FINAL REPORT

LAGUNA DEL CAMPO DAM
OSE FILING NO. D313

BREACH ANALYSIS REPORT

RIO ARRIBA COUNTY, NEW MEXICO

Prepared for:

New Mexico Department of Game and Fish
One Wildlife Way
Santa Fe, NM 87507

Under contract with:

U.S. Fish and Wildlife Service
4401 N. Fairfax Drive
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List of Abbreviations

%	Percent
ac-ft	Acre feet
cfs	Cubic feet per second
DEM	Digital elevation model
EAP	Emergency Action Plan
FEMA	Federal Emergency Management Agency
ft	Foot or feet
GIS	Geographic Information System
HEC-HMS	Hydraulic Engineering Center – Hydrologic Modeling System
HMR	Hydrometeorological Report
hr	Hour
in	Inch
L	Length of longest watercourse from point of concentration to boundary of drainage watershed
Lca	Distance along L from the dam to a point nearest the centroid of the watershed
Lg	Watershed lag time
mi	Mile
min	Minute
NID	National Inventory of Dams
NMDGF	New Mexico Department of Game and Fish
NOAA	National Oceanic and Atmospheric Administration
OSE	New Mexico Office of the State Engineer
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
IDF	Inflow Design Flood
Reclamation	United States Department of the Interior, Bureau of Reclamation
S	Overall channel slope of watershed
sq mi or mi^2	Square mile
URS	URS Group, Inc.
USACE	United States Army Corps of Engineers
USDA	United States Department of Agriculture
USGS	United States Geological Survey
USWB	United States Weather Bureau
WPA	Works Progress Administration
WSEL	Water Surface Elevation

FINAL REPORT
ENGINEER'S CERTIFICATE

LAGUNA DEL CAMPO DAM
OSE FILING NO. D313

BREACH ANALYSIS REPORT

RIO ARRIBA COUNTY, NEW MEXICO

State of Colorado

City and County of Denver

I, Bradley W. Rastall, hereby state that I am a professional engineer licensed in the state of New Mexico, qualified in civil engineering; that the accompanying Breach Analysis Report for Laguna Del Campo Dam was prepared by me or under my supervision; that the accompanying Breach Analysis Report for Laguna Del Campo Dam is in compliance with the Dam Design, Construction and Dam Safety Regulations (19.25.12 NMAC) and that the same are true and correct to the best of my knowledge and belief.

License Number: 19161

Date Submitted 2/8/2012

BRADLEY WILLIAM RASTALL
NEW MEXICO
19161
PROFESSIONAL ENGINEER

FINAL REPORT
STATE ENGINEER'S CERTIFICATE

LAGUNA DEL CAMPO DAM
OSE FILING NO. D313

BREACH ANALYSIS REPORT

RIO ARRIBA COUNTY, NEW MEXICO

State of New Mexico

County of Santa Fe

I hereby certify that the accompanying Breach Analysis Report for Laguna Del Campo Dam and its appurtenant structures has been duly examined by me and accepted for filing on the 26th day of OCTOBER, 2012.

Charles T. Thompson

FOR State Engineer

1.1 INTRODUCTION

Laguna Del Campo Dam (Laguna Dam) is located in north central New Mexico, approximately 2 miles northwest of Tierra Amarilla, New Mexico. The dam and reservoir are located on the left overbank area of the Rio Chama, northwest of the intersection of U.S. Highway 84 and State Highway 112 in Rio Arriba County, New Mexico. The Parkview Ditch drainage basin is located on the western slope of the Peňasco Amarillo and contributes runoff to the Laguna Del Campo Reservoir (Laguna Reservoir). Runoff from Laguna Del Campo Dam flows southwest for about 2,500 ft to Rio Chama. The latitude and longitude of Laguna Del Campo Dam are 36° 42' 49" N and 106° 35' 2" W, respectively. Construction of the dam was completed in 1940. **Figure 1-1** and **Figure 1-2** present a general location map and a vicinity map of Laguna Del Campo Dam, respectively.

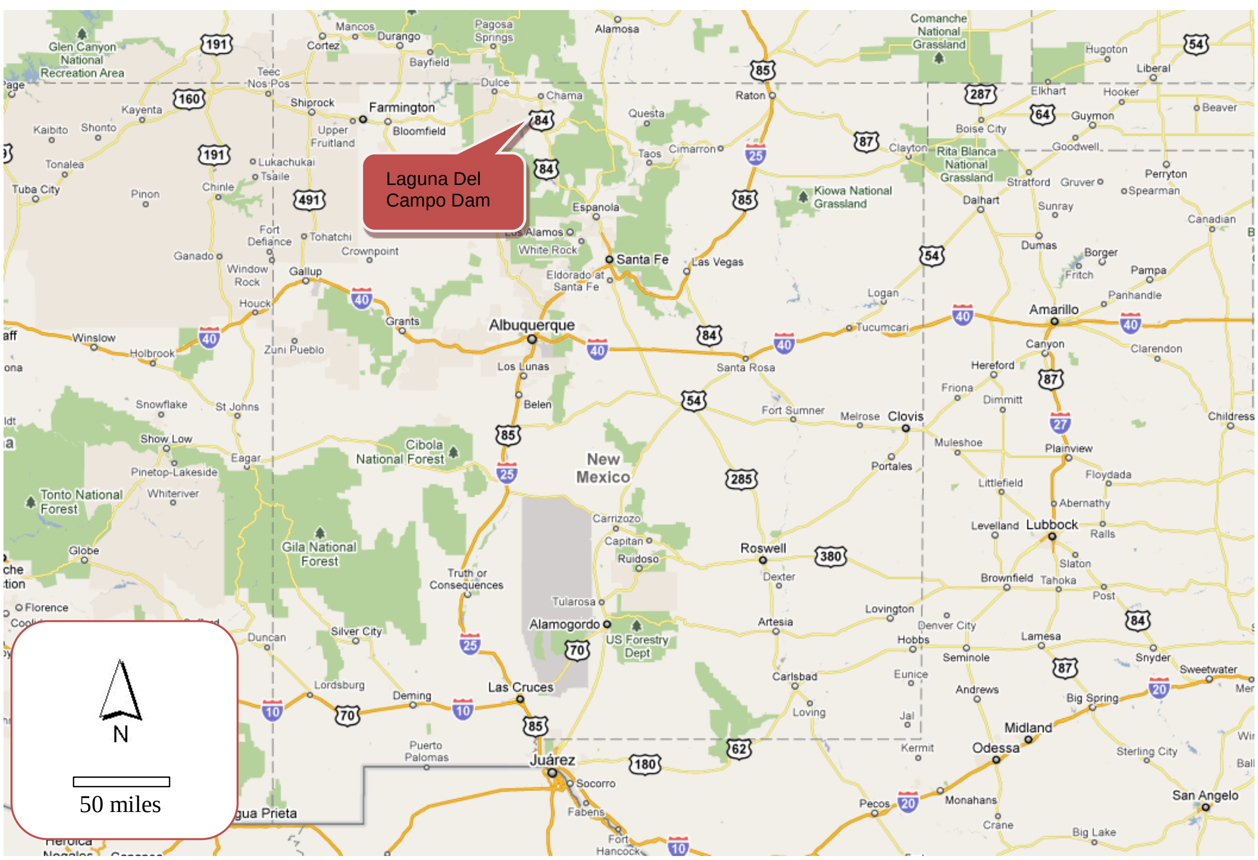

Figure 1-1
General Location Map of Laguna Del Campo Dam

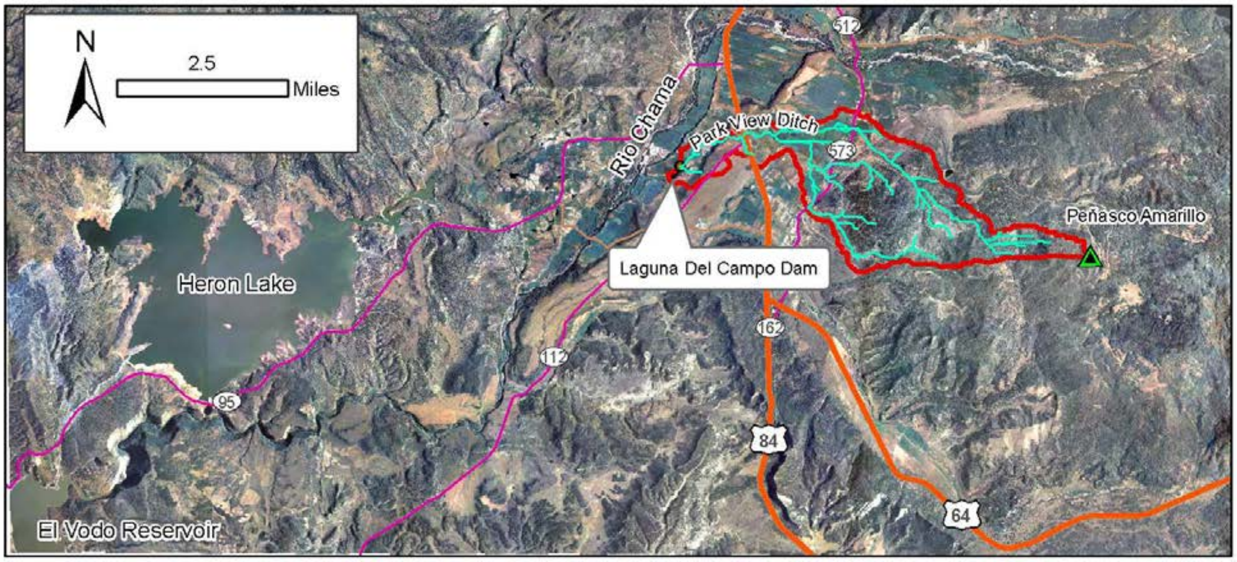

Figure 1-2
Vicinity Map of Laguna Del Campo Dam

The dam is owned by the New Mexico Department of Game and Fish and operated by the Los Ojos State Hatchery. The dam NID is NM00313, or D-313 for the New Mexico Office of the State Engineer (OSE). Repair work was performed on the dam in the fall of 1979, which included repair of the concrete spillway, and removal of woody growth on the embankment of the dam, in the spillway channel, and in the outlet works discharge channel. Currently, regrowth of willows and woody type vegetation is occurring on the dam embankment slopes and spillway. According to a 2009 OSE dam inspection report, the dam is in “Poor” condition due to poor maintenance of the dam and spillway, spillway deficiencies, and the lack of design and construction reports and as-built drawings documenting design and construction of the dam. Laguna Del Campo Dam is classified as a small sized, high hazard dam. In the absence of a hazard classification study for the dam, the OSE assumed a high hazard classification based on the potentially occupied structures shown on the inundation maps developed herein. The OSE classifies high hazard dams as “those dams where failure or misoperation will probably cause loss of human life” (OSE, 2010).

