameter galvanized downspout pipes penetrate the cutoff wall and allow discharge from the gravel blanket upstream of the cutoff wall to daylight downstream of the transverse construction joint and cutoff wall. The transverse construction joints do not have waterstops, but do have continuous longitudinal reinforcement.

The bottom slab is reinforced with a single mat of 3/4” longitudinal bars spaced at 11 inches and 1/2” transverse bars spaced at 18 inches. The 8-inch-thick side slabs have a single mat of 5/8” longitudinal bars spaced at 10 inches and 1/2” transverse bars spaced at 18 inches. Extensions added to the side slabs in 1989 are comprised of 8-inch-thick slabs from Station 2+00 to 2+33.5 and concrete gravity walls from Station 2+33.5 to 2+60. The side slab extensions have slopes that progressively transition from 1.5H:1V at Station 2+00 to 1H:1V at Station 2+33.5 to 1H:4.6V at Station 2+44.25 to 1H:38V at Station 2+55 to 1H:39.4V at Station 2+60. The 8-inch-thick slab extensions have a single mat of 1/2” longitudinal bars spaced at 12 inches and 5/8” transverse bars spaced at 12 inches. From Station 2+33.5 to 2+44.25 the gravity wall extension has 3/4" vertical bars spaced at 12 inches and 3/4" horizontal bars spaced at 9 inches along each face of the wall. From Station 2+44.25 to 2+60 the gravity wall extension has 5/8" vertical bars spaced at 10 inches and 1/2" horizontal bars spaced at 10 inches along each face of the wall. The gravity wall extensions have stepped spread footings that are founded on undisturbed native soils.

The channel section begins at Station 2+60 and extends to the downstream end of the chute at Station 6+65. The channel has an 8-inch-thick bottom slab 8 feet wide and 8-inch-thick side slabs sloped at 1.5H:1V. The channel profile slopes at 18 percent from Station 2+60 to 2+90 and then increases in slope by 0.5 percent for every 10 feet until reaching 23.5 percent at Station 4+00. From Station 4+00 to 4+20 the slope increases by 1 percent for every 10 feet to become 25.5 percent at Station 4+20. For the next 20 feet the slope increases by 1.5 percent for every 10 feet to become 28.5 percent at Station 4+40. The channel then slopes at 30.5 percent for 10 feet followed by another 10 feet at 32 percent. From Station 4+60 to 5+70 the channel slopes at 33.3 percent. At Station 5+70 the slope begins decreasing to 31.8, 26.5, 21.1, 17, 13.6, 11.2, 8.2, 7.1, 5.6, and 2.6 percent for each successive 10-foot length until reaching Station 6+70.

From Station 2+60 to its junction with the abandoned spillway at approximate Station 6+20, the bottom slab is underlain by an 8-inch-thick gravel blanket constructed on top of weathered bedrock. There are transverse construction joints every 25 feet beginning at Station 2+75 that are located atop reinforced concrete cutoff walls of trapezoidal section that extend 2 feet vertically below the bottom of the slab and laterally at least 2 feet beyond each exposed edge of the bottom slab. Two, 2-inch-diameter galvanized downspout pipes penetrate the cutoff wall and allow discharge from the gravel blanket upstream of the cutoff wall to daylight downstream of the transverse construction joint and cutoff wall. The transverse construction joints do not have waterstops, but do have continuous longitudinal reinforcement. The bottom slab is reinforced with a single mat of 3/4” longitudinal bars spaced at 11 inches and 1/2” transverse bars spaced at 18 inches. A 6-inch-diameter open joint drain tile enveloped by 6 inches of gravel is located below and along each edge of the 8-inch-thick gravel blanket from Station 5+17 to 6+65. The drain tile passes through precast concrete panels incorporated into the cutoff walls to prevent seepage through the gravel envelope surrounding the drain tile (**Figure 2-1**). The material comprising the drain tile is not documented, but the author suspects it is vitrified clay pipe.

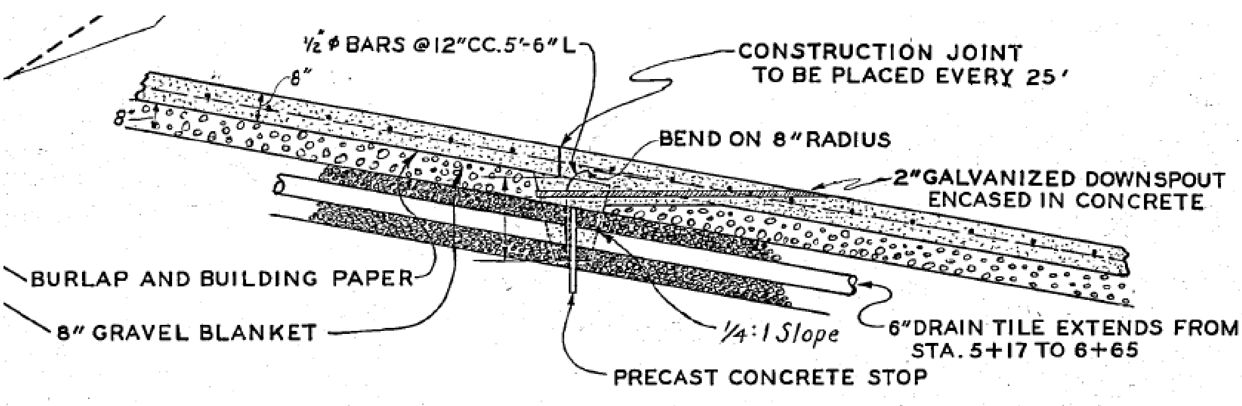

Figure 2-1: Typical section showing underdrain and cutoff wall.

The height of the side slabs varies from 9.8 feet at Station 2+60 to 6 feet at Station 6+60. Essentially vertical training walls were added to the top of the side slabs in 1989. These training walls vary in height from 5.5 feet at Station 2+60 to 5.0 feet at Station 6+60. The 8-inch-thick side slabs have a single mat of

5/8” longitudinal bars spaced at 10 inches and 1/2” transverse bars spaced at 18 inches. The training wall extensions have a single mat of 5/8” vertical bars at 12 inches and 1/2" horizontal bars at 14 inches. The training walls were constructed as a cantilever with a spread footing founded on undisturbed native soils. Training wall segments are 20 feet in length as measured along the slope and have a footing key that extends 3 feet into undisturbed native soils at the midpoint of the segment. No waterstops nor continuous horizontal reinforcement exists between training wall segments.

Structural design calculations for the chute spillway or extensions added in 1989 were not available for this review. The structural assessment presented herein is primarily based upon review of the construction drawings.

Findings and recommendations based on the structural review are summarized in Section 5.

2.1.3 Hydrology and Hydraulics

Our review of the spillway hydrology and hydraulics is based on the following documents:

- Middle Fork Dam STID (Wagner & Bonsignore 2006), Section 6, Hydrology and Hydraulics
- Seventh Part 12D Consultant’s Safety Inspection Report (Mead & Hunt, 2017)

The drainage basin to Schaads reservoir is approximately 28.5 square miles. The current Probable Maximum Flood (PMF) study was completed by James C. Hanson, Consulting Civil Engineer, in connection with the Middle Fork Dam Improvement Project of 1989. That PFM study used Hydrometeorological Report (HMR) No. 36 to determine the probable maximum precipitation (PMP). A 72-hour PMP depth of 31.88 inches resulted in a routed outflow of 13,220 cubic feet per second (cfs) from Middle Fork Dam and a peak reservoir elevation of 3039.8 feet.

In 2009, a PMF update was completed by Wagner & Bonsignore, Consulting Civil Engineers, using a PMP based on HMR 58/59. A 72-hour PMP depth of 33.78 inches resulted in a routed outflow of 14,670 cfs and a peak reservoir elevation of 3041.1 feet, which overtops the embankment dam and dike by 1.1 feet.

As the STID does not contain any information regarding hydraulic profiles in the spillway chute, we developed a HEC-RAS model of the spillway chute and computed the water surface profile at the PMF discharge of 13,220 cfs. The PMF water surface profile overtops the transition section and upper portion of the channel section by up to 2.1 feet.

Findings and recommendations from the Hydrology and Hydraulics review are summarized in Section 5.

2.2 Construction and Maintenance History

Our review of the construction and maintenance history is based on the Middle Fork Dam STID (Wagner & Bonsignore 2006), Section 3, Construction History, and DSOD field reports.

Middle Fork Dam was originally constructed in 1939. The original chute spillway failed in March 1940 due to uplift caused by overtaxing of the drainage system through and under the floor at the upstream end of the chute. In 1941, a new chute spillway was constructed in weathered bedrock to the right of the topographic saddle where the original spillway was located.

In 1975, the dam crest was raised 1.4 feet to provide a total freeboard of 16 feet. In 1989, the Middle Fork Dam Improvement Project added: stabilizing fill to form a 2.5H:1V slope on the downstream face of the dam, stabilizing fill and cut to form a 2.5H:1V slope on the upstream face and a 2H:1V slope on the downstream face of the dike, fill to raise the crest of the dam and dike by 4 feet to increase total freeboard to 20 feet, side slope extensions in the transition section and training walls in the channel section of the chute spillway to contain passage of the design flood, and extensions to the main outlet conduit and upper outlet conduit to accommodate the new embankment section.

2.3 Operational History

Our review of the spillway operational history is based on the following documents:

- Middle Fork Dam STID (Wagner & Bonsignore 2006), Section 4, Standard Operation Procedures
- Seventh Part 12D Consultant's Safety Inspection Report (Mead & Hunt, 2017)

The control section of the chute spillway is an overflow spillway of trapezoidal section with a fixed crest at elevation 3020.0 feet. Schaads reservoir is filled by natural inflow from the Middle Fork Mokelumne River. The rate of inflow varies seasonally, with higher flows occurring in the winter and spring from precipitation and snowmelt. When the reservoir elevation exceeds 3020.0 feet, flow passes down the spillway.

The spillway typically operates during normal and above-normal water years, although large spills are relatively infrequent. The flood of record as measured at the United States Geological Survey (USGS) stream gage on the Middle Fork Mokelumne River at West Point, CA (USGS Gage No. 11317000) occurred on January 2, 1997, and had a peak discharge of 5,040 cubic feet per second (cfs). The period of record at Gage No. 11317000 is from October 1911 until the time of writing. There is no device to measure flows at the Project, so the peak discharge at the Project on January 2, 1997 is unknown. Based on a ratio of the drainage area at the Project (28.5 square miles) to the drainage area at Gage No. 11317000 (68.4 square miles), the maximum flow at the Project is estimated to be approximately 2,100 cfs.

2.4 Historical Surveillance and Monitoring

Our review of the spillway surveillance and monitoring is based on the following documents:

- Middle Fork Dam STID (Wagner & Bonsignore 2006), Section 7, Surveillance and Monitoring Plan
- Seventh Part 12D Consultant's Safety Inspection Report (CSIR) (Mead & Hunt, 2017)
- Middle Fork Dam Safety Surveillance and Monitoring Plan (DSSMP) (Mead & Hunt, 2017)

The reservoir level is recorded using a sensor located near the right abutment of the dam. There is also a staff gage in the reservoir used to verify the headwater elevation determined from the headwater sensor or measure the reservoir elevation when the water surface is below the sensor.

Trained CPUD operations personnel inspect the spillway monthly and record their visual observations on a checklist, a sample of which is included in Appendix C of the DSSMP. Operations personnel make observations for the following items.

- Cracking, spalling, or general deterioration of concrete
- Joint misalignment
- Seepage through cracks/joints
- Joint or crack movement
- Foundation drain outlet condition
- Presence of weeds in cracks/joints

Any unusual observations are reported to the Water System Superintendent for review and follow-up action. If further review or follow-up action is required, the Water System Superintendent forwards the information to the District Manager.

The FERC and DSOD engineers also observe the spillway annually. Every five years an Independent Consultant observes the spillway and reviews the PFMs as part of the FERC Part 12D inspection.

There is no instrumentation associated with the Middle Fork Dam spillway.

Review of the seventh CSIR prior to the detailed spillway inspection performed for this assessment revealed no significant adverse conditions, and the only recommendation regarding the spillway was to fill an offset joint of a side slab with flexible sealant.

2.5 Data Gaps

The following are items pertinent to the assessment of the spillway that were not available during our desktop review:

- Stability calculations for the channel section training wall segments under loads imposed by the PMF discharge. These calculations are apparently summarized in a letter from James C. Hanson, Consulting Civil Engineer, to TKO Power dated July 20, 1990.

3. Inspection

3.1 Summary of Detailed Field Inspection

Warren Hayden, PE and Drake Hughes, PE of Mead & Hunt performed a detailed inspection of the Middle Fork Dam spillway on September 10, 2018. The weather conditions at the time of inspection were sunny and warm and there was approximately 0.5 inch of water flowing over the spillway crest. The inspection proceeded from the crest of the spillway to the toe of the chute.

The inspection findings are presented below.

3.1.1 Spillway Approach and Crest

The spillway approach channel contained a substantial amount of woody debris upstream of the log boom (**Figure 3-1**). CPUD staff indicated that they planned to remove the debris when the reservoir level had dropped, and the approach channel was dewatered sufficiently to support heavy equipment.

Figure 3-1: Woody debris upstream of log boom across spillway approach channel.

The spillway crest was level, in good condition, with a moderate amount of small woody debris at rest there (**Figure 3-2**).

Figure 3-2: Spillway crest with moderate amount of woody debris.

3.1.2 Spillway Chute Transition

The joints on both sides were generally tight and absent offsets. The side slope slabs on the left side had weeds growing through most of the joints (**Figure 3-3**). The bottom slab showed evidence of minor scour that has exposed the concrete aggregate (**Figure 3-4**).

Figure 3-3: Weeds in joints on left side slope slabs.

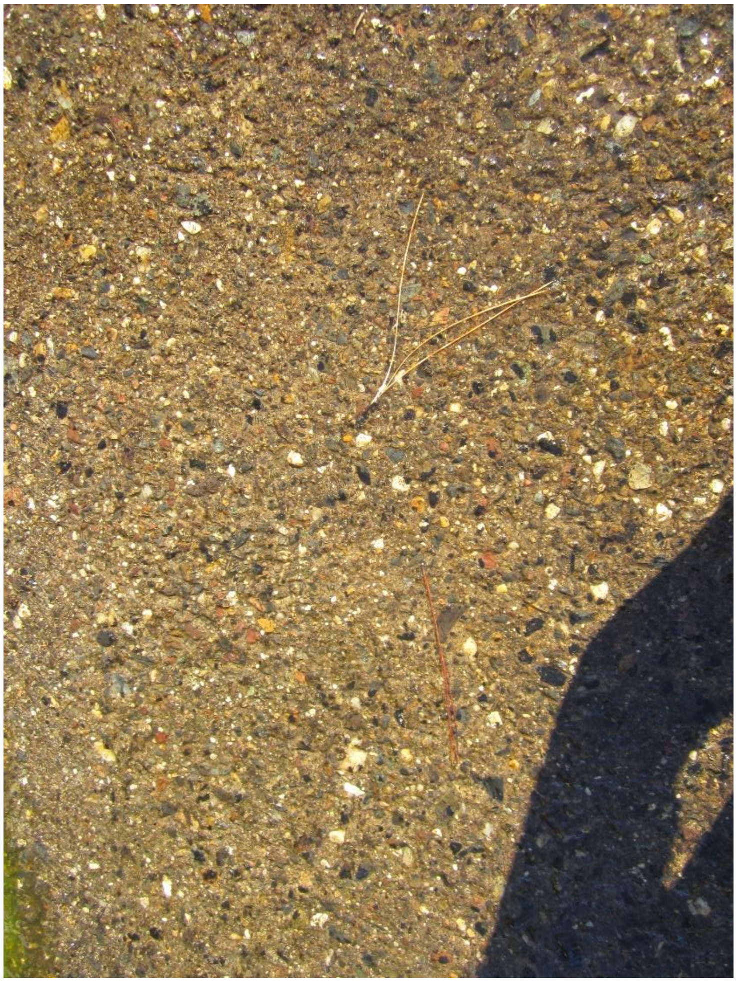

Figure 3-4: Exposed aggregate in bottom slab.

The transverse joint at Station 2+25 was tight with an approximate positive offset (downstream slab lower than upstream slab) of 1/8" (**Figure 3-5**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition.

Figure 3-5: Transverse joint at Station 2+25.

There was an area of spalled concrete at the base of the right side slope extension at approximately Station 2+42 (**Figure 3-6**). While this is essentially cosmetic damage, it should be chipped out to sound concrete and patched with repair mortar.

Figure 3-6: Spalled concrete at base of right side slope extension at Station 2+42.

The transverse joint at Station 2+50 was not measured for opening and offset, but did not appear to have an appreciable amount of either (**Figure 3-7**). An approximate 12" length at the left edge of the joint had been previously patched. Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were three transverse cracks observed between transverse joints at Stations 2+50 and 2+75. The first crack (2A) was located at approximately 10-10.5' (all crack locations were measured along the channel slope from the upstream joint) and had been previously patched (**Figure 3-8**). The second crack (2B) was located at approximately 14' and had a maximum opening of approximately 3/8" (**Figure 3-9**). The third crack (2C) was located at approximately 18.5' and had no appreciable opening nor offset (**Figure 3-10**).

Figure 3-7: Transverse joint at Station 2+50.

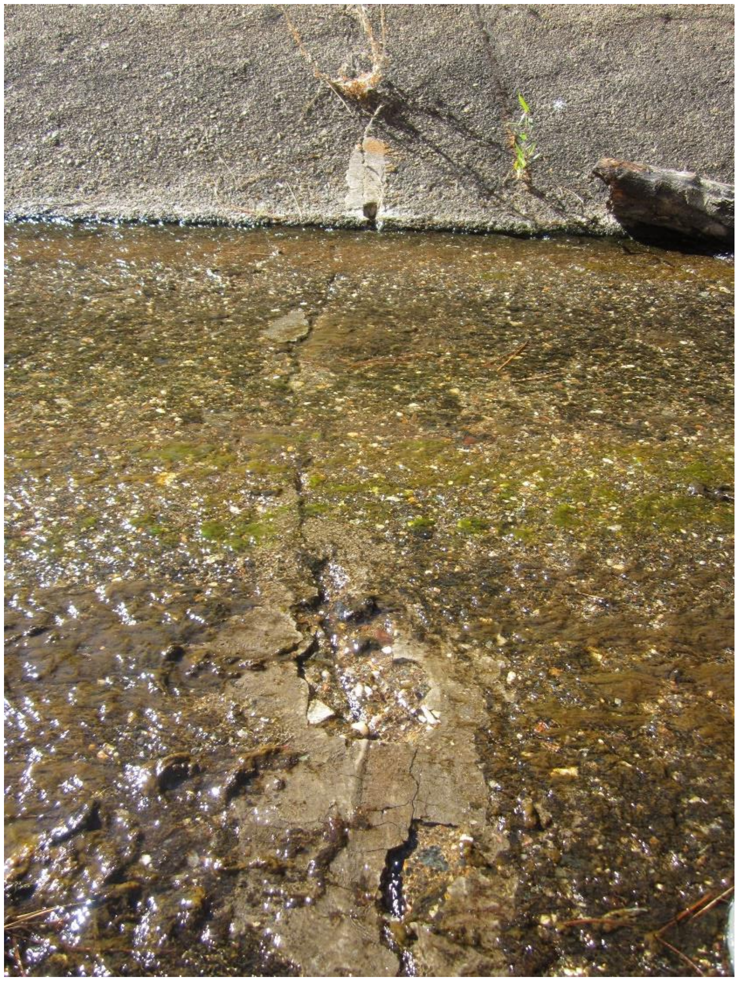

Figure 3-8: Crack 2A.