Based on 1937, 1938 and 1979 design drawings, the dam is an earthen embankment and consists of a main dam and a north dike. The main dam is approximately 36 ft high at the maximum section, with 3H:1V (Horizontal:Vertical) and 2H:1V side slopes on the upstream and downstream slopes of the dam, respectively. The north dike was assumed to have the same crest elevation as the main embankment based on the design drawings and a December 2010 site visit by URS Group, Inc. The 1937, 1938 and 1979 design drawings are included in **Appendix A**. The outlet works consists of a concrete intake structure with a slide gate, and an approximately 150-ft long, 2 ft by 2 ft concrete outlet conduit. Control is provided by an outlet gate operator which is mounted on a corrugated metal pipe (CMP) riser and is accessible only by boat. According to the 2009 OSE dam inspection report, the gate is normally kept closed; therefore, only the emergency spillway will operate during flood events.

The emergency spillway is located at the left (south) abutment of the dam. The spillway includes a weir that was reconstructed as a compound weir in 1979 with a 4-ft wide by 0.6-ft high low flow notch at the center of the spillway weir. Upstream from the weir, the spillway approach channel is approximately 50 ft long, and is lined with concrete. Downstream from the weir, the spillway discharge channel is lined with concrete for about 50 ft. The spillway channel side slopes are nearly vertical with concrete retaining walls upstream and downstream of the weir. The current spillway capacity is 1185 cfs with the reservoir at the dam crest (El. 104).

Pertinent data is presented in **Table 1-1**. This information was taken from the 1937 and 1938 design drawings and the 1979 emergency spillway repair design drawings. These drawings are based on an assumed vertical datum with elevation 104 ft. corresponding to the dam crest. The 1938 drawing shows that the north dike crest is at the same elevation as dam crest, which agrees with the observations made during the December 2010 site visit by URS Group, Inc. Drawings presented in **Appendix A** show the general configuration of the dam and its appurtenant structures. **Appendix B** presents photographs of the facilities taken during a December 2010 site visit by URS Group, Inc.

Table 1-1
Summary of Pertinent Data

Feature	Pertinent Data
Dam Crest Elevation, ft(1)	104
North Dike Crest Elevation, ft	104
Dam Crest Length, ft (Main Dam/North Dike) (1/2)	500/1030
Dam Crest Width, ft(1)	13
Dam Height, ft(1)	36
Hydraulic Height, ft(3)	30
Upstream Embankment Slope (H:1V) (1)	3:1
Downstream Embankment Slope (H:1V) (1)	2:1
Emergency Spillway Crest Elevation, ft (at weir crest) (4)	98.75
Emergency Spillway Crest Length, ft(4)	28
Outlet Works, Outlet Invert Elevation, ft(2)	73
Irrigation Pool/Normal Storage Capacity (Emergency Spillway Crest)	Elev.: 98.75 ft Storage: 99.6 ac-ft
Maximum Storage Capacity (Dam Crest)	Elev.: 104 ft Storage: 177.5 ac-ft

Notes: (1) Information obtained from 1937 design drawing. Elevations are based on an assumed vertical datum with elevation 104 ft. corresponding to the dam crest.
(2) Information obtained from 1938 design drawing.
(3) Hydraulic height was calculated from downstream outlet center line elevation, El. 74 ft, to the dam crest, El. 104 ft.
(4) Emergency spillway width is shown as 30 feet on the 1979 Design Drawing, and as 28 feet on the 1937 Design Drawing. For the purposes of developing EAP inundation maps the narrower dimension was conservatively used.

1.2 PURPOSE

The purpose of this report is to present the assumptions, criteria, and calculations for the inflow flood hydrology and breach analysis with subsequent dam failure flood routing at Laguna Del Campo Dam for the development of inundation maps for the Emergency Action Plan (EAP). Based on the OSE Rules and Regulations Governing Dam Design, Construction and Dam Safety (OSE, 2010), the inundation maps should include the sunny day failure limits as well as the limits from failure at the high water line. The high water line is defined as the reservoir water elevation during the Inflow Design Flood (IDF). Laguna Del Campo Dam is a small size, high hazard dam. Based on OSE Rules (2010), the required IDF for Laguna Del Campo Dam is 100% of the Probable Maximum Precipitation (PMP). The OSE Rules allow a third inundation limit to be shown, as determined by the dam owner. Therefore, three dam breach scenarios were analyzed in this study; the sunny day breach, 50% PMP breach, and 100% PMP breach.

The scope of this dam breach analysis included developing the inflow floods resulting from the 50% and 100 % PMP storm events, and mapping the inundation areas based on the OSE Rules and Regulations (OSE, December 2010). A PMP depth-duration relationship was developed using Hydrometeorological Report Number 55A (HMR 55A). The inflow flood hydrographs were estimated using unit hydrographs for the “Southwest Desert, Great Basin, and Colorado Plateau” hydrologic region obtained from Reclamation’s Flood Hydrology Manual (Cudworth, 1989). Precipitation losses were calculated using initial and constant infiltration rates. The USACE HEC-HMS computer model (Version 3.4) was used to compute the inflow and outflow flood hydrographs for Laguna Del Campo Dam.

Breach parameters were estimated for the sunny day breach, 50% PMP dam breach, and 100% PMP (IDF) dam breach. Two-dimensional flood inundation modeling was conducted using FLO-2D software (Version 2007.06) for each dam breach scenario to estimate downstream inundation extents. The model extended from Laguna Del Campo Dam downstream to the El Vado Reservoir.

1.3 AVAILABLE INFORMATION AND REFERENCES

Existing information for this study in OSE and NMDGF files is limited. The following information was provided to URS:

- New Mexico OSE Dam Safety Bureau Inspection Reports for Laguna Del Campo Dam (also known as Brood Pond No. 3 Dam) (from 1940 to 2010).
- Drawing titled “Brood Pond No. 3, Parkview Fish Hatchery (Kenneth A. Heron, Engineer)", 1937.
- Drawing titled “Burn Canyon Dam". New Mexico Works Progress Administration, 1938.
- Drawing titled “Repairs to Brood Pond No. 3 Spillway, Parkview Fish Hatchery, Rio Arriba County, New Mexico". Chambers, Campbell, Isaacson, Chaplin, Inc. 1979.

The relevant references used to perform the watershed hydrology study, to determe required design floods, and to perform the dam breach analysis are as follows:

- Cudworth, A.G., Jr., 1989. Flood Hydrology Manual, A Water Resources Technical Publication. U.S. Department of the Interior, Bureau of Reclamation. United States Government Printing Office, Denver.

- United States Department of Interior, Bureau of Reclamation (USBR), 1987. Design of Small Dams. Water Resources Technical Publication. Third Edition.

- New Mexico Office of the State Engineer (OSE), 2010. Rules and Regulations Governing Dam Design, Construction and Dam Safety.

- United States Department of Commerce, et. al., 1988. Hydrometeorological Report No. 55A. Silver Spring, MD. June.

- New Mexico Office of the State Engineer (OSE), 2008. Hydrologic Analysis for Dams, Dam Safety Bureau. August 15.

2.1 OVERVIEW

This section presents the analyses performed for developing the rainfall-runoff characteristics of the watershed, preparing the PMP estimate, and reservoir routing.

2.2 WATERSHED DESCRIPTION

Laguna Del Campo Dam is located in an off-channel area of the Rio Chama known as Parkview Ditch. The confluence of the downstream channel from Laguna Del Campo Dam with the Rio Chama is located approximately 2,500 ft downstream and southwest of Laguna Del Campo Dam. The total drainage area of the watershed was delineated using USGS DEM and is shown on **Figure 2-1.**

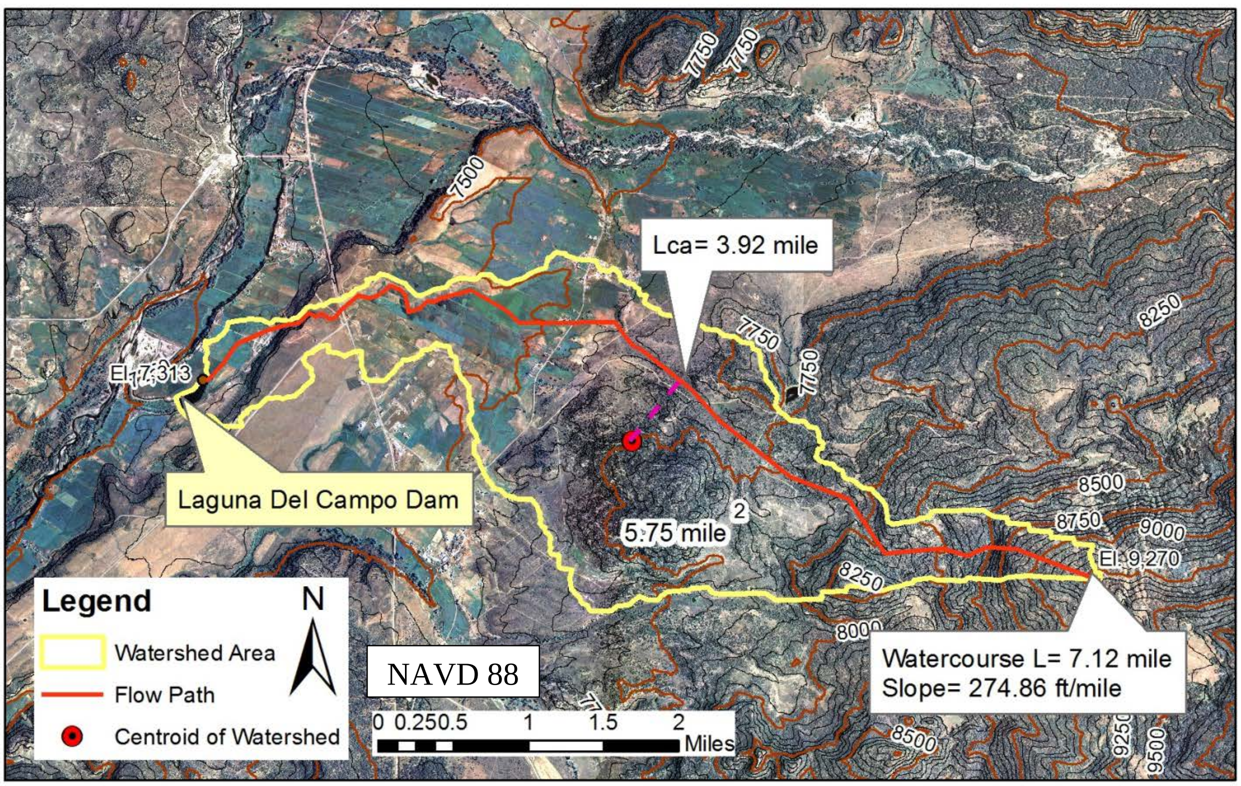

Figure 2-1
Laguna Del Campo Dam Watershed Map (Elevation: NAVD 88)

The Laguna Del Campo Dam watershed is about 5.75 sq. mi., and is located in Rio Arriba County, New Mexico. Watershed elevations range from approximately 9,300 ft at the Peňasco Amarillo to about 7,300 ft at the Laguna Del Campo Reservoir. The entire basin is located in the Tierra Amarilla Land Grant. The upper region of the watershed is undeveloped with coniferous forest located primarily on west facing slopes. The lower region of the watershed consists of agricultural areas, including farms and ranches, and residential properties, having a cover of sparse shrubs and grasses. An off-channel of the Rio Chama known as Parkview Ditch is the inflow channel contributing surface runoff to the Laguna Del Campo Reservoir. The baseflows flowing into the reservoir are expected to be minimal, particularly when compared to PMP inflows.

2.2.1 Infiltration

The initial abstraction depth for the entire watershed was assumed to be zero inches in order to provide a conservative estimate of runoff during the PMP. Uniform infiltration rates were estimated using soil map data developed by the United States Department of Agriculture (USDA) in Geographic Information System (GIS) software. This GIS database contains hydrologic soil groups within the watershed area. The soil data was plotted within the watershed delineation as presented in **Figure 2-2**, and the percentage of each hydrologic soil group that lies within the subbasin was calculated using GIS software.

Four hydrologic soil groups (A, B, C, and D) are defined by the Soil Conservation Service (SCS). Group A soils have high infiltration rates and consist mainly of sands or gravels. Group B soils have moderate infiltration rates and consist mainly of moderately fine to moderately coarse textures. Group C soils have low infiltration rates and consist mainly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. Group D soils have very low infiltration rates and consist mainly of clay.

Table 2-1
Laguna Del Campo Dam Watershed Infiltration Characteristics

Hydrological Soil Group	Area (ac)	Area Percentage	Recommended Infiltration Range (in/hr)*	Suggested Ave. Infiltration Rate (in/hr)	Weighted Infiltration Rate (in/hr)
A	0	0%	---	---	---
B	86.7	2.4%	0.15~0.30	0.225	0.005
C	217.6	5.9%	0.05~0.15	0.1	0.006
D	3,363.8	91.3%	0.00~0.05	0.025	0.023
Water	14.5	0.4%	0	0	0.000
Total Area=	3,683	Ave. Infiltration Rate (in/hr):			**0.034**

* Recommended soil infiltration rate in Flood Hydrology Manual (Cudworth, 1989)

Approximately 2%, 6%, and 91% of the watershed consists of "B," "C," and "D" soil groups, respectively. The Flood Hydrology Manual (Cudworth, 1989), presented in **Appendix C**, describes a range of infiltration rates that can be expected for each hydrologic soil group, as shown in **Table 2-1**. The impervious area of the watershed is very minor and is comprised of the Laguna Del Campo Reservoir surface area. The weighted average infiltration rate was calculated for the entire watershed, based on the hydrologic soil group percentages indicated above. The result was an infiltration rate of approximately 0.034 inches per hour. An initial loss of 0.0 inch per hour was assumed to represent saturated conditions in the watershed due to previous rainfall events. Rainfall losses and excess are shown in **Table 2-6**.

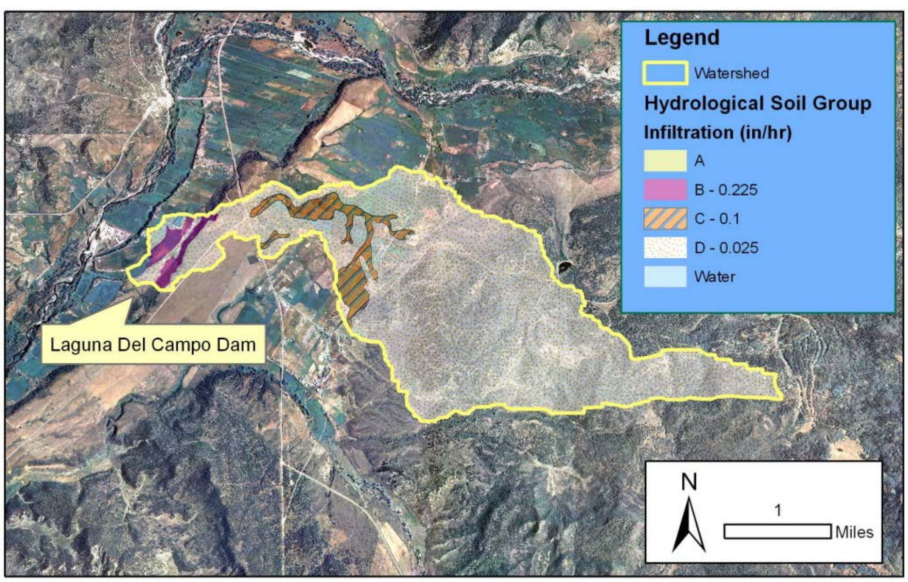

Figure 2-2
Map of Hydrologic Soil Groups in Watershed

2.2.2 Future Basin Development

Future development within the Laguna Del Campo Dam watershed is expected to be minimal since the entire watershed is in the Tierra Amarilla Land Grant. Therefore, increases in the watershed impervious area or modifications to the basin lag time were not evaluated for potential future conditions.

2.3 UNIT HYDROGRAPH

A unit hydrograph for the watershed was developed using the methodology for developing a synthetic unit hydrograph as outlined in Reclamation's Flood Hydrology Manual (Cudworth, 1989). Selected pages of the Flood Hydrology Manual are presented in **Appendix C**. The reservoir watershed lies within the "Southwest Desert, Great Basin, and Colorado Plateau" hydrologic region as shown on **Figure 2** on **Page D-3** in **Appendix D**.

Lag time must be calculated as part of the procedure to obtain the unit hydrograph for a watershed. Lag time is the time from the mid-point of the rainfall excess that half of the volume of unit runoff from the watershed passes the concentration point (Cudworth, 1989). It is influenced by the shape, slope, and roughness of the watershed. The following Reclamation equation was used to estimate the lag time for the watershed:

$$L_g = 26\, K_n \left[\frac{L\, L_{ca}}{S^{0.5}} \right]^{0.33}$$

where:

L_g = Lag time, hours
K_n = Average Manning's n value for principal watershed drainages
L = Length of longest watershed course, mi
L_{ca} = Distance from dam to point opposite of watershed centroid, mi
S = Overall slope of L measured from dam to watershed divide, ft/mi

Unit hydrographs are based on measurable and observed physical parameters of the watershed. These parameters (drainage area, length of longest watercourse (L), distance to centroid (L_{ca}), and slope (S)) were obtained using GIS software and 2009 USGS 1/3-Arc Second National Elevation Data of the watershed area.

The K_n value for the Laguna Del Campo Dam watershed was estimated to be 0.055 for the PMP storms, based on the existing watershed conditions. This K_n value is consistent with the average of the suggested range (0.042 to 0.070) as found in Cudworth (1989) for the hydrologic region. Consequently, and the computed lag time is 1.7 hours. The calculations of these parameters are shown in **Appendix F** and summarized in **Table 2-2**.

Table 2-2
Summary of Laguna Del Campo Dam Watershed Parameters

Parameter	Value
Drainage Area (mi^2)	5.75
Length of Longest Watercourse (mi)	7.12
Distance to Basin Centroid (mi)	3.92
Watercourse Slope (ft/mi)	274.86
Average Weighted Manning's n (K_n)	0.055
Lag Time (hour)	1.7

The watershed unit hydrograph is shown on **Figure 2-3** and tabular data is presented in **Table 2-3**.

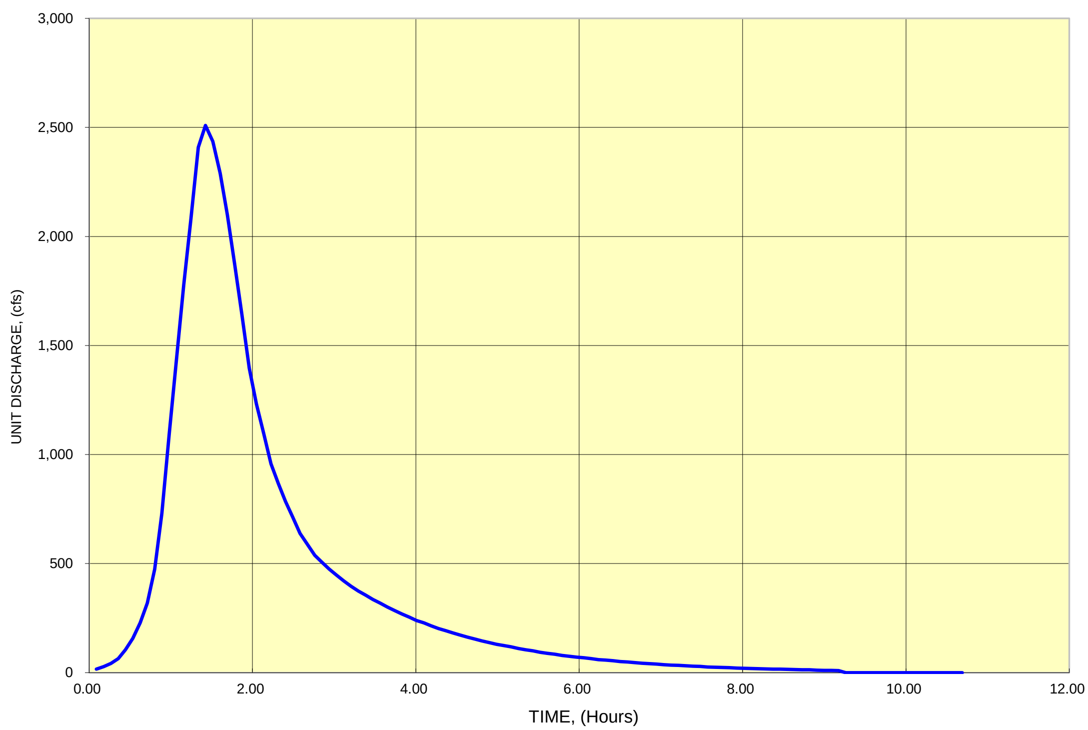

Figure 2-3
Laguna Del Campo Dam Watershed Unit Hydrograph

Table 2-3
Laguna Del Campo Dam Watershed Unit Hydrograph Data

Time (hr)	Unit Discharge (cfs)	Time (hr)	Unit Discharge (cfs)	Time (hr)	Unit Discharge (cfs)
0.08	15	3.00	456	5.92	73
0.17	26	3.08	430	6.00	70
0.25	39	3.17	407	6.08	67
0.33	58	3.25	386	6.17	63
0.42	92	3.33	366	6.25	59
0.50	137	3.42	349	6.33	57
0.58	196	3.50	331	6.42	55
0.67	272	3.58	315	6.50	51
0.75	385	3.67	298	6.58	49
0.83	565	3.75	283	6.67	46
0.92	836	3.83	269	6.75	44
1.00	1,171	3.92	255	6.83	42
1.08	1,488	4.00	240	6.92	40
1.17	1,809	4.08	230	7.00	38
1.25	2,091	4.17	217	7.08	36
1.33	2,398	4.25	206	7.17	34
1.42	2,499	4.33	196	7.25	32
1.50	2,448	4.42	187	7.33	31
1.58	2,323	4.50	177	7.42	29
1.67	2,154	4.58	168	7.50	28
1.75	1,951	4.67	160	7.58	26
1.83	1,738	4.75	152	7.67	24
1.92	1,515	4.83	144	7.75	23
2.00	1,321	4.92	136	7.83	23
2.08	1,178	5.00	129	7.92	21
2.17	1,050	5.08	124	8.00	20
2.25	935	5.17	118	8.08	19
2.33	851	5.25	112	8.17	19
2.42	775	5.33	106	8.25	18
2.50	707	5.42	101	8.33	17
2.58	639	5.50	95	8.42	16
2.67	592	5.58	90	8.50	15
2.75	545	5.67	86	8.58	14
2.83	512	5.75	81	8.67	14
2.92	483	5.83	77	8.75	13

2.4 PRECIPITATION

The General Storm PMP and the Local Storm PMP for the watershed were estimated using procedures presented in HMR 55A. The PMP is theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area, at a particular geographic location, at a certain time of the year. PMP calculations and pertinent HMR 55A materials are presented in **Appendix D**.