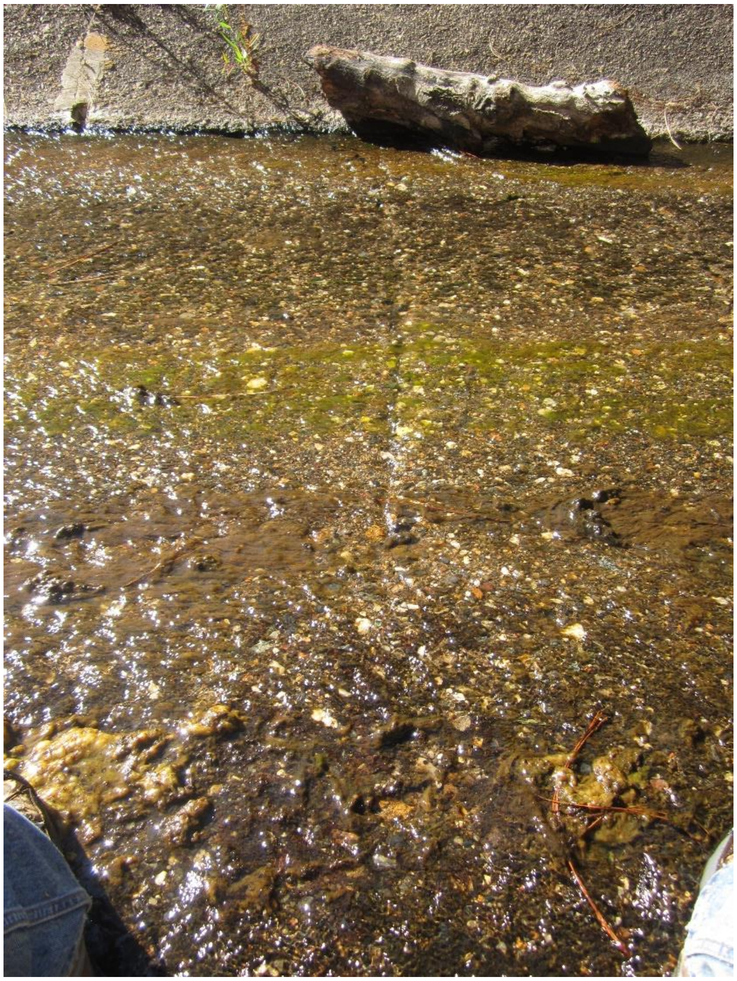

Figure 3-9: Crack 2B.

Figure 3-10: Crack 2C.

3.1.3 Spillway Chute Channel

The transverse joint at Station 2+75 had an approximate positive offset of 1/4" (**Figure 3-11**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There was a little gravel observed downstream of the right blanket drain outlet pipe (**Figure 3-12**). Presumably this gravel originated from the gravel blanket upstream of the cutoff wall at Station 2+75. There was one transverse crack observed between transverse joints at Stations 2+75 and 3+00. That crack (3A) was located at approximately 11.5' and was tight (**Figure 3-13**).

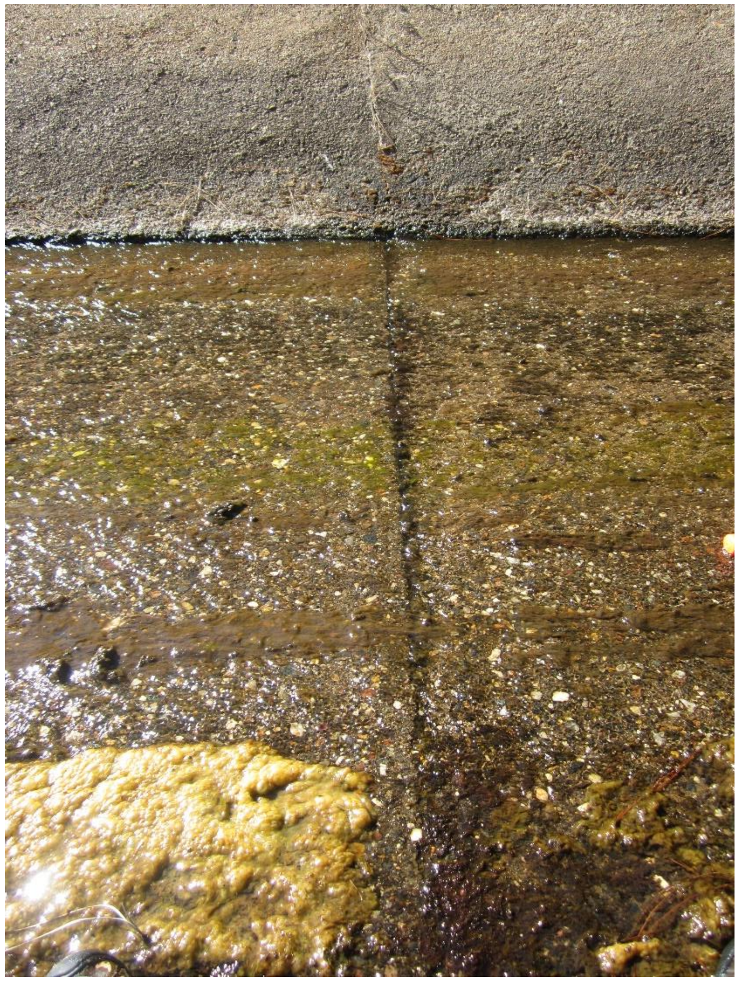

Figure 3-11: Transverse joint at Station 2+75.

Figure 3-12: Gravel downstream of right blanket drain outlet beyond Station 2+75.

Figure 3-13: Crack 3A.

The transverse joint at Station 3+00 had an approximate positive offset of 1/4" (**Figure 3-14**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. The joint between the right side slope slabs at approximately Station 3+00 had a maximum opening of approximately 1/4" and a positive offset of 2" at the top of the slope (**Figure 3-15**). We suspect this offset is being caused by root intrusion from nearby trees. The seventh CSIR recommended filling this joint with sealant; however, that had not been done at the time of this inspection. There were two transverse cracks observed between transverse joints at Stations 3+00 and 3+25. The first crack (4A) was located at approximately 15.5' and spanned about half the bottom slab (**Figure 3-16**). The second crack (4B) was located at approximately 22.5' and had a maximum opening of approximately 1/4" (**Figure 3-17**). Portions of this crack had been previously patched.

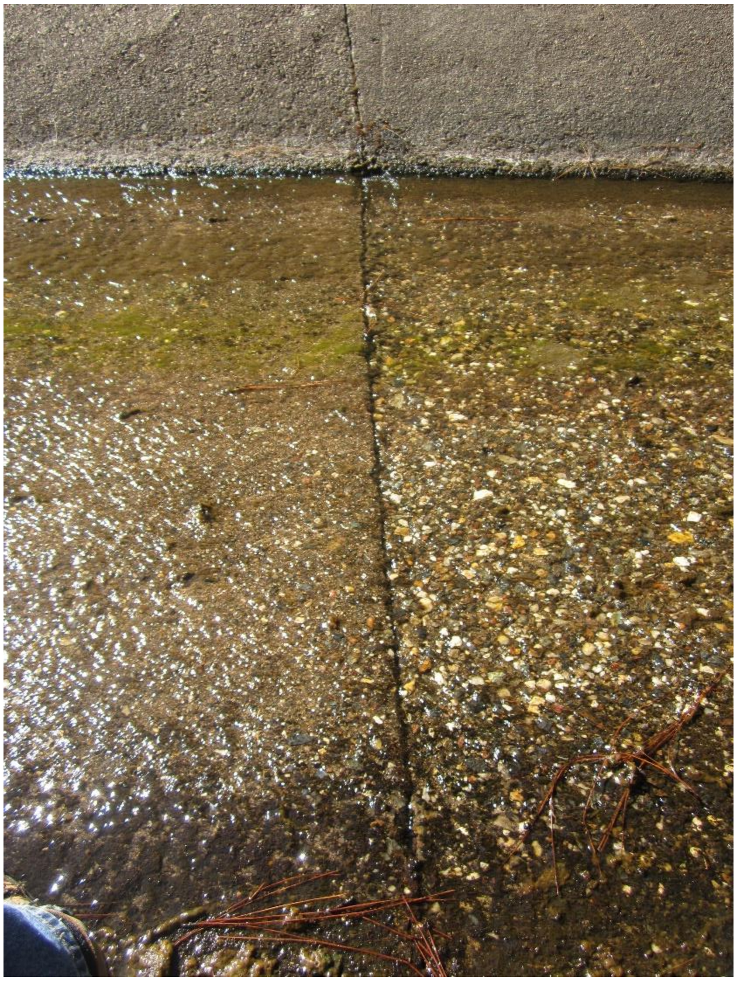

Figure 3-14: Transverse joint at Station 3+00.

Figure 3-15: Positive offset at top of right side slope slab at Station 3+00.

Figure 3-16: Crack 4A.

Figure 3-17: Crack 4B.

The transverse joint at Station 3+25 had a maximum opening of approximately 1/4"-1/2" (**Figure 3-18**). On the right side of the joint there was a hole approximately 6" long and 4" deep (**Figure 3-19**). This hole should be patched with repair mortar. Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There was one transverse crack observed between transverse joints at Stations 3+25 and 3+50. That crack (5A) was located at approximately 9.75' (**Figure 3-20**).

Figure 3-18: Transverse joint at Station 3+25.

Figure 3-19: Hole in slab at right side of joint at Station 3+25.

Figure 3-20: Crack 5A.

The transverse joint at Station 3+50 had an approximate positive offset of 1/2" (**Figure 3-21**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were two transverse cracks observed between transverse joints at Stations 3+50 and 3+75. The first crack (6A) was located at approximately 8' near where the blanket drain outlet pipe daylight (**Figure 3-22**). The second crack (6B) was located at approximately 13' and had a maximum opening of approximately 1/8"-1/2" (**Figure 3-23**). Portions of this crack had been previously patched.

Figure 3-21: Transverse joint at Station 3+50.

Figure 3-22: Crack 6A.

Figure 3-23: Crack 6B.

The transverse joint at Station 3+75 had a maximum opening of approximately 5/8” (**Figure 3-24**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were three transverse cracks observed between transverse joints at Stations 3+75 and 4+00. The first crack (7A) was located at approximately 3.5’ and had a maximum opening of approximately 1/8” (**Figure 3-25**). The second crack (7B) was located at approximately 12.5’ and had a maximum opening of approximately 1/4” (**Figure 3-26**). The third crack (7C) was located at approximately 23’ and had a maximum opening of approximately 1/2” (**Figure 3-27**).

Figure 3-24: Transverse joint at Station 3+75.

Figure 3-25: Crack 7A.

Figure 3-26: Crack 7B.

Figure 3-27: Crack 7C.

The transverse joint at Station 4+00 had an approximate positive offset of 3/8”-1/2” and had been previously repaired along its right side (**Figure 3-28**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There was a small spalled area upstream of the right blanket drain outlet pipe (**Figure 3-29**). There were two transverse cracks observed between transverse joints at Stations 4+00 and 4+25. The first crack (8A) was located at approximately 12’ and had a maximum opening of approximately 1/4” (**Figure 3-30**). There is a hole approximately 3.5” deep in the bottom of the left side slab at its junction with the bottom slab (**Figure 3-31**). This hole should be patched with repair mortar. The second crack (8B) was located at approximately 24.75’ and was tight (**Figure 3-32**).

Figure 3-28: Transverse joint at Station 4+00.

Figure 3-29: Spall upstream of right blanket drain outlet pipe.

Figure 3-30: Crack 8A.

Figure 3-31: Hole in bottom of left side slab at Station 4+00.

Figure 3-32: Crack 8B.

The transverse joint at Station 4+25 had an approximate positive offset of 1/2” and maximum opening of approximately 1/4” (**Figure 3-33**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were four transverse cracks observed between transverse joints at Stations 4+25 and 4+50. The first crack (9A) was located at approximately 6.5’ and was tight and had been previously patched (**Figure 3-34**). The second crack (9B) was located at approximately 8.5’ and was tight (**Figure 3-35**). The third crack (9C) was located at approximately 12.5’ and the right side had been previously patched but was raveling open approximately 1/8” (**Figure 3-36**). At the left side of this crack there are two holes approximately 2.5” deep in the bottom of the left side slab at its junction with the bottom slab (**Figure 3-37**). These holes should be patched with repair mortar. The fourth crack (9D) was located at approximately 25’ and had a maximum opening of approximately 1/16” and had been previously patched (**Figure 3-38**).

Figure 3-33: Transverse joint at Station 4+25.

Figure 3-34: Crack 9A.

Figure 3-35: Crack 9B.

Figure 3-36: Crack 9C.

Figure 3-37: Holes in bottom of left side slab at left side of crack 9C.

Figure 3-38: Crack 9D.

The transverse joint at Station 4+50 had an approximate positive offset of 1/4" and maximum opening of approximately 1/2" and had been previously repaired (**Figure 3-39**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were six transverse cracks observed between transverse joints at Stations 4+50 and 4+75. The concrete comprising the bottom slab in this reach exhibited several cracks and areas of surficial deterioration (**Figure 3-40**). It is our opinion that the bottom slab should be reconstructed in this reach. The first crack (10A) was located at approximately 4.5' and was tight and had an area of deteriorated concrete on the right side (**Figure 3-41**). The second crack (10B) was located at approximately 6.5' and was tight (**Figure 3-42**). The third crack (10C) was located at approximately 8.5' and had been previously patched and had an area of deteriorated concrete upstream and to the right of the patch (**Figure 3-43**). The fourth crack (10D) was located at approximately 12' and had an area of deteriorated concrete in the center and left side (**Figure 3-44**). The fifth crack (10E) was located at approximately 18' and was tight. The sixth crack (10F) was located at approximately 19.5', had a maximum opening of approximately 1/8", had been previously patched, and had areas of deteriorated concrete along its length (**Figure 3-45**).

Figure 3-39: Transverse joint at Station 4+50.

Figure 3-40: Bottom slab between Stations 4+50 and 4+75.

Figure 3-41: Crack 10A.

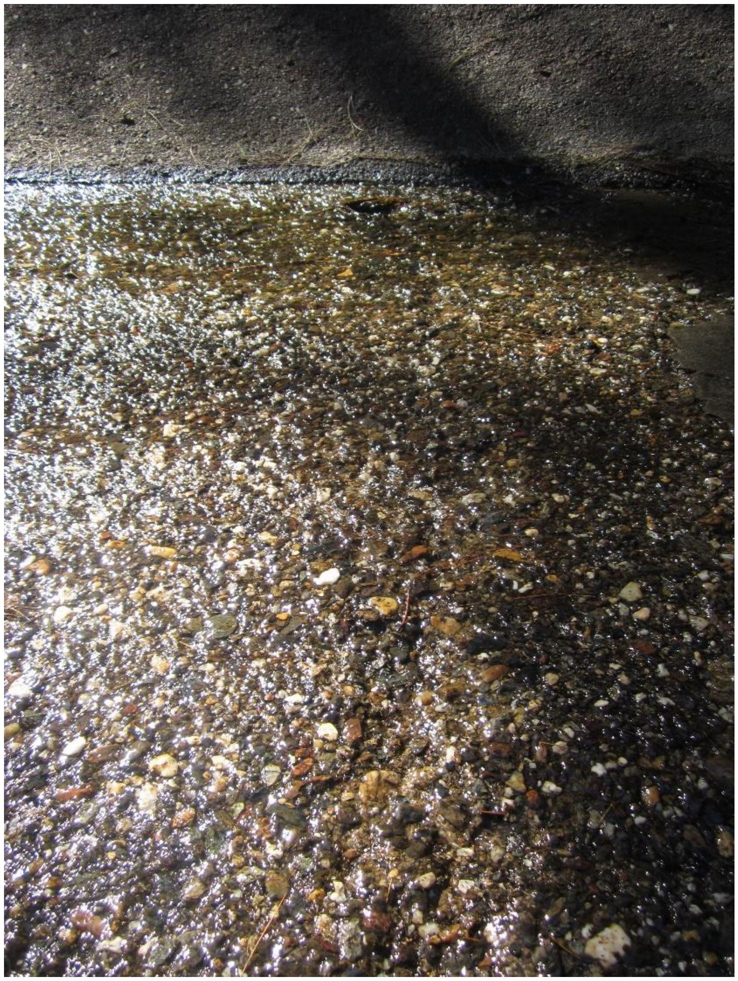

Figure 3-42: Crack 10B.

Figure 3-43: Crack 10C.

Figure 3-44: Crack 10D.

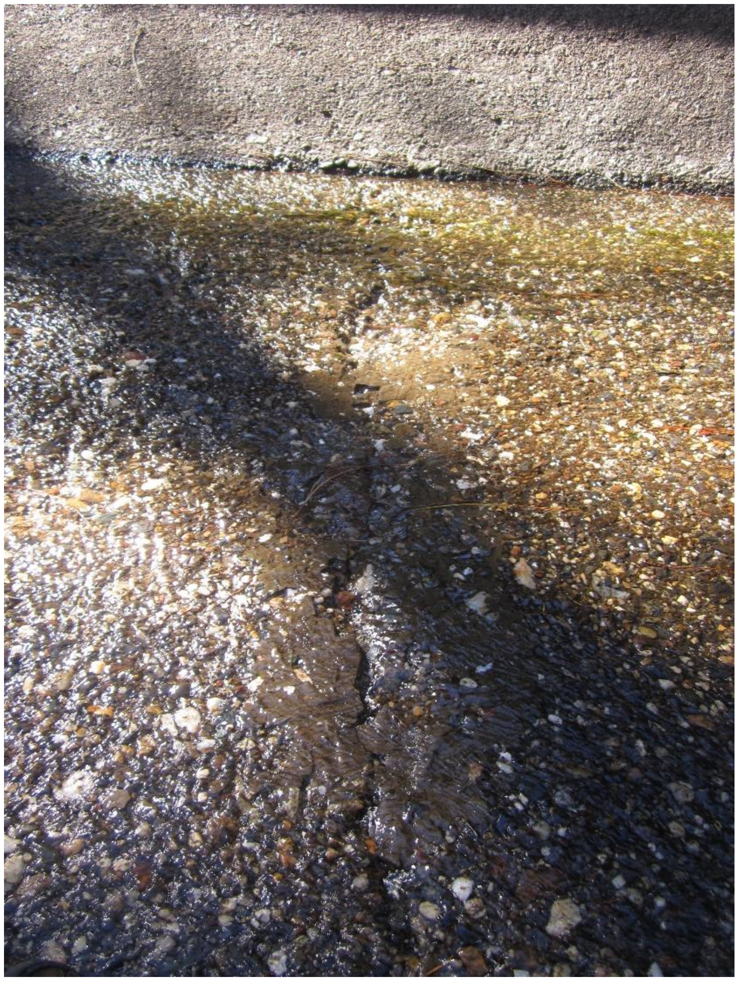
Figure 3-45: Crack 10F.