2.4.1 General Storm PMP Event

The General Storm PMP is considered a storm event that usually produces precipitation over an area larger than 500 square miles for durations longer than 6 hours. This type of storm is primarily created by cyclonic precipitation associated with large-scale weather features, such as pressure systems and fronts. The 1-hour to 72-hour PMP depths for the General Storm PMP for Laguna Del Campo Dam were estimated from HMR 55A. The General Storm PMP estimates are shown in **Table 2-4**.

Table 2-4
General Storm PMP Depth-Duration Estimates (HMR 55A)

Duration (hr)	Precipitation (in)
1	4.5
6	9.3
12*	13.0
18*	15.5
24	17.0
30*	17.7
36*	18.5
42*	19.0
48*	19.5
54*	20.0
60*	20.8
66*	21.3
72	22.0

*Interpolated from depth-duration plot of HMR 55A values

A summary of the methodology used for the development of the HMR 55A General Storm PMP event is presented below. Plates Ic, IIc, IIIc, and IVc were used to estimate the 1-hour, 6-hour, 24-hour, and 72-hour precipitation depths, respectively, for the watershed. The watershed was located on each plate and the PMP values were estimated. Additional precipitation depths were interpolated using the depth duration curve to estimate precipitation depths at 6 hour increments. **Appendix D** presents the development of the General Storm PMP values. Weighted area reduction factors were not applied since the watershed area is less than 10 square miles.

According to the recommendations in the Hydrologic Analysis for Dams (OSE, 2008), two types of rainfall distribution for General Storm PMP are recommended. They are “center-peaking” and “late-peaking” (2/3 peak). Either the center-peaking or late-peaking distribution are acceptable for 72-hour General Storm PMP. Both distributions were estimated for evaluating the General Storm PMP. The rainfall distributions of the 72-hour General Storm PMP for center-peaking and late-peaking are shown on **Figures 2-4** and **2-5**, respectively. Refer to **Appendix D** for material pertaining to generation of PMP estimates using HMR 55A for the General Storm PMP.

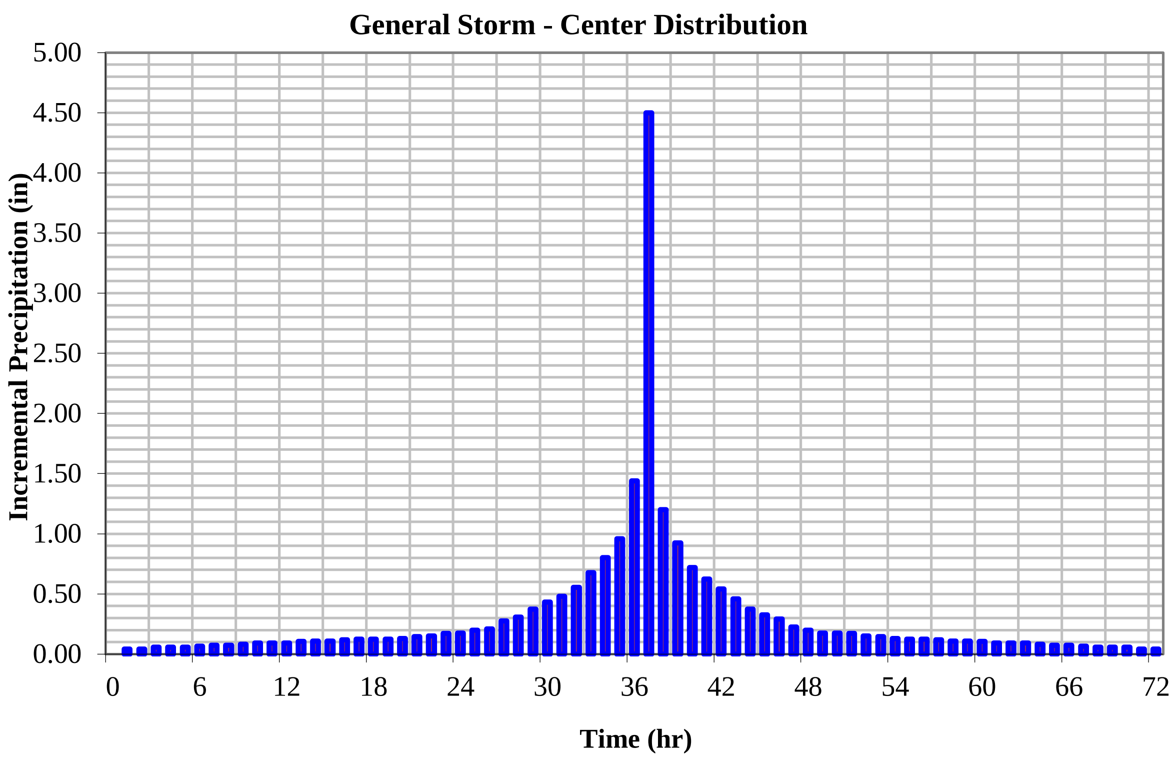

Figure 2-4
72-hr General Storm Hyetograph (Center Peaking)

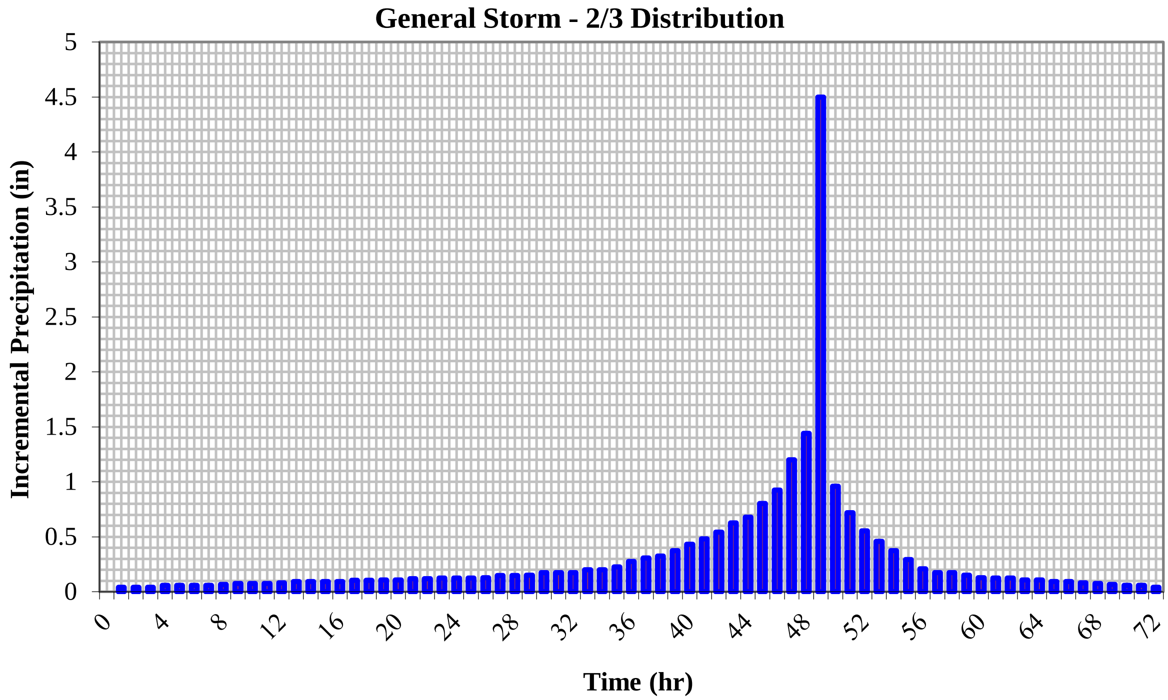

Figure 2-5
72-hr General Storm Hyetograph (2/3-Peaking)

2.4.2 Local Storm PMP Event

The Local Storm PMP is generally considered a storm event that is confined in duration and location. A Local Storm PMP rarely occurs in areas exceeding 500 square miles and the duration is often 6 hours or less. This type of storm is primarily created by convective precipitation associated with vertical upward motion within an extended mass of moist air, where the moist air is warmer than its environment. The 15-min to 6-hour PMP depths for the Local Storm PMP were estimated from HMR 55A.

A summary of the HMR 55A methodology used for developing the Local Storm PMP is presented below. The 1-hour, 1 square mile, 5000 ft elevation precipitation depth was obtained using Plate VIc. An elevation adjustment factor was applied to the 1-hour depth using Figure 4.11 and Figure 14.3 of HMR 55A, with a mean watershed elevation of approximately 7,500 ft. New Mexico OSE suggests that two rainfall distributions be obtained from HMR 5 and USACE EM1110-2-1411. Both distributions should be evaluated to identify the critical Local Storm PMP. Therefore, precipitation depths for durations ranging between 0.25 hours and 6 hours were calculated using ratios of the 1-hour depth based on HMR 5 and USACE EM1110-2-1411. The rainfall distributions of the Local Storm PMP are shown on **Figures 2-6** and **2-7**. Areal reduction factors were then applied to each precipitation depth using Figure 12.12 of HMR 55A for Local Storms. Refer to **Appendix D** for material pertaining to Local Storm PMP estimates using HMR 55A. The Local Storm PMP precipitation depths are shown on **Table 2-5**.