The transverse joint at Station 4+75 had an approximate positive offset of 1/2” and maximum opening of approximately 1/2” and had been previously repaired (**Figure 3-46**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were five transverse cracks observed between transverse joints at Stations 4+75 and 5+00. The concrete comprising the bottom slab in this reach exhibited several cracks and areas of surficial deterioration (**Figure 3-47**). It is our opinion that the bottom slab should be reconstructed in this reach. The first crack (11A) was located at approximately 5.25’ and was tight and had an area of deteriorated concrete (**Figure 3-48**). The second crack (11B) was located at approximately 9.75’ and was tight (**Figure 3-49**). The third crack (11C) was located at approximately 14’, had a maximum opening of approximately 1/2”, had been previously patched, and had areas of deteriorated concrete along its center third which had a maximum depth of approximately 1.5” (**Figure 3-50**). The fourth crack (11D) was located at approximately 17’ and was tight (**Figure 3-51**). The fifth crack (11E) was located at approximately 24.75’, had a maximum opening of approximately 1/8”, and had areas of deteriorated concrete which had a maximum depth of approximately 1” (**Figure 3-52**).

Figure 3-46: Transverse joint at Station 4+75.

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Figure 3-47: Bottom slab between Stations 4+75 and 5+00.

Figure 3-48: Crack 11A.

Figure 3-49: Crack 11B.

Figure 3-50: Crack 11C.

Figure 3-51: Crack 11D.

Figure 3-52: Crack 11E.

The transverse joint at Station 5+00 had an approximate positive offset of 1/4” and maximum opening of approximately 1/2” (**Figure 3-53**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were four transverse cracks observed between transverse joints at Stations 5+00 and 5+25. The concrete comprising the bottom slab in this reach exhibited several cracks and areas of surficial deterioration. It is our opinion that the bottom slab should be reconstructed in this reach. The first crack (12A) was located at approximately 4.5’ and was tight and had an area of significantly deteriorated concrete (**Figure 3-54**). The second crack (12B) was located at approximately 10.75’ and had a maximum opening of approximately 1/4” (**Figure 3-55**). The third crack (12C) was located at approximately 13’, had a maximum opening of approximately 1/8”, (**Figure 3-56**). There was an area of deteriorated concrete located at approximately 18.5’ (**Figure 3-57**). The fourth crack (12D) was located at approximately 21.25’, had a maximum opening of approximately 1/4”, and had areas of deteriorated concrete which had a maximum depth of approximately 2” (**Figure 3-58**). There were areas of deteriorated concrete located on the right and left sides at approximately 25’ (**Figure 3-59**).

Figure 3-53: Transverse joint at Station 5+00.

Figure 3-54: Crack 12A.

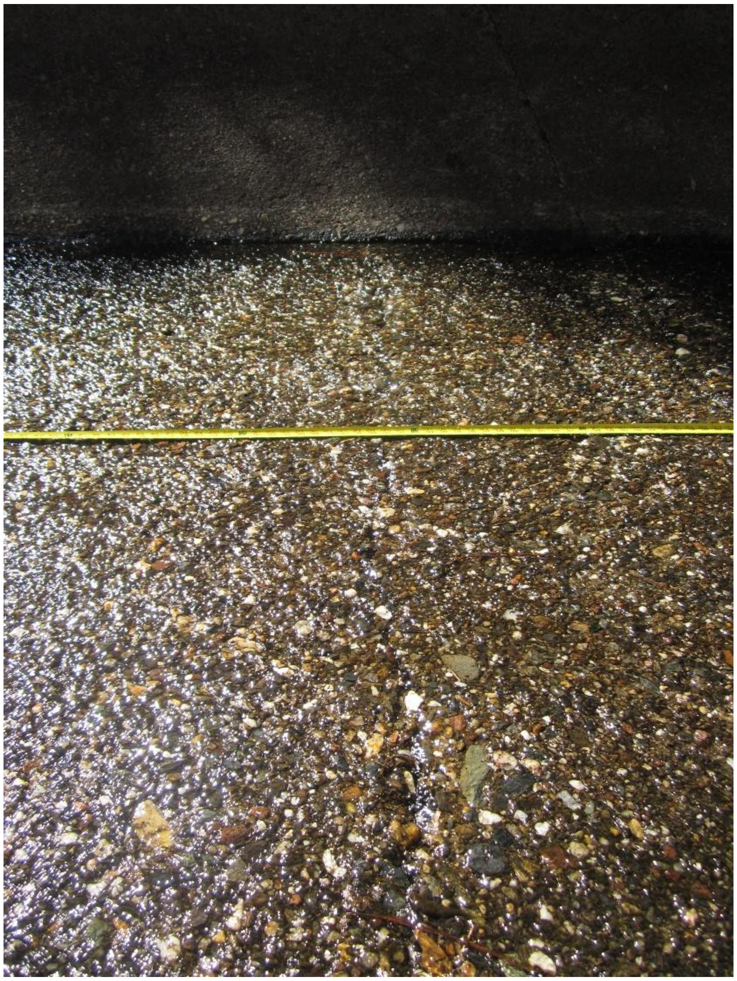


Figure 3-55: Crack 12B.

Figure 3-56: Crack 12C.

Figure 3-57: Area of deteriorated concrete at 18.5' downstream of Station 5+00.

Figure 3-58: Crack 12D.

Figure 3-59: Area of deteriorated concrete at 25' downstream of Station 5+00.

The transverse joint at Station 5+25 had an approximate positive offset of 1/4" and maximum opening of approximately 1/8" (**Figure 3-60**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were five transverse cracks observed between transverse joints at Stations 5+25 and 5+50. The concrete comprising the bottom slab in this reach exhibited several cracks and areas of surficial deterioration (**Figure 3-61**). It is our opinion that the bottom slab should be reconstructed in this reach. The first crack (13A) was located at approximately 6.5' and had areas of deteriorated concrete along its length (**Figure 3-62**). The second crack (13B) was located at approximately 7.75' and had areas of deteriorated concrete along its length (**Figure 3-62**). The third crack (13C) was located at approximately 13.75', had a maximum opening of approximately 1/8", and had areas of deteriorated concrete along its length which had a maximum depth of approximately 1.5" (**Figure 3-63**). The fourth crack (13D) was located at approximately 15.75' and was tight and had areas of deteriorated concrete along its length which had a maximum depth of approximately 1" (**Figure 3-64**). There was an area of deteriorated concrete located at approximately 20-22' on the left side of the slab (**Figure 3-65**). The fifth crack (13E) was located at approximately 24.5', was tight, had been previously repaired, and had areas of deteriorated concrete which had a maximum depth of approximately 1.5" (**Figure 3-66**).

Figure 3-60: Transverse joint at Station 5+25.

Figure 3-61: Bottom slab between Stations 5+25 and 5+50.

Figure 3-62: Crack 13A and 13B.

Figure 3-63: Crack 13C.

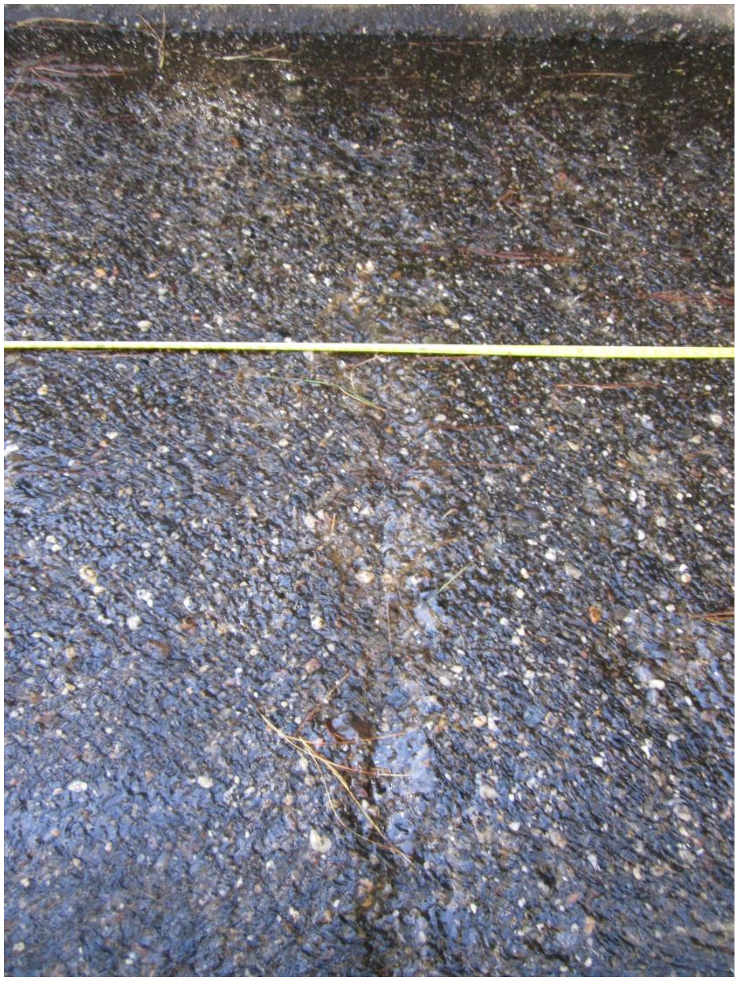


Figure 3-64: Crack 13D.

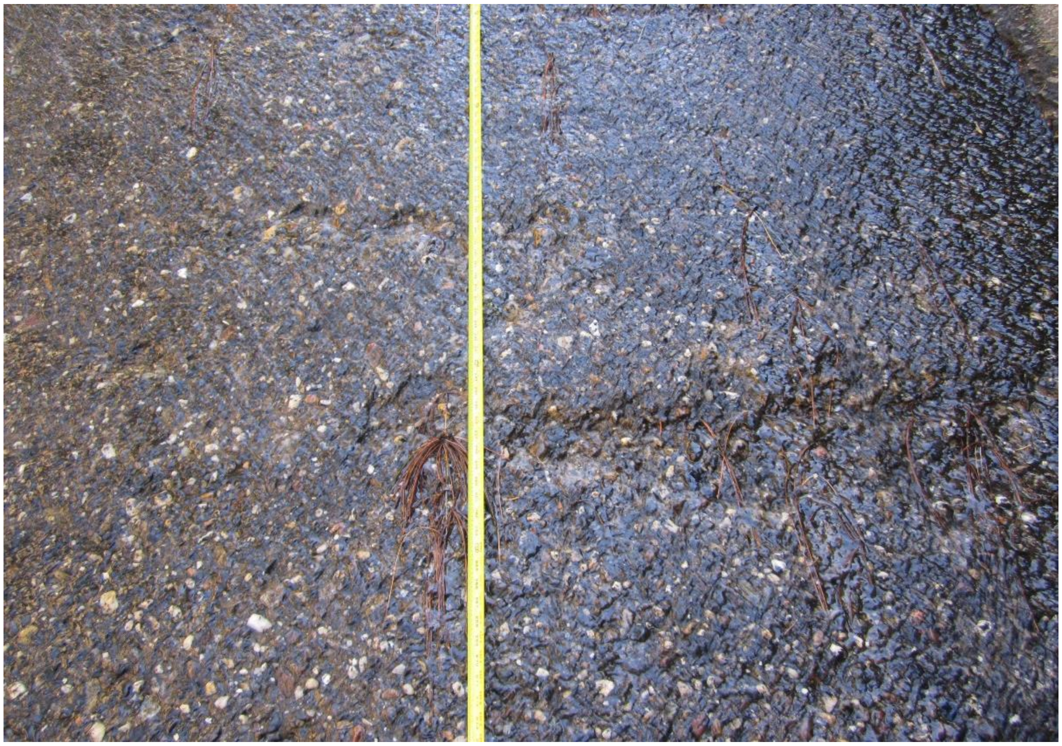


Figure 3-65: Area of deteriorated concrete at 20-22' downstream of Station 5+25.

Figure 3-66: Crack 13E.

The transverse joint at Station 5+50 had a maximum opening of approximately 1/8" (**Figure 3-67**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were three transverse cracks observed between transverse joints at Stations 5+50 and 5+75. The concrete comprising the bottom slab in this reach exhibited several cracks and areas of surficial deterioration (**Figure 3-68**). It is our opinion that the bottom slab should be reconstructed in this reach. The first crack (14A) was located at approximately 13' and had been previously repaired in the center (**Figure 3-69**). There was a hole 5" deep in the bottom of the right side slab at its junction with the bottom slab (**Figure 3-70**). This hole should be patched with repair mortar. The second crack (14B) was located at approximately 21.5' and was tight and had areas of deteriorated concrete along its length (**Figure 3-71**). The third crack (14C) was located at approximately 24' and was tight and had areas of deteriorated concrete along its length (**Figure 3-72**).

Figure 3-67: Transverse joint at Station 5+50.

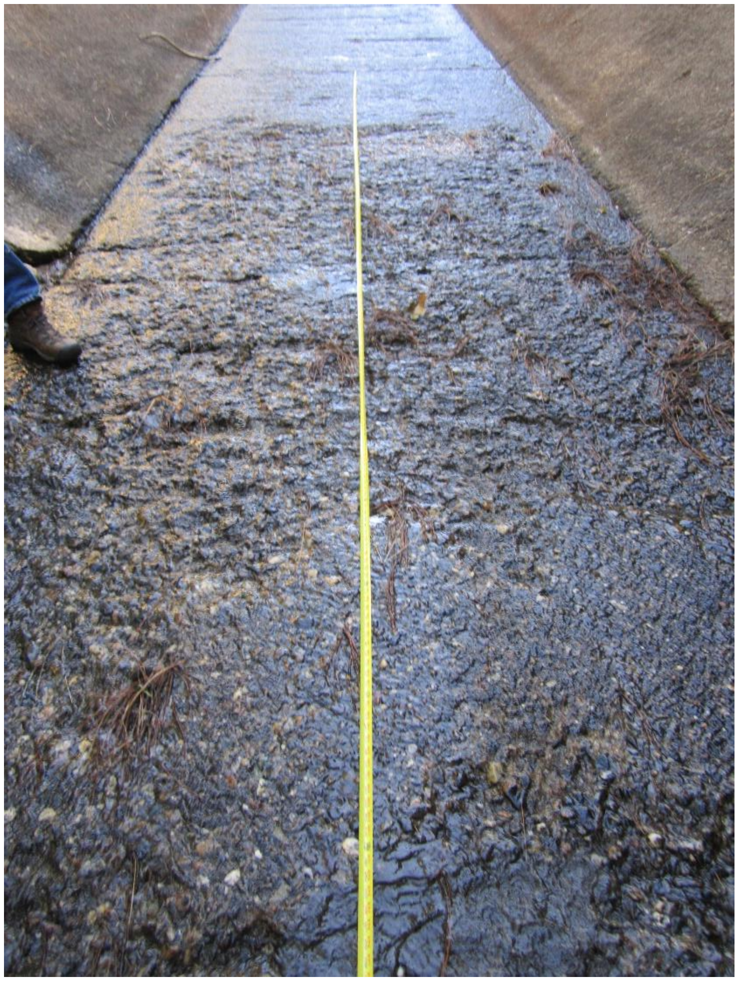


Figure 3-68: Bottom slab between Stations 5+50 and 5+75.

Figure 3-69: Crack 14A.

Figure 3-70: Hole at right side of crack 14A.

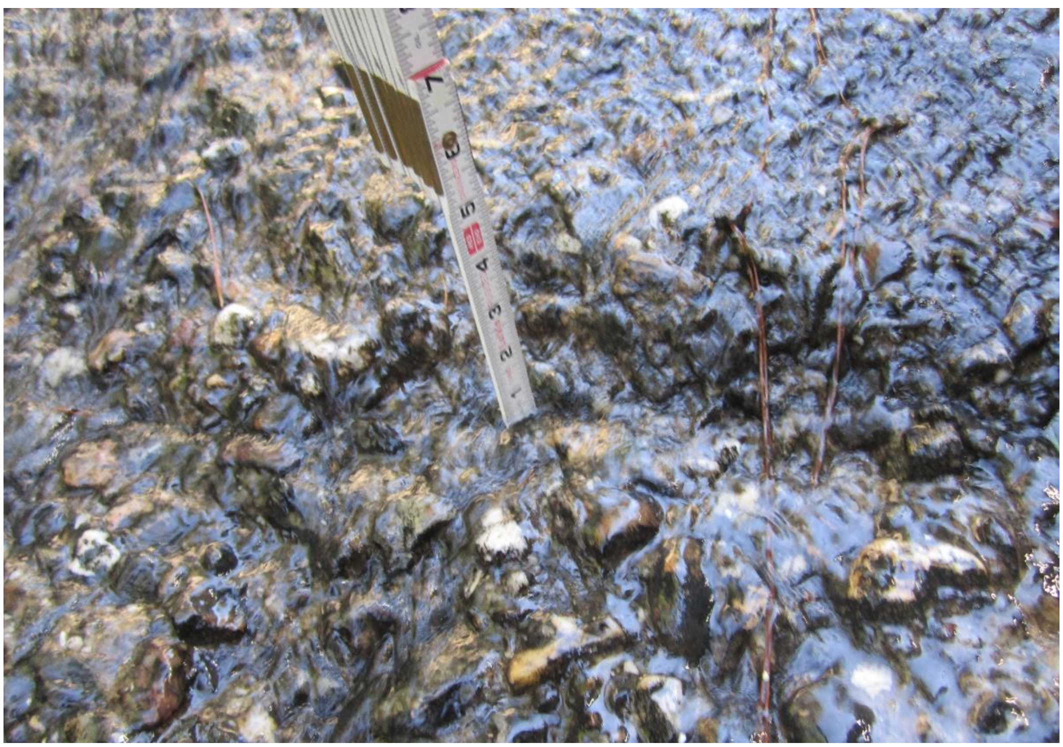


Figure 3-71: Crack 14B.

Figure 3-72: Crack 14C.

The transverse joint at Station 5+75 had an approximate positive offset of 1/4" and maximum opening of approximately 1/4" (**Figure 3-73**). Downstream of this joint, each blanket drain outlet pipe daylighting near the right and left edges of the bottom slab was in good condition. There were four transverse cracks observed between transverse joints at Stations 5+75 and 6+00. The concrete comprising the bottom slab in this reach exhibited several cracks and areas of surficial deterioration (**Figure 3-74**). It is our opinion that the bottom slab should be reconstructed in this reach. The first crack (15A) was located at approximately 4.5' and was tight and had areas of deteriorated concrete along its length which had a maximum depth of approximately 1.5" (**Figure 3-75**). The second crack (15B) was located at approximately 8' and had areas of deteriorated concrete along its length (**Figure 3-76**). The third crack (15C) was located at approximately 13.5', was tight, had been previously repaired, and had areas of deteriorated concrete at the right side which had a maximum depth of approximately 1.5" (**Figure 3-77**). There was an area of deteriorated concrete located at approximately 22' which had a maximum depth of approximately 1.5" (**Figure 3-78**). The fourth crack (15D) was located at approximately 23.75' and was tight and had areas of deteriorated concrete along its length which had a maximum depth of approximately 1" (**Figure 3-79**).