Table 2-5
Local Storm PMP Depth-Duration Estimates (HMR 55A)

Duration (hr)	**Precipitation (in)**
0.25	5.2
0.50	6.8
0.75	7.6
1	8.2
2	9.7
3	10.4
4	10.9
5	11.4
6	11.7

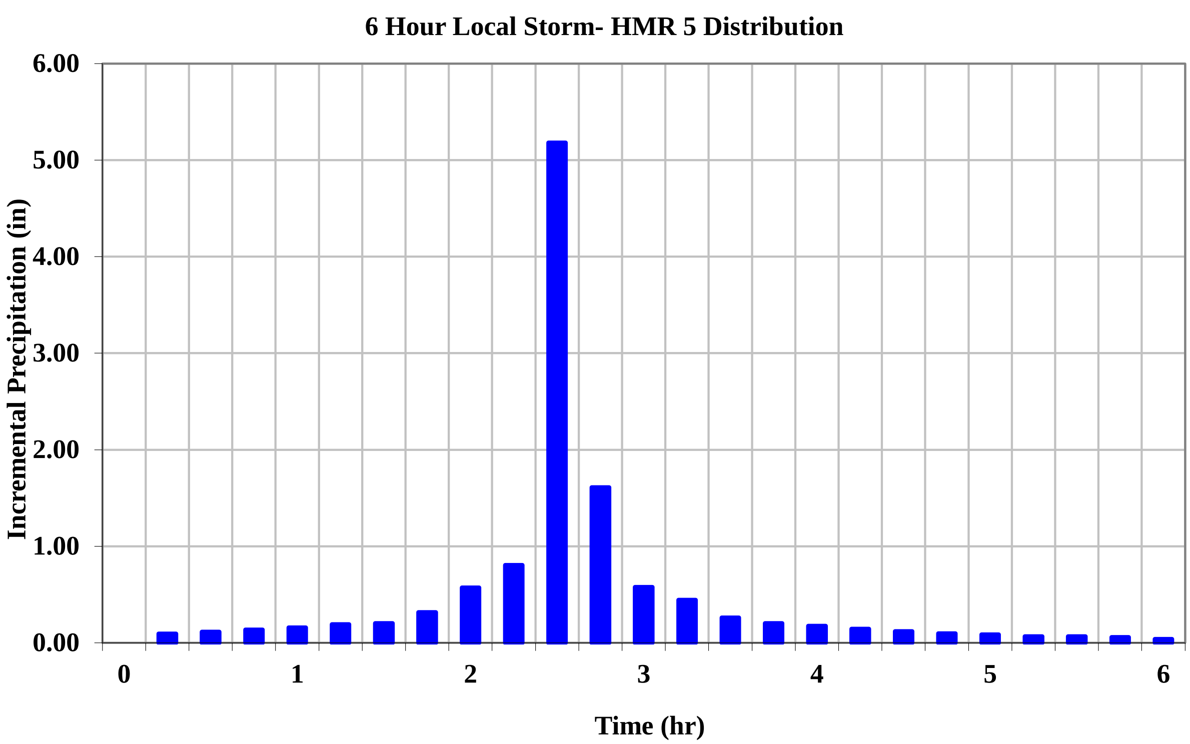

Figure 2-6
6-hr Local Storm Hyetograph (HMR 5)

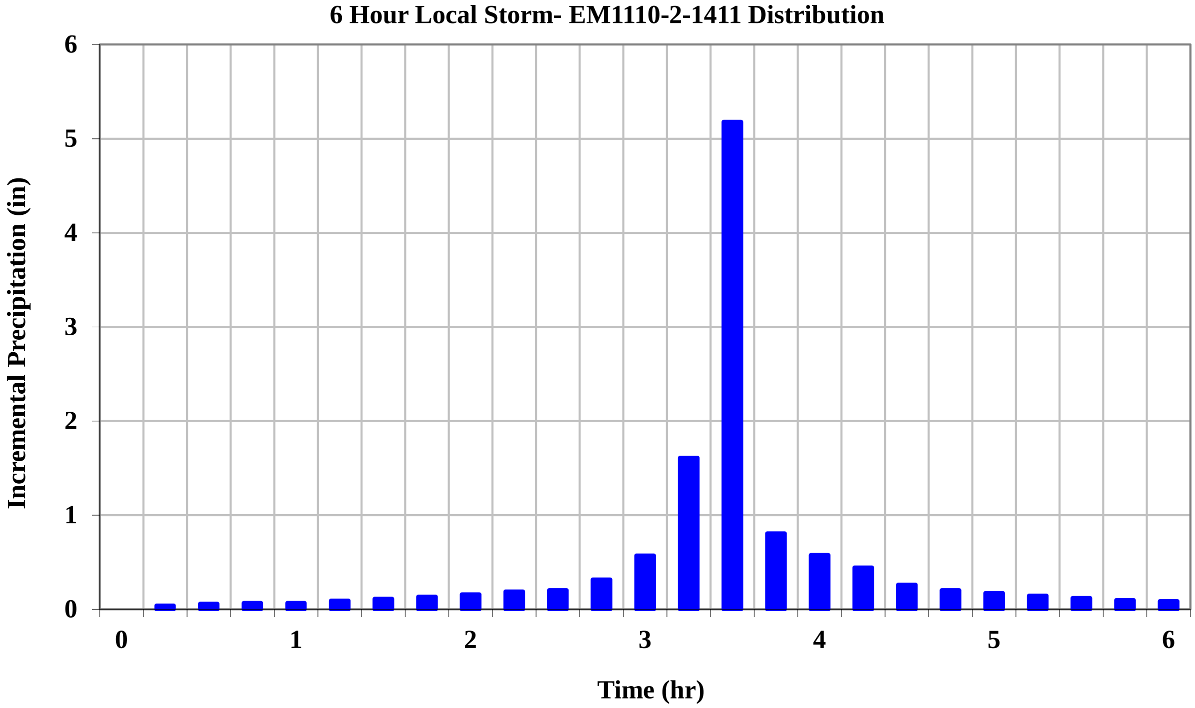

Figure 2-7
6-hr Local Storm Hyetograph (EM1110-2-1411)

2.5 HYDROLOGIC MODELING

Hydrologic modeling was completed using the HEC-HMS computer program, version 3.4, developed by the USACE Hydrologic Engineering Center. The estimated General Storm PMP and the Local Storm PMP were modeled to determine inflow hydrographs at Laguna Del Campo Reservoir. Hydrologic parameters including watershed area, unit hydrograph, precipitation distribution, and infiltration rates were entered into the HEC-HMS model. Base flow is not expected to significantly contribute to the hydrograph, so it was not included in the model. Time steps of 5 minutes and 1 minute were selected for General Storm and Local Storm modeling, respectively, to adequately represent the unit hydrograph and peak rainfall intensities. Screen captures of the HEC-HMS model are provided in **Appendix F**.

The 72-hour General Storms and 6-hour Local Storms were modeled using the hyetographs shown on **Figures 2-4 to 2-7** in the HEC-HMS computer program.

2.6 FLOOD ROUTING RESULTS

Using the aforementioned data, the runoff from the Laguna Del Campo Dam watershed was estimated for the General Storm events and the Local Storm events using HEC-HMS. The HEC-HMS model results are summarized in **Table 2-6**. In summary, the 6-hour Local Storm PMP with the EM1110-2-1411 distribution produces the highest peak discharge. It was evaluated as the worst event to produce the highest dam breach flood and was defined as the critical PMP event for EAP modeling at Laguna Del Campo Dam. Consequently, the 50% PMP induced flood, was also estimated using the Local Storm PMP. Precipitation loss was established from the HEC-HMS model.

Table 2-6
Flood Routing Results

Flood Scenario		Total Precipitation Depth (cfs)	Precipitation Loss (in)	Precipitation Excess (in)	Peak Reservoir Inflow (cfs)	Critical Event
Storm	**Rainfall Distribution**					
100% 6hr Local Storm PMP	EM 1110-2-1411	11.7	0.2	11.5	19,846	X (Inflow Design Flood)
	HMR No. 5	11.7	0.2	11.5	19,799	
100% 72hr General Storm PMP	Center Peaking	22.1	2.4	19.7	10,768	
	2/3 Peaking	22.1	2.4	19.7	10,817	
50% 6hr Local Storm PMP	EM 1110-2-1411	5.8	0.2	5.7	9,864	

3.1 RESERVOIR ROUTING OVERVIEW

Reservoir routing was completed using the same HEC-HMS model that was created for hydrologic modeling. Additional reservoir parameters including dam elevations, dam rating curves, and reservoir stage-storage curves were entered into the HEC-HMS computer program. Peak reservoir outflows and the reservoir water surface elevations from the General Storms and Local Storms were calculated as a basis for selection of the critical PMP event. Screen captures of the HEC-HMS model are provided in **Appendix F**.

3.2 RESERVOIR MODELING

The reservoir storage capacity curve and total dam outflow rating curve were incorporated into the HEC-HMS model. The methods and assumptions used to develop both relationships are presented in the following sections and in **Appendix E**. Elevation-discharge relationships were developed for both the emergency spillway and dam overtopping flows. The initial reservoir water surface was set at the emergency spillway crest, elevation 98.75 ft.

3.2.1 Elevation-Discharge Relationship

The Laguna Del Campo Dam outflow rating curve was calculated by assuming a combined weir flow. Runoff flows through the low-flow opening in the emergency spillway weir structure first, and then flow occurs over the emergency spillway ogee crest, and ultimately the dam and north dike crests. Discharge rates for total dam outflow were calculated for elevations above the emergency spillway crest, as well as the dam and dike crests using the broad crested weir equation:

$$Q = CLH^{3/2}$$

where:

Q = Flow Rate (cfs)

C = Discharge Coefficient (dimensionless)

L = Length of the crest (ft)

H = Hydraulic Head (ft)

A discharge coefficient of 3.5 was used for calculating discharge over the ogee weir. This discharge coefficient is in the range of values suggested by Reclamation (USBR, 1987) as shown on **Figure 3-1**. The main dam crest is 500 ft long at El. 104 as discussed earlier. The length of the north dike was estimated from the 1938 reservoir plan and is about 1030 ft. The 1938 reservoir plan is also shown in **Page A-2** in **Appendix A**. The north dike crest elevation was assumed to be the same as the dam crest at El. 104 as discussed earlier. Flow overtopping the dam and north dike crest was considered broad crested weir flow and calculated using the Bentley Flow Master computer program. In the Bentley Flow Master computer program, the discharge coefficient of 3.09 is estimated for a broad-crest weir based on the input parameters. The Laguna Del Campo Dam outflow rating curve is the combination of the dam and north dike overtopping flow and emergency spillway flow.

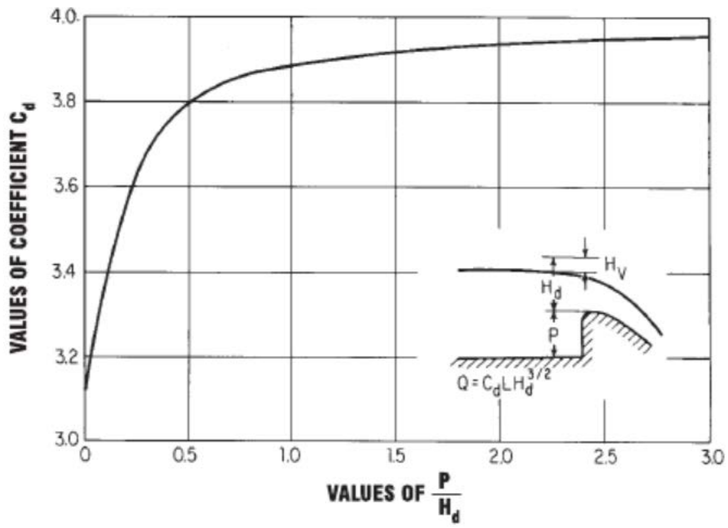

Figure 3-1
Discharge Coefficients for Vertical-Faced Ogee-Crest (from USBR, 1987)

The elevation-discharge relationship is presented in **Table 3-1** and on **Figure 3-2**. Discharge associated with the outlet works was assumed to be negligible during the PMP event.