Figure 3-73: Transverse joint at Station 5+75.

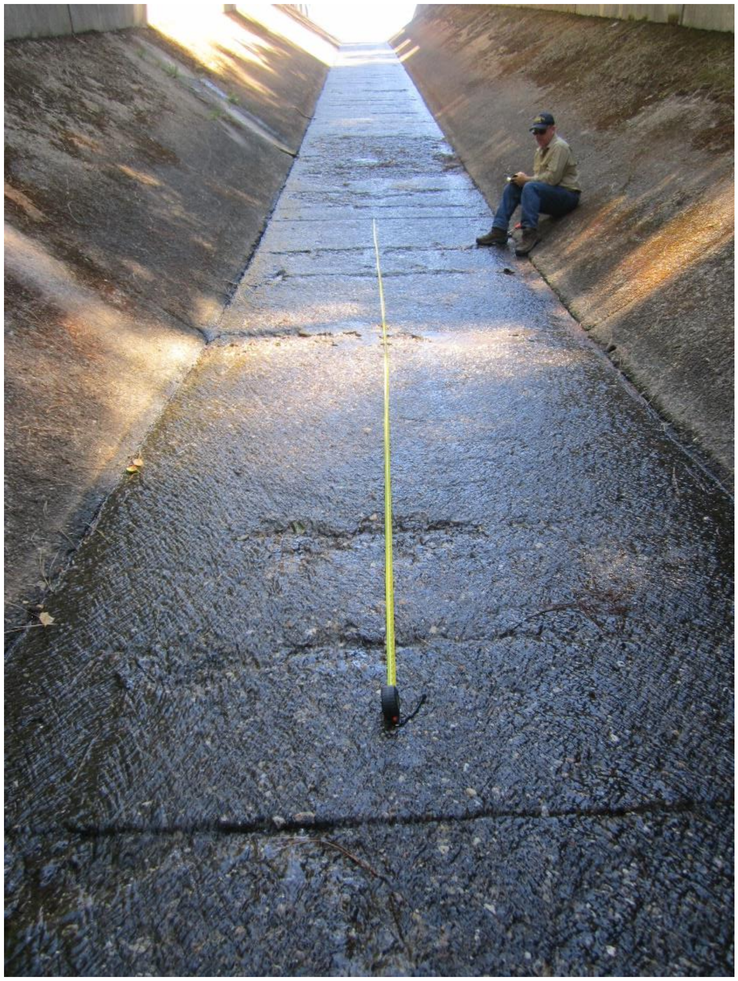


Figure 3-74: Bottom slab between Stations 5+75 and 6+00.

Figure 3-75: Crack 15A.

Figure 3-76: Crack 15B.

Figure 3-77: Crack 15C.

Figure 3-78: Area of deteriorated concrete at 22' downstream of Station 5+75.

Figure 3-79: Crack 15D.

The transverse joint at Station 6+00 had an approximate positive offset of 1/2” and maximum opening of approximately 1/4” and deteriorated concrete along its right side (**Figure 3-80**). Downstream of this joint, four blanket drain outlet pipes in good condition daylight through the bottom slab. The concrete comprising the bottom slab downstream of Station 6+00 exhibited heavy surficial deterioration which had a maximum depth of approximately 2” (**Figure 3-81**). It is our opinion that the bottom slab should be reconstructed in this reach. The end of the chute bottom slab at approximately Station 6+70 showed no evidence of undermining (**Figure 3-82**).


Figure 3-80: Transverse joint at Station 6+00.

Figure 3-81: Area of deteriorated concrete downstream of Station 6+00.

Figure 3-82: End of chute bottom slab at approximately Station 6+70.

3.1.4 Plunge Pool and Discharge Channel

The chute discharges into a plunge pool comprised of exposed bedrock (**Figure 3-83**). The bedrock appeared sound and is evidently resistant to scour from spillway flows. Downstream of the plunge pool, flow re-joins the Middle Fork Mokelumne River.

Figure 3-83: Exposed bedrock plunge pool at end of chute.

4. PFMA Session

4.1 Summary of Spillway-Focused PFMA Session

A spillway-focused PFMA session for the Middle Fork Dam spillway was held on September 10, 2018, following the detailed spillway inspection earlier that day. The PFMA session was held at the CPUD office in San Andreas, where the following participants reviewed the existing PFMs specific to the spillway and developed three new PFMs associated with the spillway.

Table 1. PFM Review Team

Name	Function	Organization
Warren Hayden	IC	Mead & Hunt
Drake Hughes	Engineer	Mead & Hunt
Donna Leatherman	Exemptee	CPUD
Bret Beaudreau	Exemptee	CPUD
Golam Kabir	FERC Inspector	FERC NYRO
Kelly Owens	FERC Inspector	FERC NYRO

In reviewing the existing PFMs and developing new PFMs, participants considered the relevant geological, geotechnical, hydrology, hydraulics, structural, construction, operation, maintenance, and surveillance and monitoring information associated with the spillway together with findings from the detailed spillway inspection earlier that day.

4.1.1 Existing Spillway PFMs

Five existing PFMs related to the spillway were reviewed. Information regarding these PFMs is taken from Section 3 of the seventh CSIR (Mead & Hunt, 2017).

Potential Failure Mode 4B – Blockage of the spillway at flood pool due to a landslide of the slope to the left of the spillway

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, a landslide of the slope to the left of the spillway enters the spillway and blocks discharge from the spillway, causing the reservoir pool to rise until it overtops a portion of the embankment crest, initiating erosion of the embankment section and headcutting that further lowers the crest elevation, thereby increasing the overtopping discharge and progressively eroding the embankment section until it breaches, resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
	• Landform left of spillway not able to mobilize sufficient material to block spillway • Complete blockage of flowing spillway is unlikely

Categorization and Rationale for Assigned Category

The review team unanimously classified this PFM as Category IV because the landform to the left of the spillway is not large enough to mobilize sufficient material to block the spillway.

The spillway-focused PFMA review team concurred that PFM 4B was correctly classified as Category IV.

Potential Failure Mode 4C – Blockage of the spillway at flood pool due to a landslide of the slope to the right of the spillway

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, a landslide of the slope to the right of the spillway enters the spillway and blocks discharge from the spillway, causing the reservoir pool to rise until it overtops a portion of the embankment crest, initiating erosion of the embankment section and headcutting that further lowers the crest elevation, thereby increasing the overtopping discharge and progressively eroding the embankment section until it breaches, resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
	• Landform right of spillway is set back from spillway • There is a relatively flat slope between landform and spillway • Complete blockage of flowing spillway is unlikely

Categorization and Rationale for Assigned Category

The review team unanimously classified this PFM as Category IV because the landform to the right of the spillway is set back from the spillway channel and there exists a relatively flat slope between the spillway and landform.

The spillway-focused PFMA review team concurred that PFM 4C was correctly classified as Category IV.

Potential Failure Mode 9 – Collapse of spillway crest at flood pool due to hydraulic jacking of spillway liner and progressive backward unraveling of spillway liner

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, flow in the chute spillway enters an open concrete joint and penetrates below a slab section, causing hydraulic jacking and failure of that slab section, leading to undermining and erosion of the adjacent upstream slab section. The progressive backward erosion and failure of slab sections continues upstream until reaching the spillway crest, whereupon the spillway cutoff wall is then undermined, leading to collapse of the spillway crest, and resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
• Joints exist in spillway chute	• Gravel blanket with drainage pipes underlies bottom slab • Concrete cutoff walls below bottom slab exist every 25 feet along alignment

Categorization and Rationale for Assigned Category

The review team was divided on the classification of this PFM. Some members had the opinion that this PFM should be classified as Category II because the physical possibility of slab jacking and progressive unraveling during a longer-duration spillway flow cannot be ruled out. Other members had the opinion that this PFM should be classified as Category IV because the concrete cutoff walls between slabs would likely halt the progressive upstream unraveling prior to collapse of the crest. It is the opinion of the IC that this PFM should be classified as Category II because hydraulic jacking of a spillway slab and progressive backward unraveling is physically possible, and therefore cannot be ruled out.

Potential risk-reduction measures

- Continue monthly observations of the spillway to monitor for displacement of joints

The spillway-focused PFMA review team concurred that PFM 9 was correctly classified as Category II. The review team also suggested an additional risk-reduction measure of sealing the joints in the chute to prevent the ingress of water that could cause hydraulic jacking of a slab.

Potential Failure Mode 9A – Collapse of spillway crest at flood pool due to undermining of spillway liner at its exit and progressive backward unraveling of spillway liner

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, flow in the chute spillway undermines the slab at its downstream end, causing failure of that slab section and leading to undermining and erosion of the adjacent upstream slab section. The progressive backward erosion and failure of slab sections continues upstream until reaching the spillway crest, whereupon the spillway cutoff wall is then undermined, leading to collapse of the spillway crest, and resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
	• Downstream end of spillway chute is founded on hard, jointed quartz diorite bedrock • Concrete cutoff walls below bottom slab exist every 25 feet along alignment

Categorization and Rationale for Assigned Category

The review team was divided on the classification of this PFM. Some members had the opinion that this PFM should be classified as Category II because the physical possibility of exit undermining and progressive unraveling during a longer-duration spillway flow cannot be ruled out. Other members had the opinion that this PFM should be classified as Category IV because the concrete cutoff walls between slabs would likely halt the progressive upstream unraveling prior to collapse of the crest. It is the opinion of the IC that this PFM should be classified as Category II because exit undermining and progressive backward unraveling is physically possible, and therefore cannot be ruled out.

Potential risk-reduction measures

- Continue monthly observations of the spillway exit to monitor for undermining

The spillway-focused PFMA review team concurred that PFM 9A was correctly classified as Category II.

Potential Failure Mode 13 – Overtopping failure of main dam embankment at flood pool due to debris blockage of spillway

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, a large quantity of floating debris accumulates at the entrance to the spillway and partially blocks discharge from the spillway, causing the reservoir pool to rise until it overtops a portion of the embankment crest, initiating erosion of the embankment section and headcutting that further lowers the crest elevation, thereby increasing the overtopping discharge and progressively eroding the embankment section until it breaches, resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
• Large trees exist along the reservoir rim and in the watershed • Essentially no freeboard exists at the IDF	

Categorization and Rationale for Assigned Category

The review team unanimously classified this PFM as Category II because surcharging of the reservoir due to debris blockage of the spillway is physically possible, and therefore this PFM cannot be ruled out.

Potential risk-reduction measures

- Regularly dispose of debris that accumulates at the entrance to the spillway during normal flows

The spillway-focused PFMA review team concurred that PFM 13 was correctly classified as Category II.

4.1.2 New Spillway PFMs

The spillway-focused PFMA review team developed three new PFMs related to the spillway. Those PFMs are reported below and numbered as PFMs 14, 15, and 16.

Potential Failure Mode 14 – Collapse of spillway crest at flood pool due to overtopping of spillway training walls

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, the water surface profile in the chute overtops the training walls, eroding the backfill and foundation soils supporting the training walls, thereby causing the training walls to overturn and release additional flow along the sides of the chute which then undermines the side slabs which subsequently collapse, leading to undermining of the bottom slab whose removal leads to undermining and erosion of the adjacent upstream slab sections. The progressive backward erosion and failure of slab sections continues upstream until reaching the spillway crest, whereupon the spillway cutoff wall is then undermined, leading to collapse of the spillway crest, and resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
• The hydraulic profile in the chute at the IDF (PMF) is above the top of the training walls • The training walls are cantilever walls with spread footings founded on erodible soils • There is not continuous reinforcement between training wall segments	• Concrete cutoff walls below bottom slab exist every 25 feet along alignment • Cutoff wall at spillway crest is 22 feet deep

Categorization and Rationale for Assigned Category

The review team unanimously classified this PFM as Category I because the hydraulic profile in the chute at the IDF is above the top of the training walls.

Potential risk-reduction measures

- Raise the top of training wall profile above the hydraulic profile by adding a concrete parapet

Potential Failure Mode 15 – Collapse of spillway crest at flood pool due to undermining of spillway training walls from surface drainage

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, concentrated surface drainage flowing along the outside of the training walls erodes the backfill and foundation soils supporting the training walls, thereby causing the training walls to overturn and release additional flow along the sides of the chute which then undermines the side slabs which subsequently collapse, leading to undermining of the bottom slab whose removal leads to undermining and erosion of the adjacent upstream slab sections. The progressive backward erosion and failure of slab sections continues upstream until reaching the spillway crest, whereupon the spillway cutoff wall is then undermined, leading to collapse of the spillway crest, and resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
• The training walls are cantilever walls with spread footings founded on erodible soils • There is not continuous reinforcement between training wall segments	• Concrete cutoff walls below bottom slab exist every 25 feet along alignment • Cutoff wall at spillway crest is 22 feet deep • Long-duration intense precipitation needed to generate volume of runoff necessary to erode backfill and foundation soils

Categorization and Rationale for Assigned Category

The review team unanimously classified this PFM as Category II because erosion of the backfill and foundations soils is physically possible, and therefore this PFM cannot be ruled out.

Potential risk-reduction measures

- Construct an upslope swale to capture runoff and convey it downslope prior to reaching training walls
- If stability analyses indicate it is safe to do so, add additional backfill behind training walls to increase volume of soil that would have to be eroded

Potential Failure Mode 16 – Collapse of spillway crest at flood pool due to overtopping of spillway training walls from tree obstruction

Description of PFM

At the IDF reservoir pool elevation of 3039.8 feet, a tree alongside the chute falls into the channel causing the water surface profile in the chute to overtop the training walls, eroding the backfill and foundation soils supporting the training walls, thereby causing the training walls to overturn and release additional flow along the sides of the chute which then undermines the side slabs which subsequently collapse, leading to undermining of the bottom slab whose removal leads to undermining and erosion of the adjacent upstream slab sections. The progressive backward erosion and failure of slab sections continues upstream until reaching the spillway crest, whereupon the spillway cutoff wall is then undermined, leading to collapse of the spillway crest, and resulting in an unintended release of the reservoir.

Factors Discussed Relative to the Failure Mode

Likely/Adverse	Not Likely/Positive
• Large trees are present along the chute • The training walls are cantilever walls with spread footings founded on erodible soils • There is not continuous reinforcement between training wall segments	• Concrete cutoff walls below bottom slab exist every 25 feet along alignment • Cutoff wall at spillway crest is 22 feet deep

Categorization and Rationale for Assigned Category

The review team unanimously classified this PFM as Category II because large trees are present along the chute and overtopping of the training walls due to a fallen tree is physically possible, and therefore this PFM cannot be ruled out.

Potential risk-reduction measures

- Remove trees along the chute

4.1.3 Classification Summary

Of the five existing PFMs and the three new PFMs related to the spillway, one has been classified as Category I, five as Category II, and two as Category IV. The Category I PFM can be addressed by raising the top of the training wall profile above the IDF hydraulic profile by adding a concrete parapet.

5. Conclusions

5.1 Adequacy of Design and Current Condition

Based on the results of this assessment, we conclude that the Middle Fork Dam spillway is vulnerable to overtopping and potential failure during passage of the IDF. For discharges up to approximately 12,000 cfs, the spillway should provide acceptable performance, provided that the open joints in the slabs are sealed and areas of deteriorated concrete are repaired.

Specific findings and conclusions of the assessment are detailed in the following sections.

5.1.1 Geotechnical/Geological Finding Summary

The spillway is founded on weathered granitic bedrock whereas the training walls are founded on firm residual soils, with foundation keys cut 3 feet into stiff in-place native material. The spillway discharges onto exposed, scour-resistant bedrock.

The dam is in an area with moderate seismic activity and there are no faults underlying the spillway.

Based on the engineering review, inspection, and PFMA, the end of the spillway chute is not prone to undermining from scour of the plunge pool during future spill events.

5.1.2 Structural Assessment Finding Summary

Structural design calculations for the chute spillway or extensions added in 1989 were not available for this review; therefore, the factors of safety for stability of the training walls are unknown.

The detailed spillway inspection found numerous open and offset transverse joints and cracks in the bottom slab, in addition to minor scour along the entire length of the bottom slab that has exposed the concrete aggregate. None of the transverse cracks or joints noted had an adverse offset. There were also a few holes noted in the bottom or side slabs that were typically located at the junction of the side slab with the bottom slab. The bottom slab in the upstream half of the chute length was in better condition than that in the downstream half. From Station 4+50 to the end of the chute at approximately Station 6+20, the bottom slab exhibited several cracks and areas of surficial deterioration that justify reconstruction of this reach. Other items of note were a spalled area on the right side of the slope extension at Station 2+42, and a 2” positive offset at the top of the right side slab at Station 3+00.

Based on the engineering review, inspection, and PFMA, the presence in the spillway of open joints without waterstops poses a risk of introducing flow below the slabs that could exceed the capacity of the blanket drain system and create uplift on the slabs. For this reason, it is important to seal the open joints to prevent the inflow of water beneath the slabs.

5.1.3 Hydrology and Hydraulics Assessment Finding Summary

Based upon the hydrologic and hydraulic review, the PMF will overtop the embankment dam and dike by 1.1 feet if HMR 58/59 is judged to provide a better estimate of the PMP than HMR 36. Even if the lower PMF discharge of 13,220 cfs is passed through the spillway, the water surface profile in the chute will overtop the transition section and upper portion of the channel section by up to 2.1 feet. PFM 14 addresses failure of the spillway due to overtopping of the training walls, and was classified as Category I.

5.1.4 Operations Assessment Findings

As discharge to the Middle Fork Dam is not affected by upstream dams owing to the fact there is none, and the spillway operates without human involvement, there are no assessment findings regarding spillway operations.

5.1.5 Surveillance and Monitoring Assessment Findings

Based on the engineering review, inspection, and PFMA, the condition of the chute concrete and joints should be closely inspected following significant spill events to check for damage to the concrete surfaces, missing joint sealant, movement of joints or cracks, and seepage through joints or cracks.

5.2 Recommendations

There are three primary issues that should be addressed regarding the performance of the chute spillway: adequate freeboard during passage of the PMF, replacement of deteriorated concrete in the lower half of the chute length, and sealing of joints and cracks.

Detailed recommendations are described in the following sections. In general, immediate needs should be addressed at the next feasible opportunity, near-term actions should be implemented within the next one to two years, and longer-term actions should be implemented within the next two to five years. Operations and maintenance actions and inspection and monitoring items should be incorporated into their respective facility management programs.

5.2.1 Immediate Needs

No items requiring immediate attention were identified in this assessment.

5.2.2 Near-Term Actions

Seal the open joints and cracks with flexible sealant to prevent the inflow of water beneath the slabs.

Remove trees growing adjacent to the sides of the chute. Provide upslope swales to convey drainage downslope without flowing along the training walls.

Locate and review stability calculations for the training wall extensions added in 1989.

5.2.3 Longer-Term Actions

Replace the concrete comprising the bottom slab downstream of Station 4+50.

Add a concrete parapet to the transition section and upper portion of the channel section to provide adequate freeboard for the PMF hydraulic profile.

5.2.4 Maintenance and Operations Issues

Replace damaged or missing joint sealant when discovered.

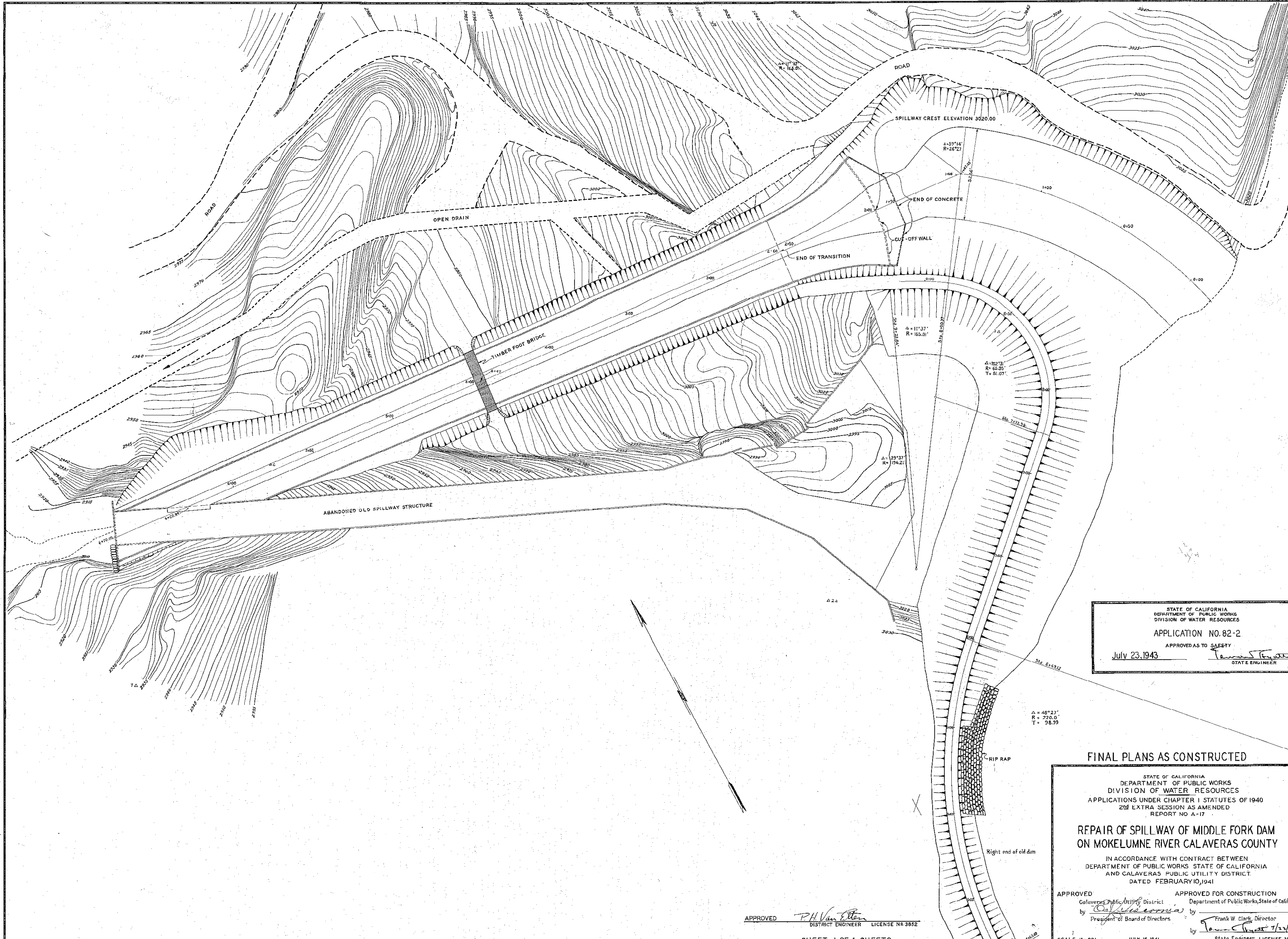
Repair areas of concrete deterioration as they occur.

5.2.5 Additional Monitoring or Inspection Required

Closely inspect the entire length of the spillway from outside and inside the chute following significant spill events to check for the presence of damage to the concrete surfaces, missing joint sealant, movement of joints or cracks, and seepage through joints or cracks.

Appendices

Appendix A – Key Spillway Design and Construction Documents



STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES

APPLICATION NO. 82-2

APPROVED AS TO SAFETY

July 23, 1943

STATE ENGINEER

FINAL PLANS AS CONSTRUCTED

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES
APPLICATIONS UNDER CHAPTER I STATUTES OF 1940
2nd EXTRA SESSION AS AMENDED
REPORT NO A-17

REPAIR OF SPILLWAY OF MIDDLE FORK DAM ON MOKELUMNE RIVER CALAVERAS COUNTY

IN ACCORDANCE WITH CONTRACT BETWEEN
DEPARTMENT OF PUBLIC WORKS STATE OF CALIFORNIA
AND CALAVERAS PUBLIC UTILITY DISTRICT
DATED FEBRUARY 10, 1941

APPROVED
Calaveras Public Utility District
by
President of Board of Directors

APPROVED FOR CONSTRUCTION
Department of Public Works, State of California
by
Frank W. Clark, Director
by 7/23/43
State Engineer LICENSE No 7

SCALE 1" = 20'
JULY 16, 1941

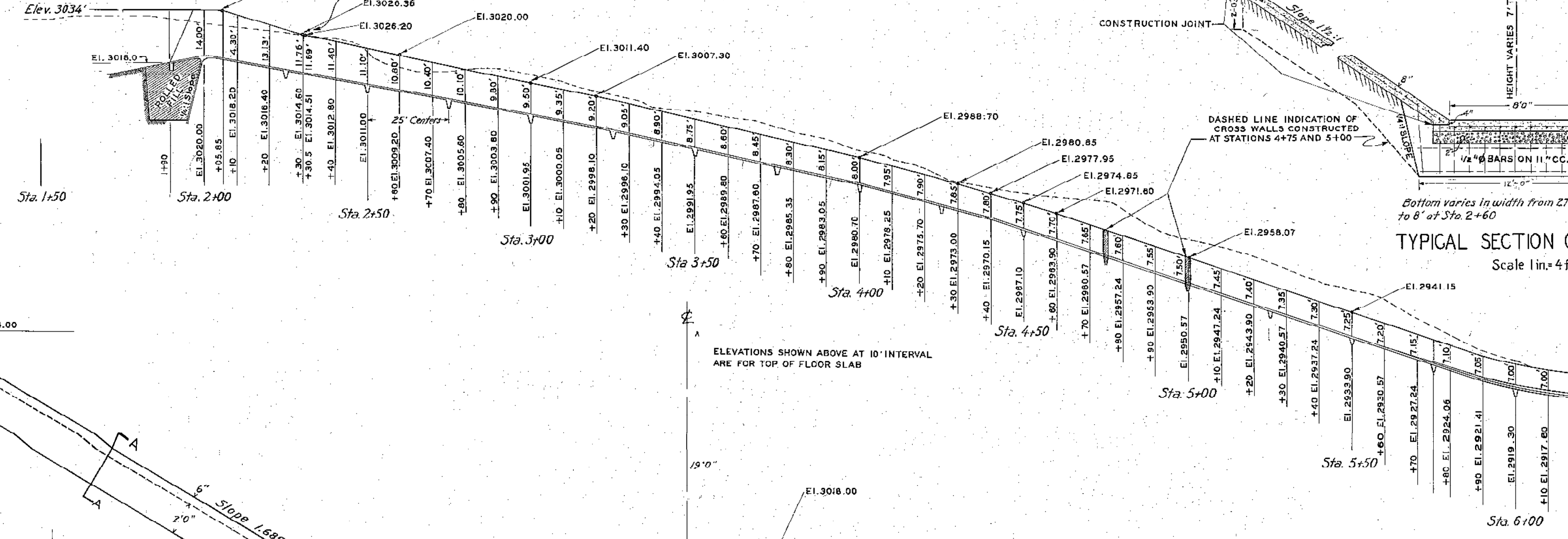
APPROVED P. H. Van Etten
DISTRICT ENGINEER LICENSE No 3852

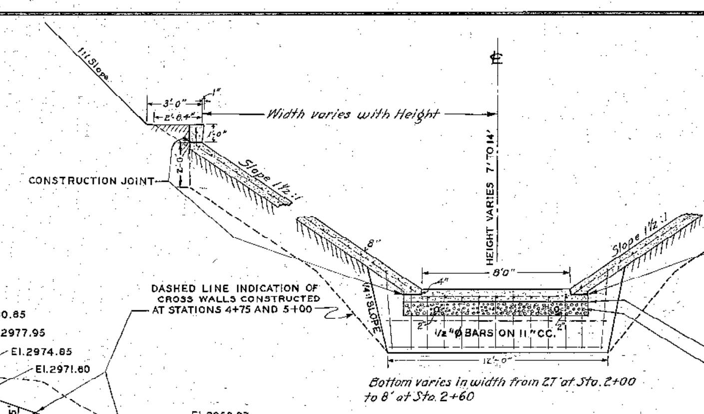
SHEET 1 OF 4 SHEETS

RECORD COPY

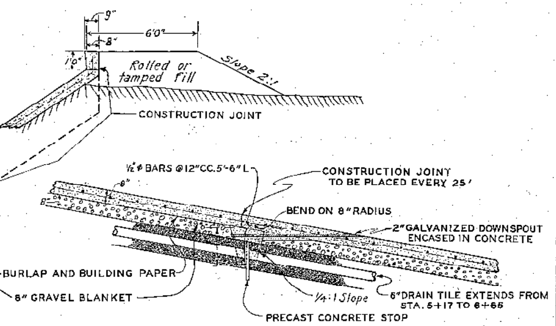
REVISED SEPT. 26, 1941
REVISED MAY 18, 1942
C.T.J.

PROFILE OF CHANNEL
Scale 1in.= 20 ft.

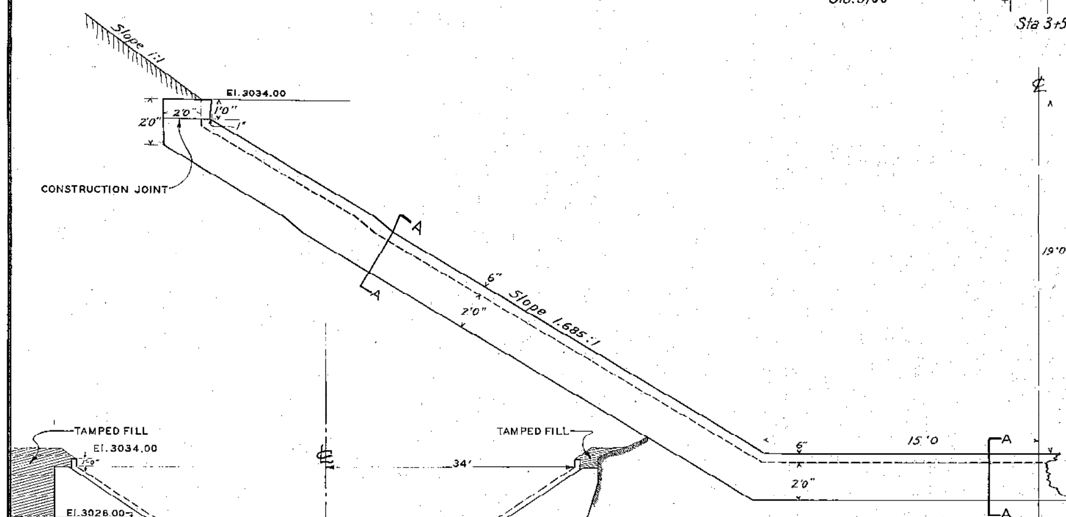




TYPICAL SECTION OF CHANNEL
Scale 1in.= 4 ft.

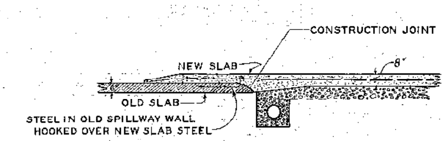


TYPICAL SECTION
SHOWING UNDERDRAINAGE AND CUT OFFS
Scale 1in.= 4 ft.

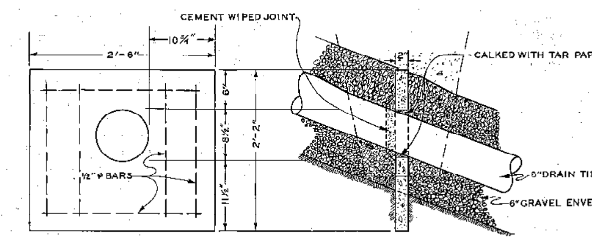


SECTION STA. 1+90
Scale 1in.= 4 ft.

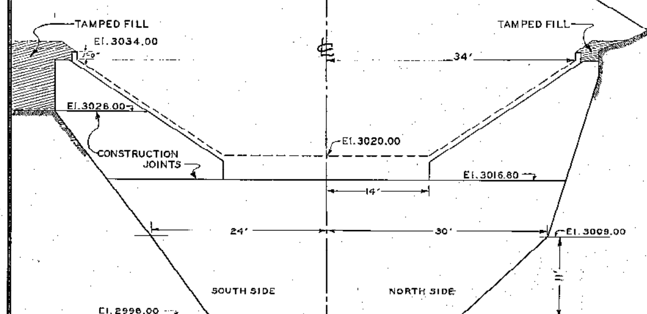
SECTION A-A
SCALE 1"= 4'



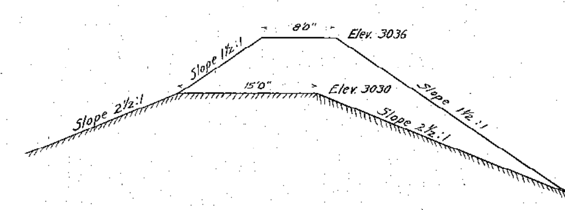
JUNCTION OF OLD SLAB WITH NEW
SCALE 1"= 4'



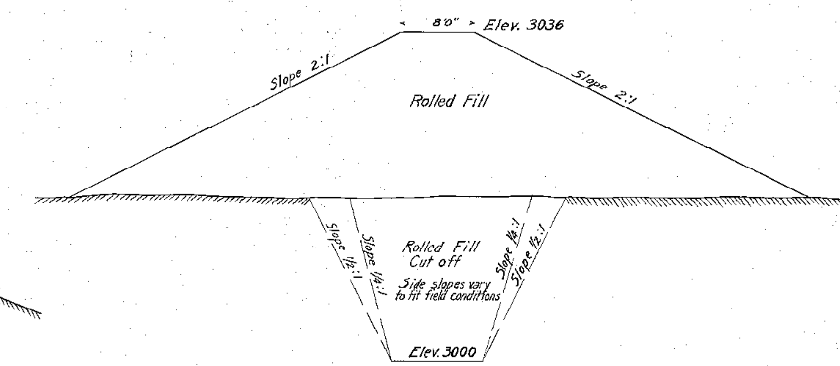
DETAIL AND LOCATION OF PRECAST CONCRETE STOP
USED AT CUTOFF WALLS STATION 5+25 TO STATION 6+00 INCL.
SCALE 1-INCH= 1-FOOT



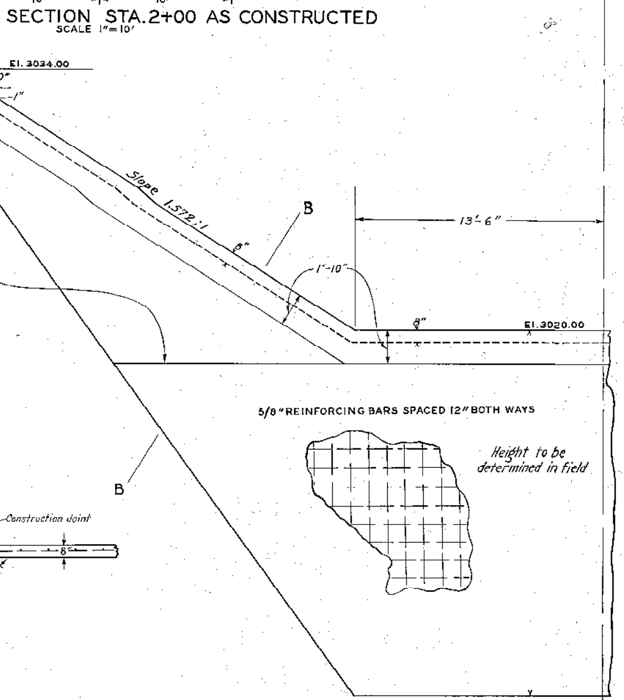
SECTION STA. 2+00 AS CONSTRUCTED
SCALE 1"= 10'

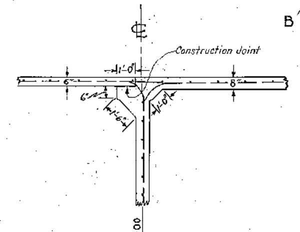


TYPICAL SECTION SHOWING RAISE ON OLD DAM
Scale 1in.= 8 ft.



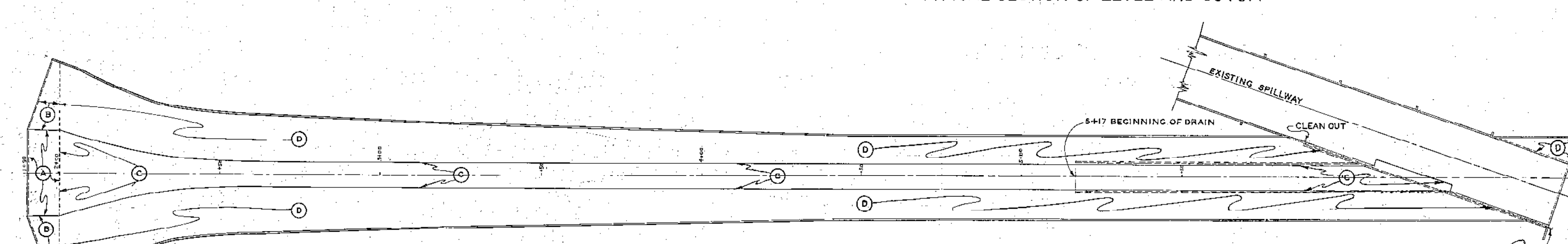
TYPICAL SECTION OF LEVEE AND CUTOFF
SCALE 1"= 8'





SECTION "B-B"
Scale 1in.= 4 ft.

SECTION STA. 2+00
SHOWING CUT OFF
Scale 1in.= 4 ft.