Table 3-1
Reservoir Elevation-Discharge Data

Reservoir Elevation (ft)	Weir Flow (cfs)				Combined Flow (cfs)
	Emergency Spillway Lowflow Weir	Emergency Spillway Weir	Main Dam	North Dike	
98.15	0	0	0	0	0
98.75	6	0	0	0	6
99	10	11	0	0	21
100	31	117	0	0	148
101	59	284	0	0	343
102	93	492	0	0	585
103	132	736	0	0	868
104	175	1010	0	0	1185
105	221	1313	1414	2913	5861
106	272	1640	4168	8586	14666
107	325	1990	7901	16275	26491
108	382	2363	12318	25376	40439

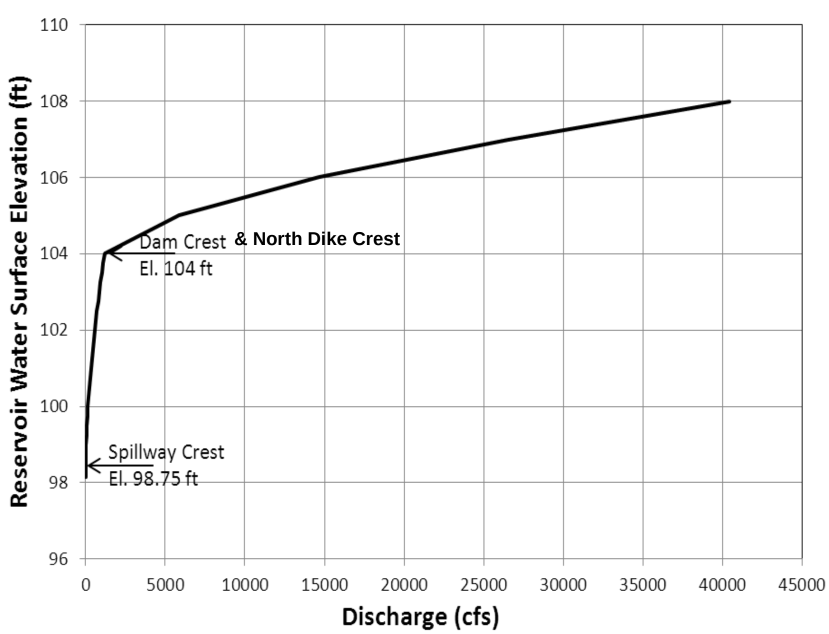

Figure 3-2
Reservoir Elevation-Discharge Rating Curve

3.2.2 Storage Capacity

The elevation-storage relationship has been estimated based on the elevation-area information shown on a 1938 design drawing (**Page A-2 in Appendix A**) for the dam. The elevation-storage-area relationship is presented in **Table 3-2.** The data was extrapolated above elevation 105 ft to elevation 110 ft (6 ft above the dam crest), as shown on **Table 3-2**.

Table 3-2
Reservoir Elevation-Storage-Area Data

Reservoir Elevation (ft)	Area (ft^2)	Depth (ft)	Area (acres) [d]	Reservoir Storage Volume		
				Incremental Vol. (ft^3)	Accumulated Vol. (ft^3)	Accumulated Vol. (Ac-Ft)
73.0	0	0.0	0.00	-		0.00
75.0	5,650	2.0	0.13	5,650	5,650	0.13
80.0	43,525	7.0	1.00	122,937	128,587	2.95
85.0	122,325	12.0	2.81	414,626	543,213	12.47
90.0	223,676	17.0	5.13	865,004	1,408,216	32.33
95.0	339,424	22.0	7.79	1,407,750	2,815,967	64.65
98.75[b]	471,635	25.8	10.83	1,520,736	4,336,703	99.56
99.0	480,449	26.0	11.03	119,011	4,455,713	102.29
104.0[c]	829,866	31.0	19.05	3,275,788	7,731,501	177.49
104.5	864,808	31.5	19.85	423,668	8,155,170	187.22
105.0	899,749	32.0	20.66	441,139	8,596,309	197.34
106.0[a]	1,007,295	33.0	23.12	953,522	9,549,831	219.23
110.0[a]	1,437,480	37.0	33.00	4,889,551	14,439,382	331.48

Notes: a) Values represent those extrapolated
b) Emergency Spillway crest El. 98.75
c) Dam crest El. 104
d) Information obtained from 1938 design drawing

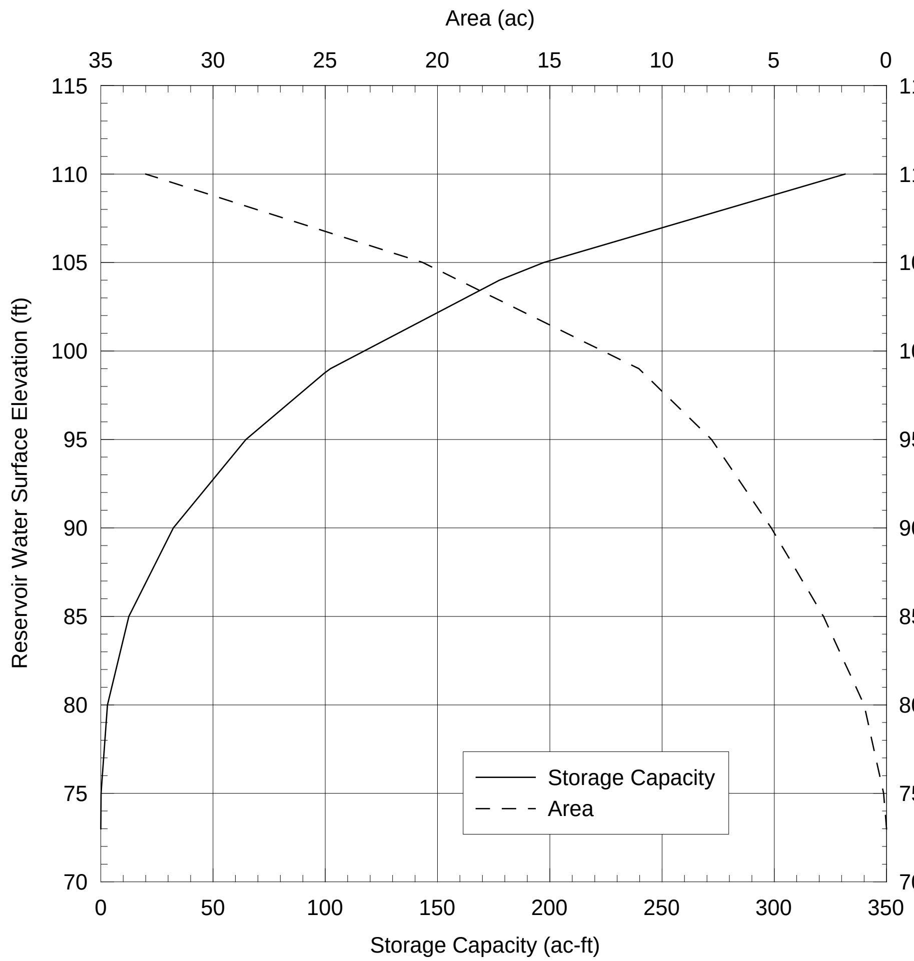

Figure 3-3
Reservoir Elevation-Storage-Area Curve

3.3 RESERVOIR ROUTING RESULTS

The reservoir routing results for the General Storm PMP and Local Storm PMP are presented in this section. Utilizing the aforementioned input data, hydrologic modeling of the PMP Storms was completed using the HEC-HMS computer program, which calculated results for peak reservoir inflows, maximum reservoir levels, and peak dam outflows. The HEC-HMS model results are summarized in **Table 3-3**. Based on the analysis results, the 6-hour Local Storm based on the EM 1110-2-1411 rainfall distribution was selected as the critical PMP event for Laguna Del Campo Dam. The peak dam outflow discharge is approximately 19,800 cfs, with a maximum reservoir surface elevation of 106.5 ft, which is approximately 2.5 ft above the dam crest.

Table 3-3
Reservoir Routing Results

Flood Scenario		Peak Inflow (cfs)	Peak Outflow (cfs)	Max. Reservoir Water Surface Elevation (ft)	Dam Overtopping Depth (ft)	Critical Event
Storm	**Rainfall Distribution**					
100% 6-hr Local Storm PMP	EM 1110-2-1411	19,846	19,793	106.5	2.5	X (IDF)
	HMR No. 5	19,799	19,733	106.5	2.5	
100% 72-hr General Storm PMP	Center Peaking	10,768	10,757	105.6	1.6	
	2/3 Peaking	10,817	10,818	105.6	1.6	
50% 6-hr Local Storm PMP	EM 1110-2-1411	9,864	9,836	105.5	1.5	

Figures 3-4 and **3-5** present the inflow-outflow hydrographs and the reservoir water surface-storage plot of the 100% and 50% Local Storm PMP, respectively, for the Laguna Del Campo Reservoir.