STEEL LAYOUT — CHANNEL LINING
SCALE 1"= 20'

TABLE OF REINFORCING STEEL BARS

	TRANSITION						SPILLWAY				
	LONGITUDINAL						LONGITUDINAL				
	STA.	STA.	LOCATION	SIZE	SPACED		STA.	STA	LOCATION	SIZE	SPACED
A	1+90	2+00	BOTTOM SLAB	5/8" Ø	12"	C	2+00	6+33	BOTTOM SLAB	3/4" Ø	11"
B	1+90	2+00	SIDE SLOPES	5/8" Ø	12"	D	2+00	6+70	SIDE SLOPES	5/8" Ø	10"
	TRANSVERSE										
	1+90	2+00	BOTTOM SLAB & SIDE SLOPES	1/2" Ø & Ø	18"						
							TRANSVERSE				
							2+00	6+70	BOTTOM SLAB & SIDE SLOPES	1/2" Ø & Ø	18"

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES

APPLICATION NO. 82-2

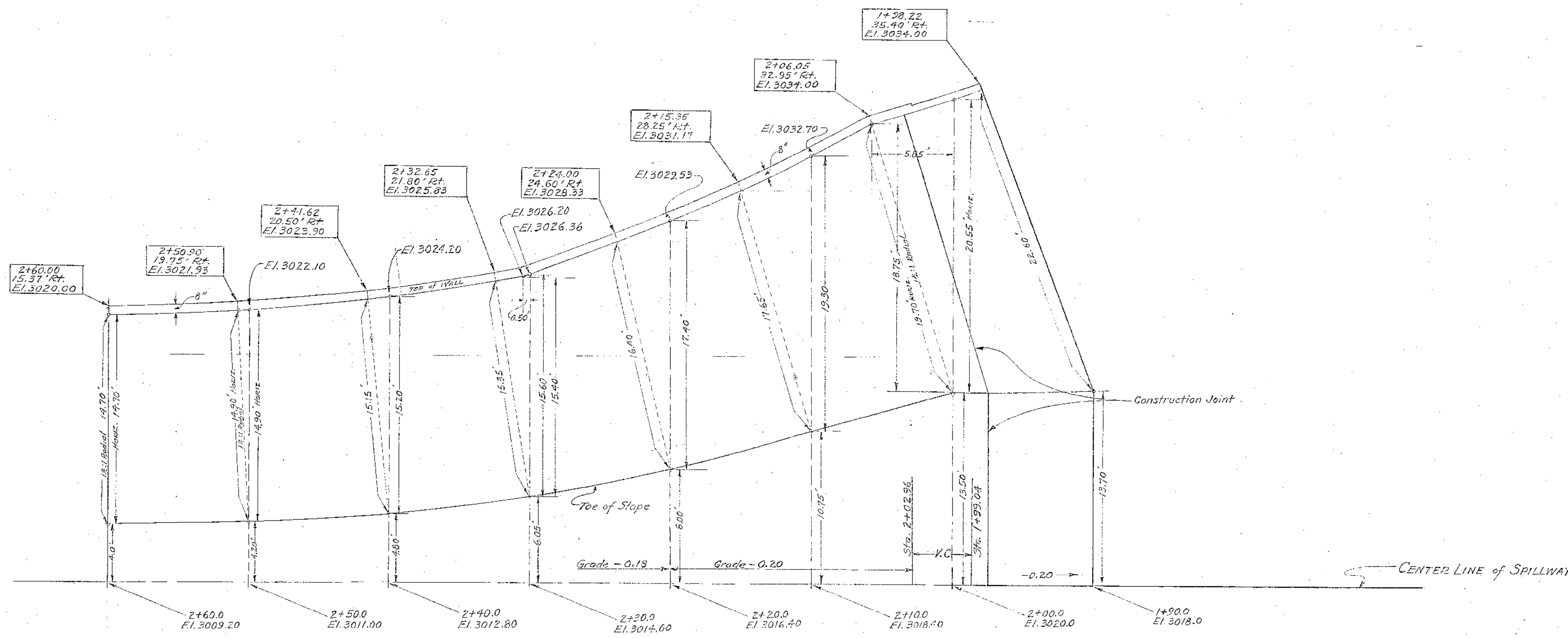
APPROVED AS TO SAFETY

July 23, 1943

STATE ENGINEER

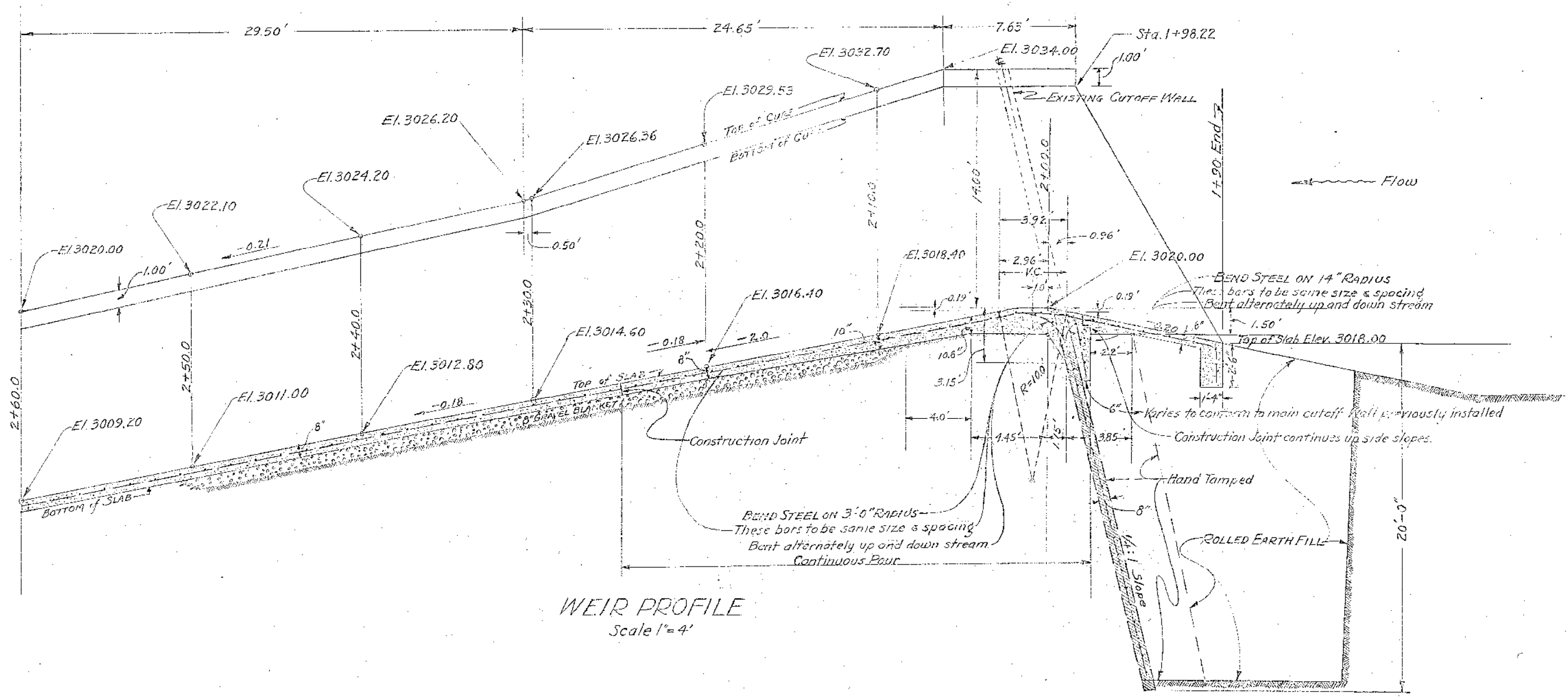
REPAIR OF SPILLWAY OF MIDDLE FORK DAM

SHEET 2 OF 4 SHEETS



HALF PLAN
Scale 1" = 4'

ALL ELEVATIONS SHOWN ARE FOR SURFACE OF CONCRETE
DEDUCT THICKNESS OF CONCRETE FOR GROUND ELEVATIONS



WEIR PROFILE
Scale 1" = 4'

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES

APPLICATION NO. 82-2

APPROVED AS TO SAFETY

July 23, 1943

STATE ENGINEER

REPAIR OF SPILLWAY OF MIDDLE FORK DAM

SHEET 3 OF 4 SHEETS

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES

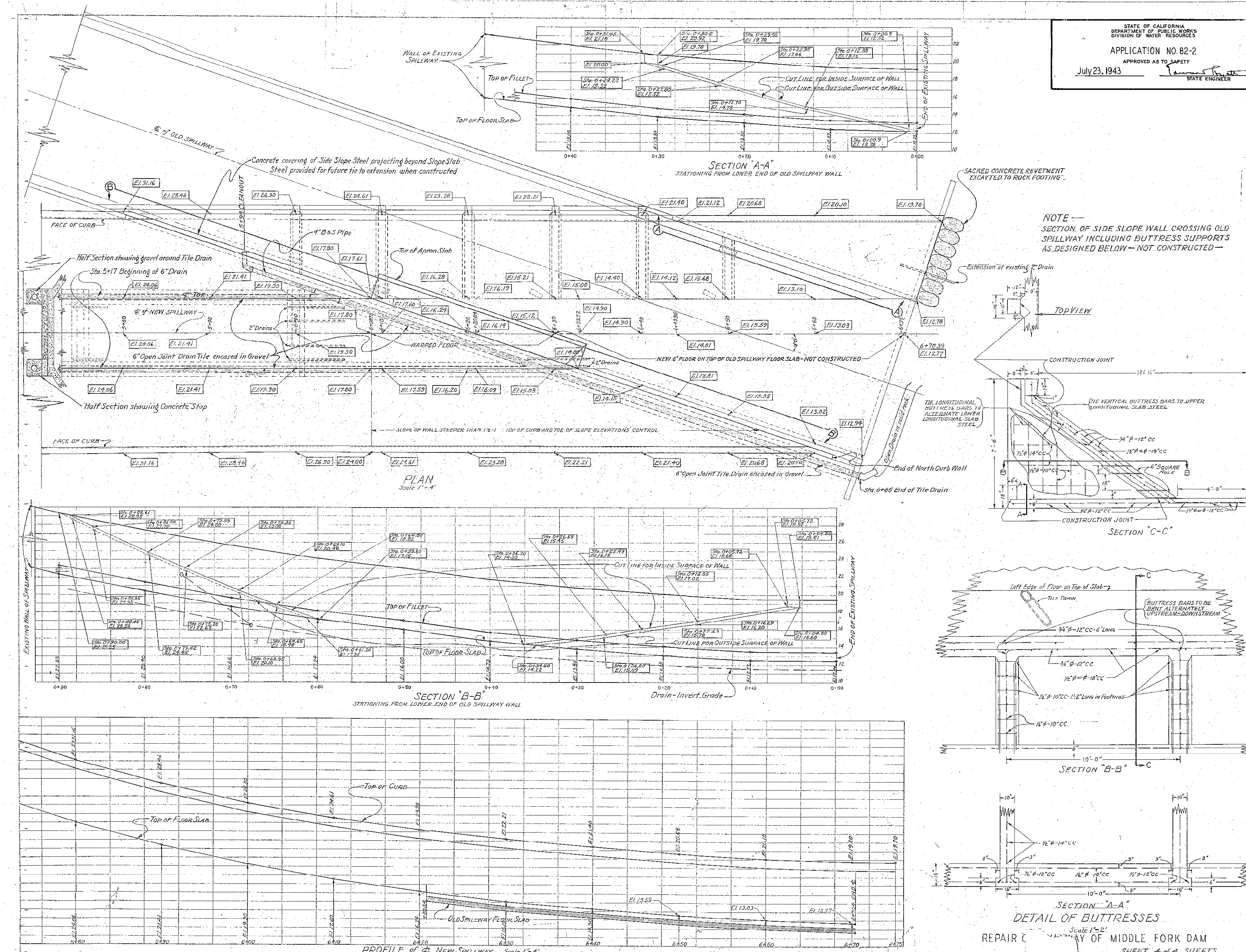
APPLICATION NO. 82-2

APPROVED AS TO SAFETY

July 23, 1943

STATE ENGINEER

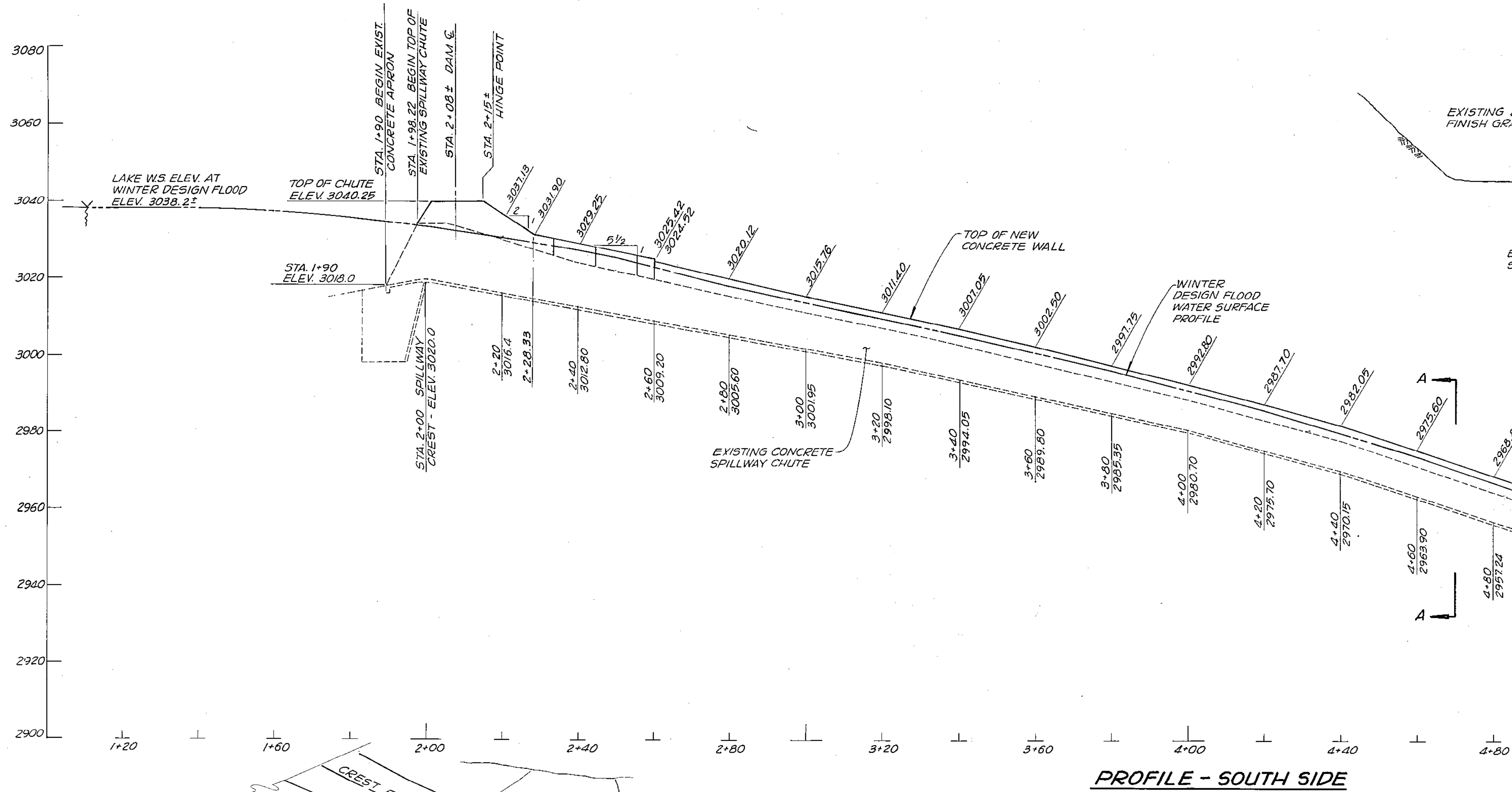
NOTE —
SECTION OF SIDE SLOPE WALL CROSSING OLD SPILLWAY INCLUDING BUTTRESS SUPPORTS AS DESIGNED BELOW — NOT CONSTRUCTED —



REPAIR OF SPILLWAY OF MIDDLE FORK DAM

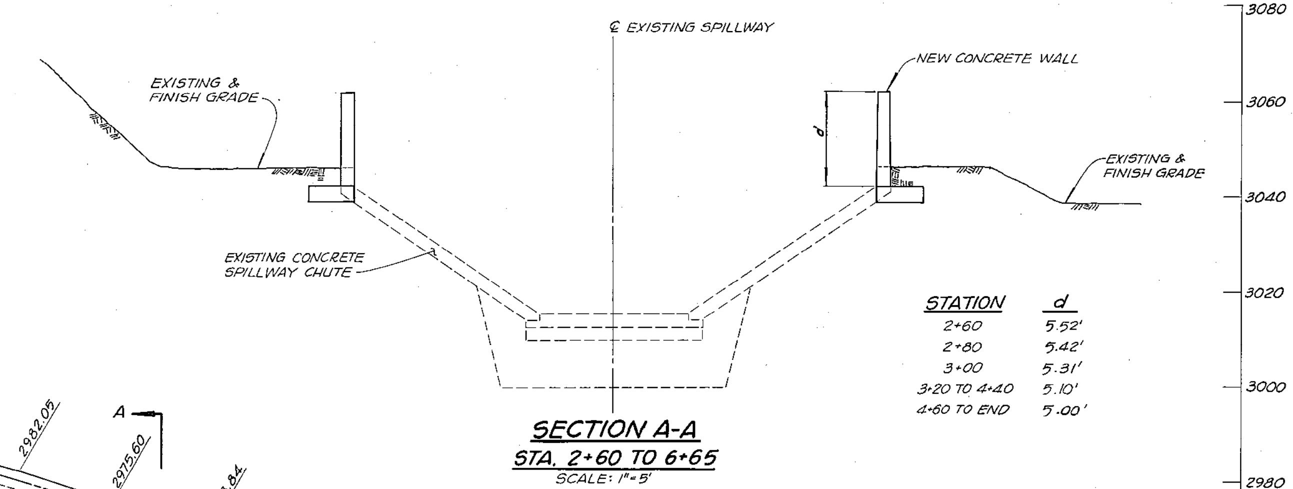
SHEET 4 of 4 SHEETS

RECORD COPY 82-2



PROFILE - SOUTH SIDE

SCALE: 1"= 20'

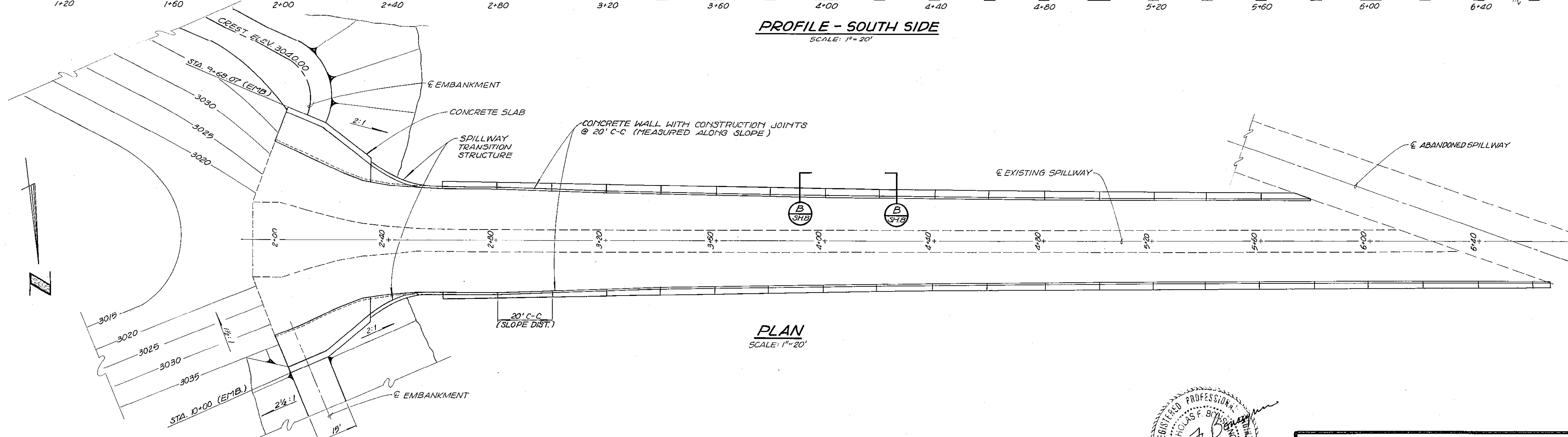


STATION	d
2+60	5.52'
2+80	5.42'
3+00	5.31'
3+20 TO 4+40	5.10'
4+60 TO END	5.00'

SECTION A-A

STA. 2+60 TO 6+65

SCALE: 1"= 5'



PLAN

SCALE: 1"= 20'

NOTE: STATIONING & EXISTING ELEVATIONS SHOWN PER ORIGINAL CONSTRUCTION DRAWINGS DATED 7-23-43.