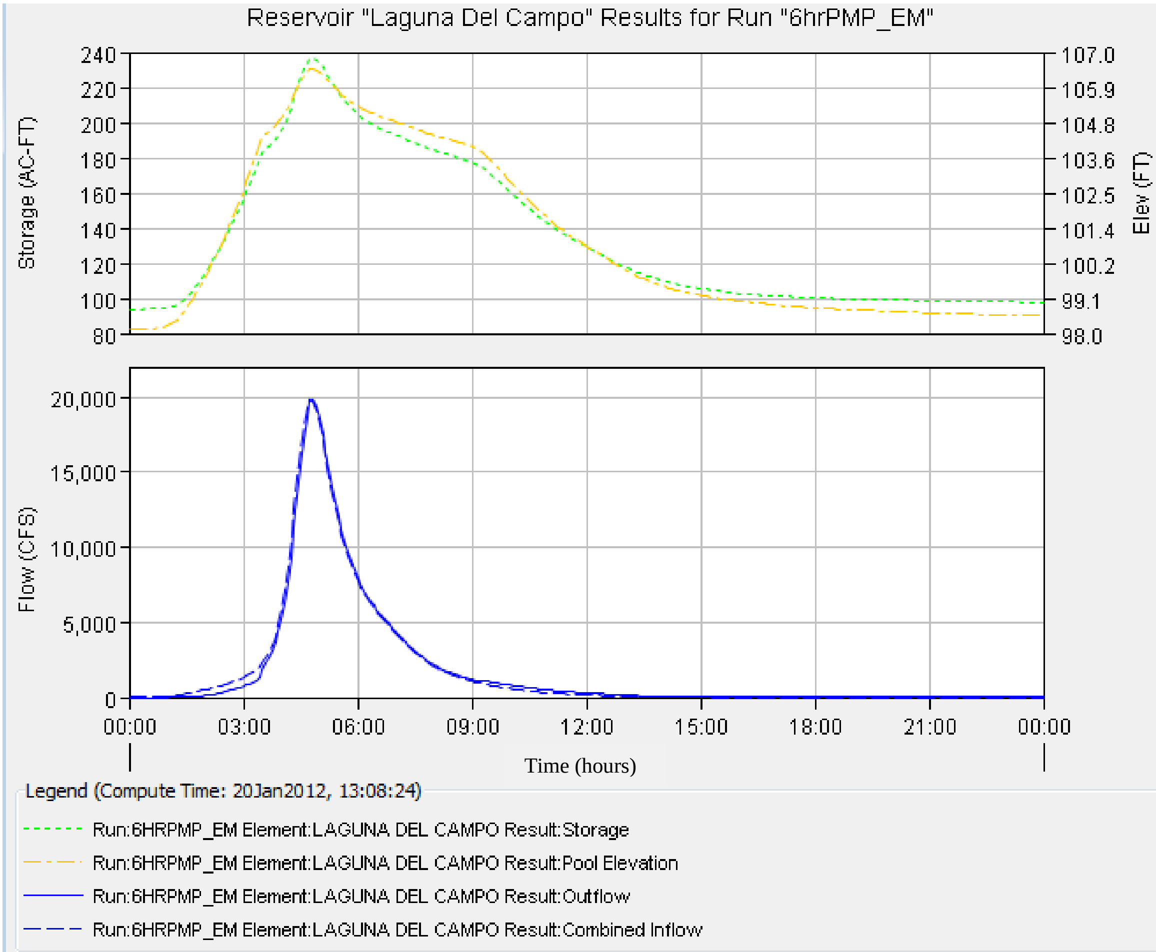

Figure 3-4
Inflow-Outflow Hydrographs and Stage-Elevation Graphs for Laguna Del Campo Reservoir (100% 6-hr Local Storm PMP)

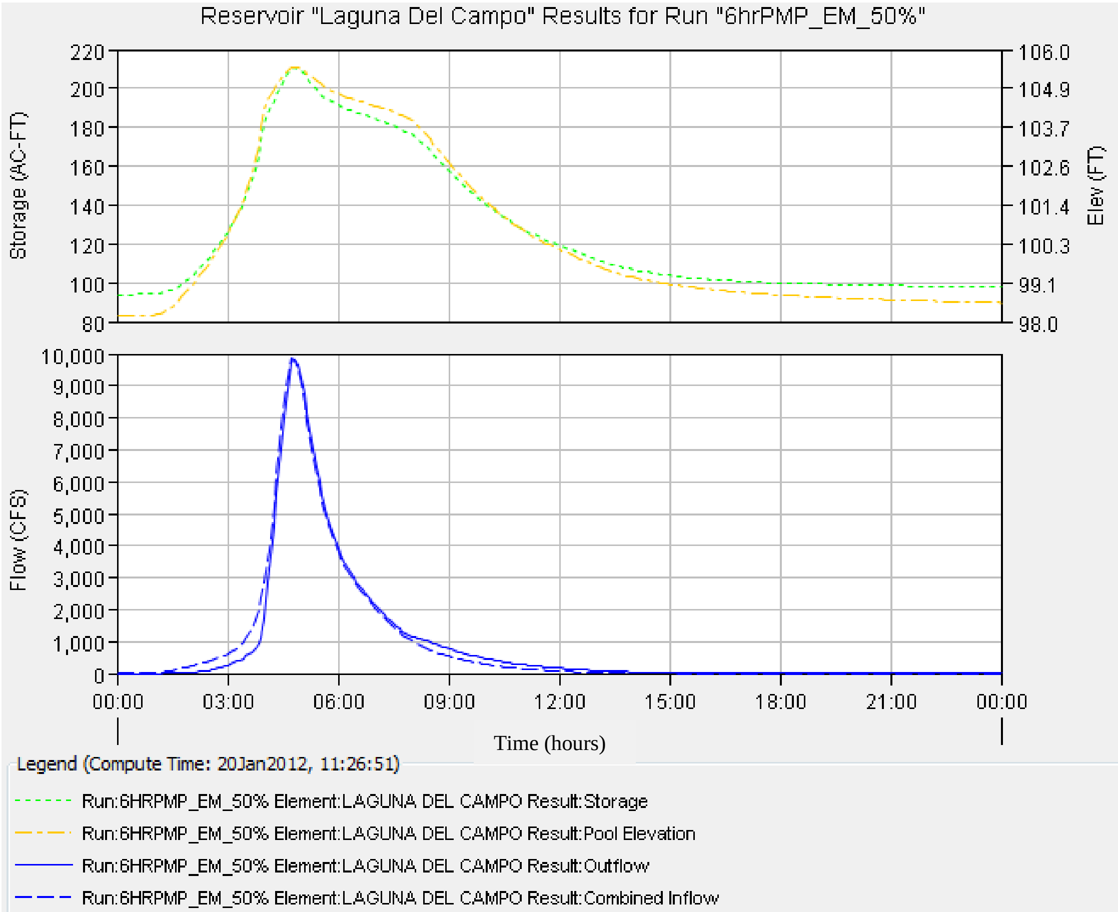

Figure 3-5
Inflow-Outflow Hydrographs and Stage-Elevation Graphs for Laguna Del Campo Reservoir (50% 6-hr Local Storm PMP)

3.4 BREACH ANALYSIS

Breach parameters were estimated for the sunny day, 50% PMP, and 100% PMP induced failures of the Laguna Del Campo Dam. A sunny day breach occurs with the water surface elevation at normal pool without a storm event occurring at the time of failure. It was modeled assuming a piping failure scenario. A sunny day breach occurs as a result of a defect related to the dam, which can arise from, but not be limited to, improper dam maintenance, construction, or operation. The PMP failures were assumed to occur during the 100% and 50% Local Storm PMP events, respectively. Laguna Del Campo Dam was assumed to fail from the dam overtopping during the PMP events.

The dam breach analyses due to piping failure and overtopping failure were completed using a combination of dam breach parameter empirical relationships and the HEC-HMS computer program. The Froehlich Breach Predictor Equations (Wahl, 1998) were used to estimate pertinent dam breach parameters for entry into the HEC-HMS model, which subsequently estimates the dam breach outflow hydrograph. HEC-HMS breach modeling input and output data have been provided in **Appendix F**.

3.4.1 Dam Breach Parameter Estimation

The value of dam breach parameters for embankment dams are suggested from several empirical ranges developed by the National Weather Service (NWS), the U.S. Army Corps of Engineers (USACE), the Federal Energy and Regulatory Commission (FERC), and the Bureau of Reclamation (USBR). **Table 3-4** shows the empirical ranges for dam breach parameters for embankment dams.

Table 3-4
Suggested Breach Parameters for Earth Dams

Source	Average Breach Width	Breach Side Slope (1 Vertical : Z Horizontal)	Breach Forming Time (hours)
NWS (1988)	1H to 5H	Z= 0 to 1	0.1 to 2.0
USACE (1980)	0.5H to 4H	Z= 0 to 1	0.5 to 4.0
FERC (1991)	1H to 5H	Z= 0 to 1	0.1 to 1.0
USBR (1982)	3H	N/A	H/100 (H in ft)

Note: H = Height of water against dam above breach bottom elevation

When modeling an embankment dam breach, the most sensitive parameters are breach bottom width and time of breach formation. A dam breach flood event is mainly dominated by reservoir storage, dam embankment material, and failure type. A estimation of the dam breach parameters was performed to support the dam breach modeling for the Laguna Del Campo Dam. Froehlich Breach Predictor Equations (Wahl, 1998) were used to estimate the dam breach parameters. Froehlich Breach Predictor Equations (Wahl, 1998) in SI units are as follows:

$$B = 0.1803 K V_w^{0.32} h_b^{0.19}$$

$$t = 0.00254 V_w^{0.53} h_b^{-0.9}$$

where B= average breach width (m); K= overtopping multiplier, 1.4 for overtopping, 1.0 for piping; V_w= volume of water mixture stored above breach invert at time of failure (m^3); h_b= height of breach (m); and t=failure time (hour). The initial reservoir water surface for sunny day failure was assumed to be at the emergency spillway crest (El. 98.75). The breach initiation reservoir elevation for piping failure formation was assumed to be at Elevation 98.74 for modeling purposes. For each overtopping failure scenario, the approximate maximum reservoir water surface elevation was obtained from the HEC-HMS modeling. These reservoir elevations indicate the water storage behind the dam and produce the probable maximum breach outflow from Laguna Del Campo Dam. **Table 3-5** shows the computed dam breach parameters for each analyzed scenario. The breach bottom widths and failure times shown in **Table 3-5** are physically related due to using the empirical equations and also are within the range of suggested dam parameters shown in **Table 3-4.** These results indicate that these estimated dam breach parameters are empirically based and reasonable for dam breach hydrologic modeling. The dam breach parameter calculation is presented in **Appendix F**.

Table 3-5
Computed Dam Breach Parameters for Laguna Del Campo Dam

Dam Failure Scenario	Failure Type	Breach Initiation Reservoir Water Surface Elevation (ft)	Water Volume (ac-ft)	Time of Failure (hr)	Breach Bottom Width (ft)
Sunny Day	Piping	98.74	99.6	0.2	7.5
100% 6-hour Local Storm PMP	Overtopping	105.8	214.7	0.2	36.9
50% 6-hour Local Storm PMP	Overtopping	105.1	199.4	0.2	35.7

3.4.2 Sunny Day Piping Breach

The HEC-HMS model was utilized to estimate the dam failure outflow hydrographs for use in subsequent downstream flood routing and inundation mapping. Using the aforementioned input values, the peak outflow resulting from a sunny day piping breach of Laguna Del Campo Dam was estimated to be approximately 8,019 cfs, as shown in **Table 3-6** and on **Figure 3-6**. Additionally, it was also found that the full embankment height breach had a higher outflow than piping at the mid-point of the final breach height. HEC-HMS output data have also been provided in **Appendix F**.