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS F. BONSIGNORE
No. 39422
Exp. 12-31-93
CIVIL
STATE OF CALIFORNIA

2	AS-BUILT	1-18-91
0	ISSUED FOR CONSTRUCT.	12/87

CALAVERAS PUBLIC UTILITY DISTRICT

MIDDLE FORK DAM

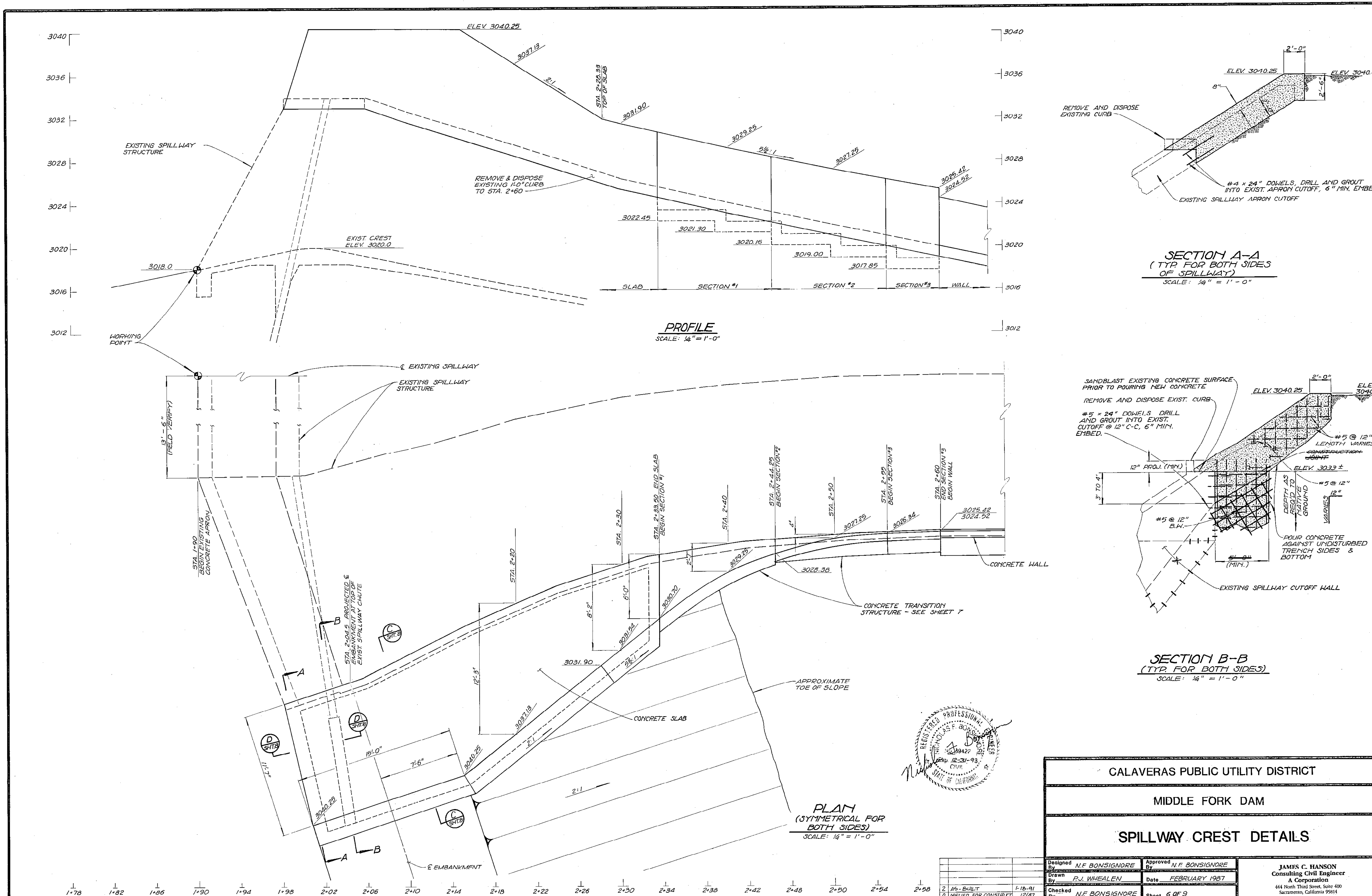
SPILLWAY MODIFICATION PLAN & PROFILE

Designed By N.F. BONSIGNORE	Approved By N.F. BONSIGNORE	JAMES C. HANSON Consulting Civil Engineer A Corporation 444 North Third Street, Suite 400 Sacramento, California 95814
Drawn By P.J. WHEALEN	Date JANUARY 1987	
Checked By N.F. BONSIGNORE	Sheet 5 OF 9	

MICROFILMED

AS BUILT

82-2



CALAVERAS PUBLIC UTILITY DISTRICT

MIDDLE FORK DAM

SPILLWAY CREST DETAILS

2	AS-BUILT	1-18-91	
0	ISSUED FOR CONSTRUCT	12/87	

Designed By N.F. BONSIGNORE — Approved By N.F. BONSIGNORE
Drawn By P.J. WHEALEN — Date FEBRUARY 1987
Checked By N.F. BONSIGNORE — Sheet 6 OF 9

JAMES C. HANSON
Consulting Civil Engineer
A Corporation
444 North Third Street, Suite 400
Sacramento, California 95814

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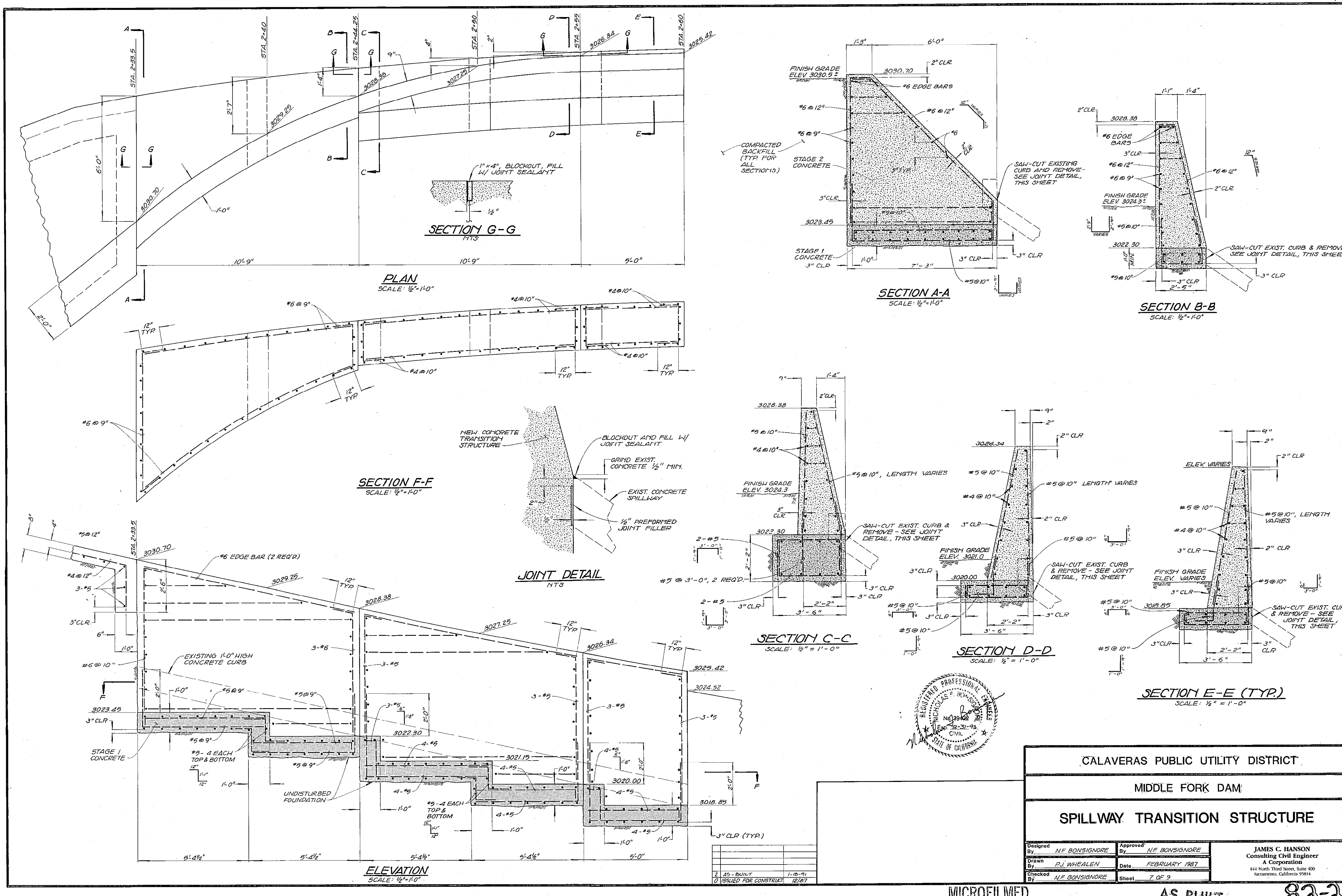
82-2



CALAVERAS PUBLIC UTILITY DISTRICT

MIDDLE FORK DAM

SPILLWAY TRANSITION STRUCTURE

Designed By N.F. BONSIGNORE	Approved By N.F. BONSIGNORE
Drawn By P.J. WHEALEN	Date FEBRUARY 1987
Checked By N.F. BONSIGNORE	Sheet 7 OF 9

JAMES C. HANSON
Consulting Civil Engineer
A Corporation
444 North Third Street, Suite 400
Sacramento, California 95814

2	AS-BUILT	1-18-91
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REGISTERED PROFESSIONAL ENGINEER NICHOLAS F. BONSIGNORE No. 39402 Exp. 12-31-93 CIVIL STATE OF CALIFORNIA

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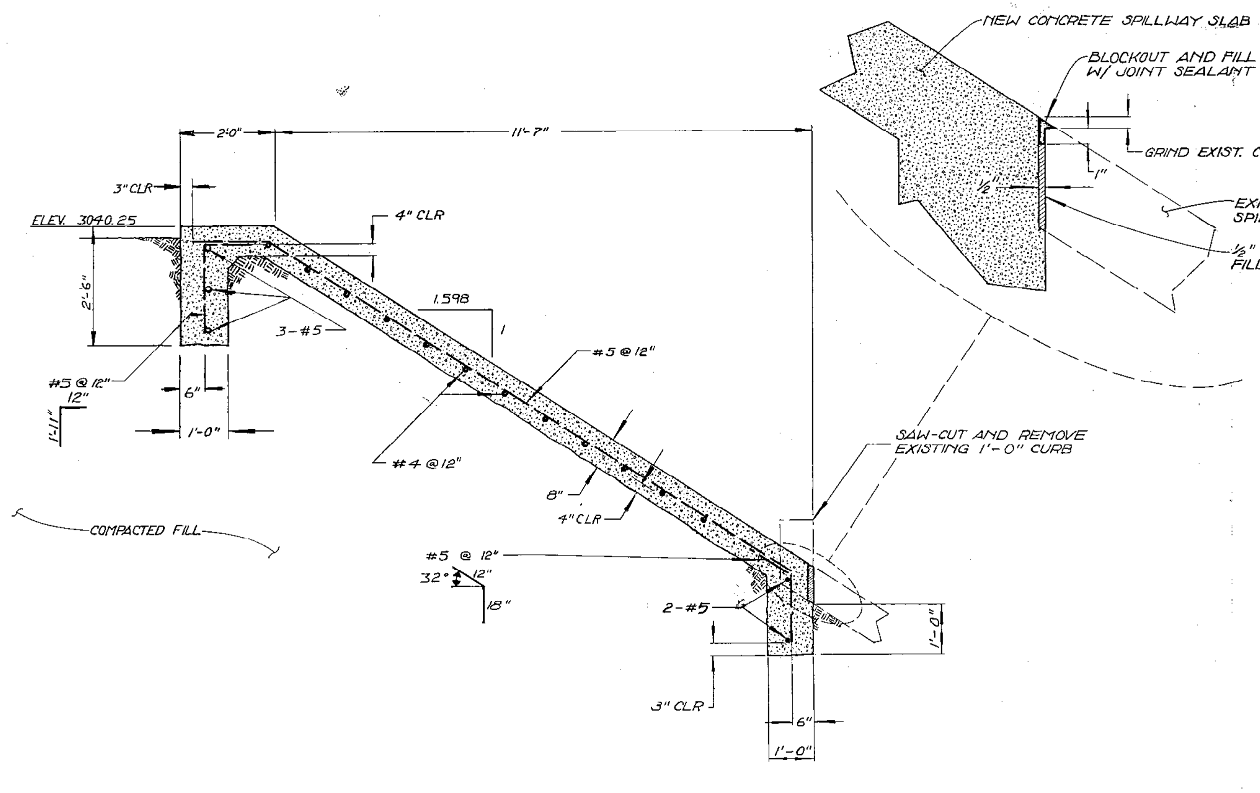
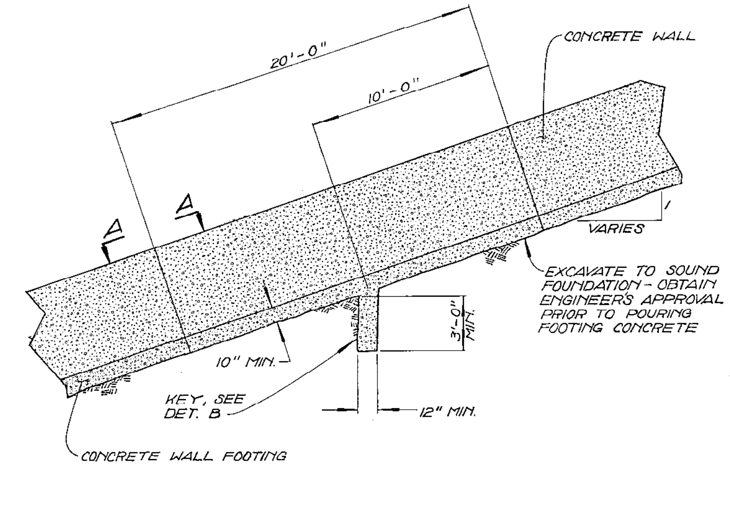


SECTION C-C (SHEET 6)

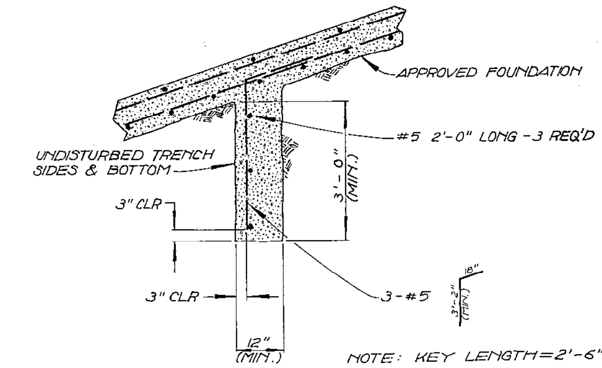
SCALE: ½" = 1'-0"



SECTION B-B (SHT. 5)

TYPICAL SPILLWAY WALL ELEVATION

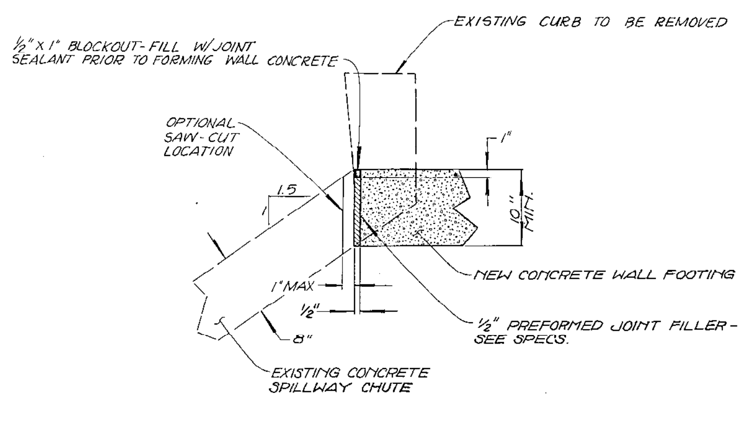
SCALE: 1" = 5'-0"



DETAIL B

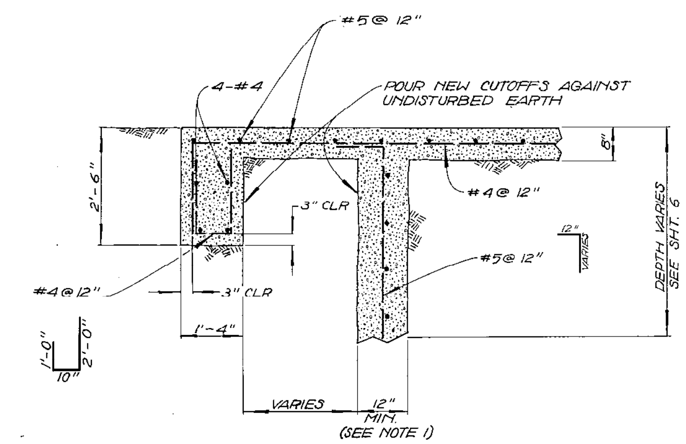
WALL FOOTING KEY

SCALE: ½" = 1'-0"



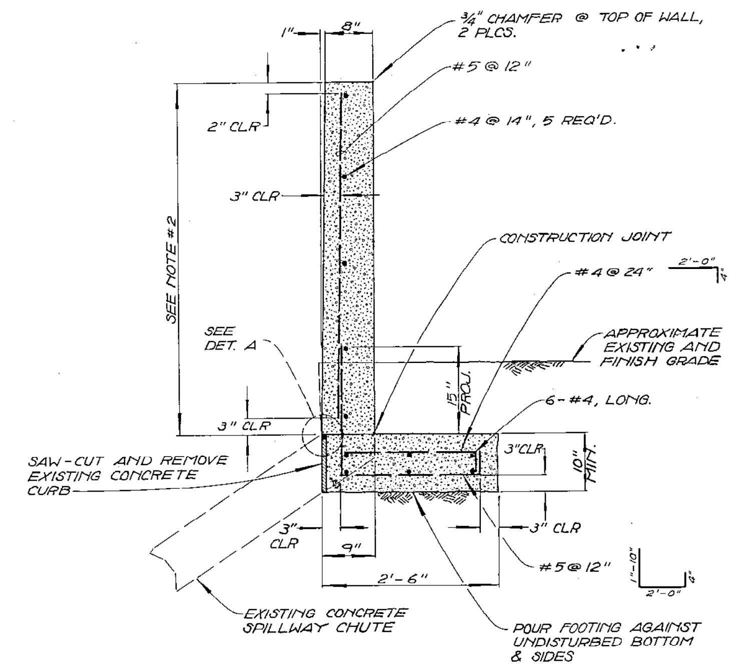
DETAIL A

SCALE: 1" = 1'-0"



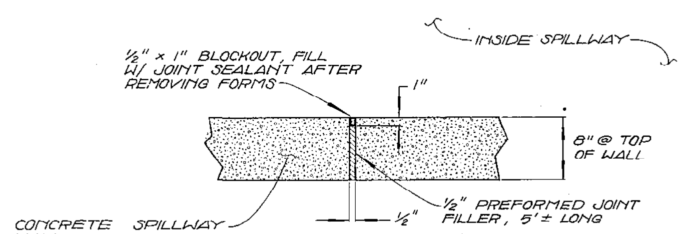
SECTION D-D (SHT. 6)

SCALE: ½" = 1'-0"



TYPICAL WALL SECTION

SCALE: ¾" = 1'-0"



SECTION A-A

NTS

NOTES

1. SPILLWAY CUTOFF MAY BE WIDER THAN 12" TO PERMIT ACCESS TO TRENCH.
2. WALL HEIGHT VARIES FROM 5'-0" TO 5'-6" ; SEE PROFILE AND SECTION A-A ON SHEET 5.

12-31-93

2	AS-BUILT	1-18-91
1	REVISED DETAIL 'A'	4/89
0	ISSUED FOR CONSTRUCT.	12/87

CALAVERAS PUBLIC UTILITY DISTRICT

MIDDLE FORK DAM

MISCELLANEOUS SPILLWAY SECTIONS & DETAILS

Designed By N.F. BONSIGNORE	Approved By N.F. BONSIGNORE	JAMES C. HANSON Consulting Civil Engineer A Corporation 444 North Third Street, Suite 400 Sacramento, California 95814
Drawn By J.A. KRAUSE	FEBRUARY 1987	
By N.F. BONSIGNORE	Sheet 8 OF 9	

MICROFILMED

AS BUILT

82-2

Appendix B – Detailed Spillway Inspection Notes

Title ~~Here~~ Middle Fork Dam Spillway
Date 9/10/18

2+25
55 **Joint 1** – no gap, tight, offset <1/4 = 1/8"

2+50 **Joint 2** –
62 66 Crack A – Patched left side, Transverse crack 10' - 10' - 6", previously repaired downstream
68 Crack B – Transverse crack 14', max open 3/8" not patched
69 Crack C – Transverse crack, just starting 18'6"

2+75 70 **Joint 3** – ¼" offset, drain right side gravel 16-20, ½"-1" 72
73 Crack A – Transverse crack – point repairs 11.5'

3+00 75 **Joint 4** – 1/4" pos offset, right side slope ~~slab~~ slab offset, even at invert, +2" at top opening = 1/4" 81
85 Crack A – ½ crack, barely a crack 15'6"
87 Crack B – Repair ¼" wide, 22'6"

3+25 88 **Joint 5** – 25 1/2 – opening 1/4 - 1/2", cavity right side 4" deep for 6" towards center 90
94 Crack A – Transverse crack 9'9"

3+50 95 **Joint 6** – ½" offset
98 Crack A – cracks at drains, 8'
99 Crack B – crack 13', previously patched, 1/8 - 1/2" openings

3+75 100 **Joint 7** – opening <5/8
101 Crack A – crack, barely 3 1/2' – 1/8" open
104 Crack B – Crack, 12' to 12'-6" – 1/8-1/4" open
105 Crack C – Crack, 1/2"-3/16" – 23'

4+00 106 **Joint 8** – 3/8 - 1/2"" offset, repair right side, spall upstream of Drain 107
112 Crack A – Crack 1/8 - 1/4" opening 12', hole left side 111
114 Crack B – crack 24-9

4+25 115 **Joint 9** – 1/2" offset, tight 1/8 - 1/4
117 Crack A – Crack, 6'-6", raveling surface tight, previously patched
119 Crack B – 8'6" between drains
121 Crack C – 12'6" right side previously patched raveling open 1/8", left side hole in ~~wingwall~~ side slab 123
122 Crack D – 25' repaired 1/16" opening

4+50 122 **Joint 10 -** 1/4" offset, 1/4-1/2 opening, previously repaired 124 bottom slab

125 Crack A – 4'-6" spalling at right side, tight

126 Crack B – Drains 6'-6" spalling, tight

127 Crack C – 8'-6" repaired, spalling

128 Crack D – 12' spalling, left center

Crack E – 18' tight

129 Crack F – 19'6", 1/8" opening, repaired, spalling, center repair, lots of surface damage

4+75 136 **Joint 11 –** repaired ½" offset, ¼-1/2 opening 145 bottom slab

137 Crack A – 5'3" above drains, tight, surface spalling

138 Crack B – 9'9" tight, raveling whole width

139 Crack C – 14' previously patched, spalling center 1/3, opening ¼ to ½, 1.5" deep, significant

140 Crack D – 17', tight raveling

141 Crack E – 24'9", 1/8" raveling, spalling 1" deep

5+00 149 **Joint 12 –** ¼" offset, ¼-1/2 opening, raveling

151 Crack A – 4'6" severe spalling, tight

152 Crack B – 10'9" raveling, ¼" crack

153 Crack C – 13', similar to B, spalling at 18 ½' 154

155 Crack D – 21'3" spalling full width to 2" deep, ¼" crack, 25' left side spalling 159

~~Crack E~~ 1/8" upstream at 13

5+25 160 **Joint 13 –** ¼ offset, 1/8" open, ravel center, spalling whole slab 161

166 ~~162~~ Crack A – 6'6" drain crack, significant spalling - spalling between cracks

166 ~~163~~ Crack B – 7'9" raveling at side

167 ~~165~~ Crack C – 13'9", 1/8" open, spalling1-½ deep

168 Crack D – 15'9" tight, spalling 1" deep - spalling 20' to 22' 169

170 Crack E – 24'6" previously repaired, tight, raveling 1.5" deep

5+50 171 **Joint 14 –** 1/8" with raveling, spalling starts at 5'6" full width to 2', 1"-2" deep 174

175 Crack A – 13' at ~~wing wall~~ side slab joint, repair middle also a hole in right side 5" deep 180

176 Crack B – 21'6" repaired middle, tight crack but deep spalling

177 Crack C – 24' tight, deep spalling, left side worse

5+75 178 **Joint 15 –** ¼" offset, 1/8 – ¼ open 194 bottom slab

182 Crack A – 4'6" tight spalling full width, 1.5" deep

184 Crack B – 8' (drain) raveling edged spall middle

188 Crack C – 13'6" repaired tight spalling right side, 1.5" deep

189 ~~Crack D~~ – Spall 22' center, 2 'x 1' x 1.5" deep

191 Crack D – 23'9" tight right spall 1" deep

6+00 192 **Joint 16 –** ½" offset, 1/8-1/4 opening, spalling downstream right, side. Heavy spalling from 2' to 16' up to 2" deep center and right side. 196

Crack A – 9'6" drains masked by spalling, right side spalling.

ATTACHMENT 3

Standard Professional Agreement

Professional Services Agreement

with

Calaveras Public Utility District

PO Box 666

San Andreas, CA 95249

Telephone 209-754-9442 Fax 209-754-9432

The terms on subsequent pages are incorporated in this document and will constitute a part of the agreement between the parties when signed.

To: Consultant

Phone: **Fax:**

Date: **Agreement No.**

Purchase Order No.

The undersigned Consultant offers to furnish the following: (scope of work)

Contract Price: **Not to exceed $________, as shown in Attachment A.**

Completion Date:

For Technical Direction by Consultant: Travis Small, General Manager, 506 W. St. Charles St., San Andreas, CA 95249, Travis.Small@CPUD.ORG, (209) 754-9442

Accepted: Calaveras Public Utility District	**Consultant:**
By: ________________________________	By: ______________________________
Travis Small	Name
General Manager	Title
Date: __________________________, 2024	Date: ________________________, 2024

Consultant agrees with Calaveras Public Utility District that:

a. **Hold-Harmless.** When the law establishes a professional standard of care for the Consultant's services, to the fullest extent permitted by law, Consultant will indemnify and hold harmless Calaveras Public Utility District, its directors, employees, and authorized volunteers from all claims and demands of all persons to the extent caused by the Consultant's negligence, recklessness, or willful misconduct in the performance (or actual or alleged non-performance) of the work under this agreement. Consultant shall defend itself against any and all liabilities, claims, losses, damages, and costs arising out of Consultant's negligent performance or non-performance of the work hereunder and shall not tender such claims to Calaveras Public Utility District nor to its directors, employees, or authorized volunteers, for defense or indemnity.

b. **Indemnification.** Other than in the performance of professional services, to the fullest extent permitted by law, Consultant will defend, indemnify and hold harmless Calaveras Public Utility District, its directors, employees and authorized volunteers from all claims and demands of all persons arising out the negligent or reckless performance of the work or furnishing of materials; including but not limited to, claims by the Consultant or Consultant's employees for damages to persons or property except to the extent caused by the negligence or willful misconduct or active negligence of Calaveras Public Utility District, its directors, employees, or authorized volunteers.

c. **Workers Compensation.** By his/her signature hereunder, Consultant certifies that he/she is aware of the provisions of Section 3700 of the California Labor Code which requires every employer to be insured against liability for workers' compensation or to undertake self-insurance in accordance with the provisions of that code, and that Consultant will comply with such provisions before commencing the performance of the professional services under this agreement. Consultant and sub-Consultants will keep workers' compensation insurance for their employees in effect during all work covered by this agreement. A sole proprietor exempt from the requirements to provide such coverage, with no employees or using no sub consultants, shall so certify on the form provided by the District.

d. **Professional Liability.** Consultant will file with Calaveras Public Utility District, before beginning professional services, a certificate of insurance satisfactory to the Calaveras Public Utility District evidencing professional liability coverage of not less than $1,000,000 per claim and annual aggregate, requiring 30 days' notice of cancellation (10 days for non-payment of premium) to Calaveras Public Utility District. Coverage is to be placed with a carrier with an A.M. Best rating of no less than A-: VII, or equivalent, or as otherwise approved by Calaveras Public Utility District. The retroactive date (if any) is to be no later than the effective date of this agreement. Consultant shall maintain such coverage continuously for a period of at least three years after the completion of the contract work. Consultant shall purchase a one-year extended reporting period i) if the retroactive date is advanced past the effective date of this Agreement; ii) if the policy is canceled or not renewed; or iii) if the policy is replaced by another claims-made policy with a retroactive date subsequent to the effective date of this Agreement. In the event that the Consultant employs other consultants (sub-consultants) as part of the work covered by this agreement, it shall be the Consultant's responsibility to require and confirm that each sub-consultant meets the minimum insurance requirements specified above.

e. **General Liability.** Consultant will file with Calaveras Public Utility District, before beginning professional services, certificates of insurance satisfactory to Calaveras Public Utility District evidencing general liability coverage of not less than $1,000,000 per occurrence ($2,000,000 general and products-completed operations aggregate (if used)) for bodily injury, personal injury and property damage; auto liability of at least $1,000,000 for bodily injury and property damage each accident limit; workers' compensation (statutory limits) and employer's liability ($1,000,000) (if applicable); requiring 30 days (10 days for non-payment of premium) notice of cancellation to Calaveras Public Utility District. The general liability coverage is to state or be endorsed to state "such insurance shall be primary, and any insurance, self-insurance or other coverage maintained by Calaveras Public Utility District, its directors, officers, employees, or authorized volunteers shall not contribute to it". The general liability coverage shall give Calaveras Public Utility District, its directors, officers, employees, and authorized volunteers additional insured status using ISO endorsement CG2010, CG2033, or equivalent. Coverage is to be placed with a carrier with an A.M. Best rating of no less than A- :VII, or equivalent, or as otherwise approved by Calaveras Public Utility District. In the event that the Consultant employs other consultants (sub-consultants) as part of the work covered by this agreement, it shall be the Consultant's responsibility to require and confirm that each sub-consultant meets the minimum insurance requirements specified above.

f. **Insurance Notification**. If any of the required coverages expire during the term of this agreement, the Consultant shall deliver the renewal certificate(s) including the general liability additional insured endorsement to Calaveras Public Utility District at least ten (10) days prior to the expiration date.

g. **Direction/Orders**. Consultant shall not accept direction or orders from any person other than the General Manager or the person(s) whose name(s) is (are) inserted on Page 1 as "other authorized representative(s)," subject to the limitations of paragraph "Changes", below. An Amendment to this Agreement will be issued in writing, incorporating Consultant's scope and mutually agreed-upon price and estimated schedule for completion. A fully executed Revised Purchase Order incorporating the additional/changed scope and price, shall also be issued, with a copy provided to Consultant.

h. **Invoices**. Consultant shall submit to the District monthly invoices for time and expenses subject to the contract limitation. Invoices shall reference the Purchase Order and project number shown on the purchase order form. Each invoice shall also include the total invoiced and paid to date, and the remainder outstanding. Invoices received without this information shall be returned to Consultant unpaid, for revision and re-submittal. Invoices shall be submitted to:

Calaveras Public Utility District

PO Box 666

San Andreas, CA 95249

i. **Payment.** Payment, unless otherwise specified, is to be 30 days after receipt of an invoice deemed acceptable in accordance with paragraph h., above, by Calaveras Public Utility District and its acceptance in meeting the criteria of this Agreement.

j. **Permits**. Permits required by governmental authorities will be obtained at Consultant's expense, and Consultant will comply with applicable local, state, and federal regulations and statutes including Cal/OSHA requirements.

k. **Changes**. Any change in the scope of the professional services to be done, method of performance, nature of materials or price thereof, or to any other matter materially affecting the performance or nature of the professional services will not be paid for or accepted unless such change, addition or deletion is approved in advance, in writing by an Agreement Amendment executed by the General Manager of Calaveras Public Utility District.

l. **Progress of Work**. Consultant shall perform the professional services promptly, diligently and in such manner and sequence as to assure the timely completion of other work dependent thereon and to permit completion of the professional services in a manner to ensure the work is completed on or before the Completion Date set forth above ("Schedule Requirements"). In this regard, Consultant shall at all times furnish and have available such sufficient and satisfactory equipment, materials, supplies and workers to perform the professional services in a prompt and timely manner in accordance with the timelines of this Agreement. In the event Consultant fails to perform the professional services in accordance with the Schedule Requirements, Consultant, at its own expense, shall provide additional equipment, work force, overtime or additional shifts so as to meet and maintain the Schedule Requirements. Consultant will pay all expenses and damages incurred by Owner resulting from the failure of Consultant to meet the Schedule Requirements, or abide by Contractor's instructions with regard to the Schedule Requirements, to Owner upon demand.

m. **Assignment**. Consultant shall not assign, delegate, sublet, or transfer any interest in or duty under this Agreement without the express prior written consent of the Calaveras Public Utility District.

n. **Termination.** District may terminate this Agreement with ten (10) days prior written notice to Consultant and identifying the Consultant's final work date. In the case of such termination Consultant shall provide the Calaveras Public Utility District a final invoice for work performed and expenses incurred prior to termination within 30 calendar days following the final work date provided in the notice of termination. No additional invoices will be accepted, nor charges paid by the Calaveras Public Utility District after this 30-day final invoicing period.

o. **Products**. All work products resulting from this Agreement, including documents and reports, drawings, models, specifications, computer drawings and other electronic expression, and the like that may be drafted, assembled, compiled, or obtained by Consultant during the performance of assigned tasks, and delivered to the Calaveras Public Utility District as Consultant's work product shall be the property of the Calaveras Public Utility District for its exclusive use. Except as may be distributed in its original form, any modification or other reuse of such work product for purposes other than those intended by this Agreement shall be at the Calaveras Public Utility District's sole risk and without liability to Consultant.

p. **Provided Information.** Calaveras Public Utility District shall furnish the Consultant with associated drawings (plan and section) of associated equipment and/or infrastructure as necessary.

q. **Third Parties.** The services to be performed by Consultant are intended solely for the benefit of the Calaveras Public Utility District. No person or entity not a signatory to this Agreement shall be entitled to rely on the Consultant's performance of its services hereunder, and no right to assert a claim against the Consultant by assignment of indemnity rights or otherwise shall accrue to a third party as a result of this Agreement or the performance of the Consultant's services hereunder. Notwithstanding the foregoing Consultant understands and agrees that Calaveras Public Utility District will be submitting the report to various State and/or Federal agencies for their review. Consultant agrees that the agencies receiving the report may and will rely on its accuracy. Moreover, this section in no way impairs Calaveras Public Utility District's rights to indemnity from Consultant as provided in this agreement, including any claims by third parties.

r. **Access to Records**. Consultant shall provide access to the Federal grantor agency, the Comptroller General of the United States, or any of their duly authorized representatives to any books, documents, papers, and records of the contractor which are directly pertinent to that specific contract for the purpose of making audit, examination, excerpts, and transcriptions.

s. **Record Retention.** Consultant shall retain all required records for three years after the Calaveras Public Utility District makes final payments and all other pending matters are closed.

t. **Modification**. No waiver, amendment or modification of any term, provision, condition or covenant of this Agreement shall be effective unless set forth in writing, signed by the Parties hereto, and which specifically identifies such waiver, amendment or modification. Such waiver, amendment or modification shall be effective only to the extent identified in such writing.

u. **Independent Contractor Relationship.** Consultant is and shall be an independent contractor of the District. Neither Consultant nor Consultant's employees shall be deemed to be employees or agents of the District. Nothing in this Agreement is intended to establish a partnership, joint venture, or agency relationship between the parties, and neither Consultant nor Consultant's employees are authorized to bind the District or make any representations on its behalf in any matter.

v. **Electronic Signatures.** All parties agree to conduct this transaction electronically and use scanned or electronic signatures in accepting and conveying this agreement electronically by email or other electronic means, and therefore both parties acknowledge this agreement is accorded legal effect and binding on both parties.

* * *