Table 3-6
Computed Peak Dam Breach Outflows from Laguna Del Campo Dam

Flood Scenario		Failure Type	Dam Overtopping Depth (ft)	Peak Discharge (cfs)
Storm	Rainfall Distribution			
Sunny Day	N/A	Piping	N/A	8,019
100% 6-hr Local Storm PMP	EM 1110-2-1411	Overtopping	1.9	26,903
50% 6-hr Local Storm PMP	EM 1110-2-1411	Overtopping	1.2	23,807

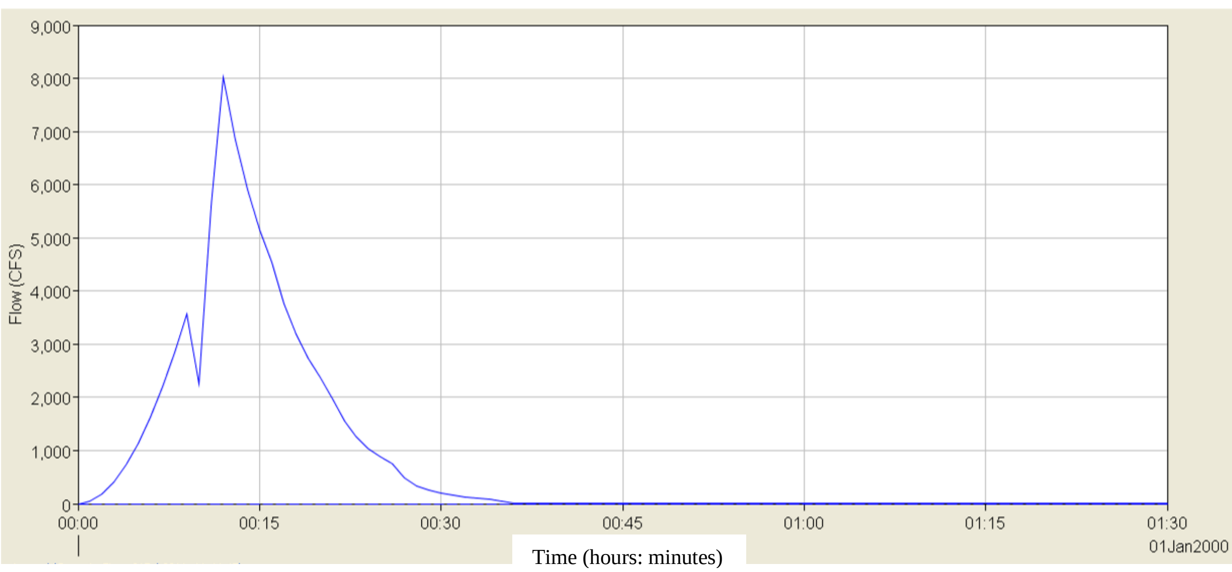

Figure 3-6
Sunny Day Piping Breach Outflow Hydrograph

3.4.3 100% and 50% PMP Overtopping Breach

Reservoir routing results indicate that Laguna Del Campo Dam overtops during the 50% and 100% PMP events. The critical PMP was identified as the 6-hour Local Storm PMP. A conservative approach to a dam overtopping breach analysis is to estimate the reservoir water surface elevation that begins the overtopping breach formation, which was verified by repeating calculations of the peak reservoir level and storage, anticipated dam breach parameters, and dam breach modeling, to approximately estimate the greatest outflow from Laguna Del Campo Dam. Consequently, the estimated dam breach parameters resulting in the greatest outflow from Laguna Del Campo Dam during an overtopping failure scenario for Laguna Del Campo Dam are shown in **Table 3-5**. The dam was modeled to breach at 1.19 ft of overtopping depth during the 100% PMP and at 1.2 ft of overtopping depth during the 50% PMP. Using these breach parameters, the peak outflow resulting from the 100% PMP overtopping breach of Laguna Del Campo Dam was simulated to be approximately 27,000 cfs, and the peak outflow from the 50% PMP overtopping breach is approximately 24,000 cfs. The simulated flood hydrographs of the PMP breach and 50% PMP breach are shown on **Figures 3-7** and **3-8,** respectively. The reservoir outflow is the sum of the dike overflow and the outflow from the main dam. HEC-HMS output data have also been provided in **Appendix F**.

The computed outflow hydrographs of a sunny day breach, a 100% PMP breach, and a50% PMP breach were used to prepare the EAP inundation mapping.

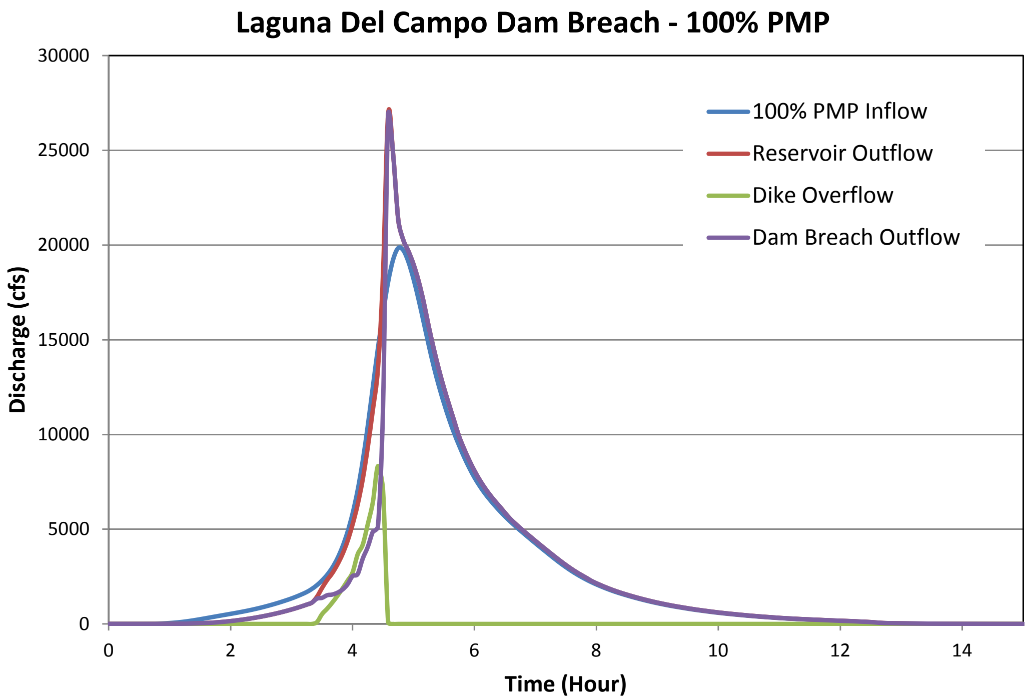

Figure 3-7
Overtopping Breach Outflow Hydrograph at 100% PMP

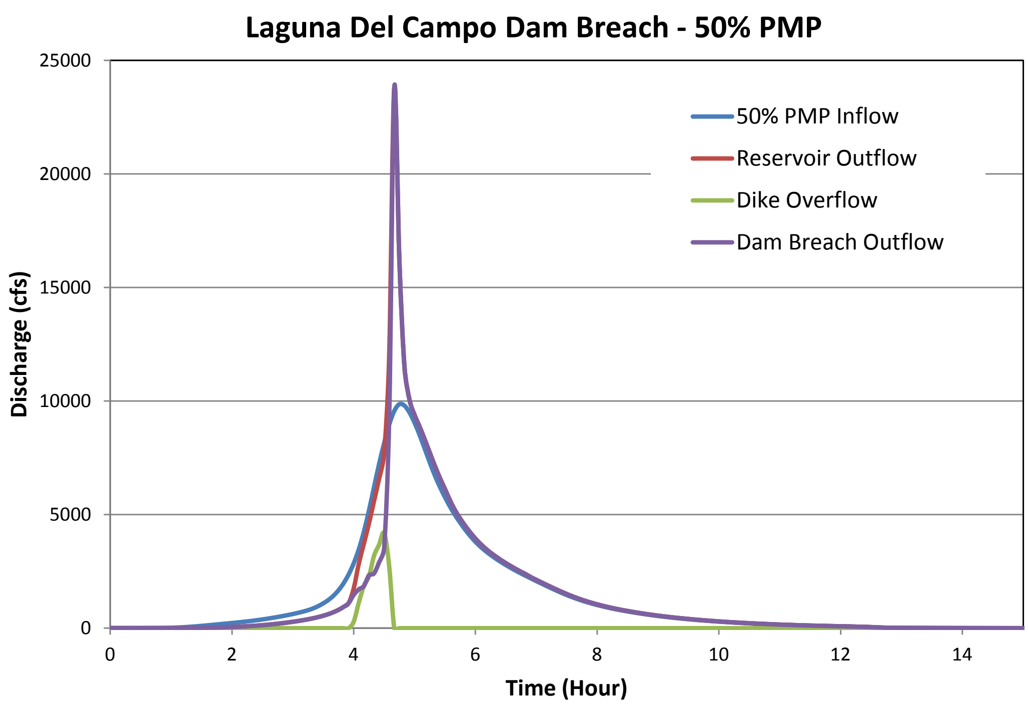

Figure 3-8
Overtopping Breach Outflow Hydrograph at 50% PMP

4.1 INUNDATION MODELING OVERVIEW

Inundation mapping downstream of the Laguna Del Campo Dam was conducted using the two-dimensional computer program FLO-2D, version 2007.06. Discussion of the channel downstream of the dam, the FLO-2D computer program, engineering methodology, and inundation modeling results are presented in this section.

4.2 DOWNSTREAM CHANNEL DESCRIPTION

Laguna Del Campo Dam is located on a side channel of the Rio Chama, about 2,500 ft upstream from the confluence with the Rio Chama. The side channel is a low flow channel conveying outflow from the Laguna Del Campo Dam to the Rio Chama. Residential and agricultural land characterizes the side channel. The study reach extends from the dam through the side channel and the Rio Chama to the downstream termination at the El Vado Reservoir. The study reach is about 13.7 miles long. The reach between the side channel/Rio Chama confluence to about 2.6 miles downstream from the confluence is generally characterized by a broad and wide floodplain. The remaining reach is a high and narrow canyon. The average slope of the side channel is about 3%. The average gradient along the Rio Chama is approximately 0.4%.

4.3 ENGINEERING METHOD

FLO-2D is a two-dimensional hydraulic model that is specifically designed for flood routing simulations over alluvial fans, in channels and floodplains. It is a finite difference model that uses a square system of grid elements overlain on the downstream topographic mapping. The flood hydrograph is routed using the full dynamic wave approximation to the momentum equation. FLO-2D is on FEMA's list of approved hydraulic models for both riverine and unconfined alluvial fan flood studies. It has been used extensively by a number of Federal agencies including the USACE, Reclamation, USGS, Natural Resource Conservation Service, U.S. Fish and Wildlife Service and the National Park Service.

For this study, FLO-2D was used to complete the flood routing in place of a traditional one-dimensional model, due to the rapidly rising peak of the dam breach hydrograph and the wide floodplains. Model stability is commonly difficult to attain with a rapidly rising hydrograph in one-dimensional models, however, model stability is generally more easily attainable with a FLO-2D simulation.

The dam breach outflow hydrographs from the Laguna Del Campo Dam obtained from the HEC-HMS model results were used as inflow hydrographs for the dam flood inundation modeling scenarios which are the sunny day breach, the 100% Local Storm PMP breach and the 50% Local Storm PMP breach. The model includes the side channel from the Laguna Del Campo Dam to the confluence with the Rio Chama and the Rio Chama from the confluence to the El Vado Reservoir. Residential or agriculture structures along the study reach were identified so that potential flooding impacts could be verified using model results. The downstream study boundary of FLO-2D is shown in **Figure 4-1**. The dam failure flood inundation simulation results are presented in **Appendix H**.

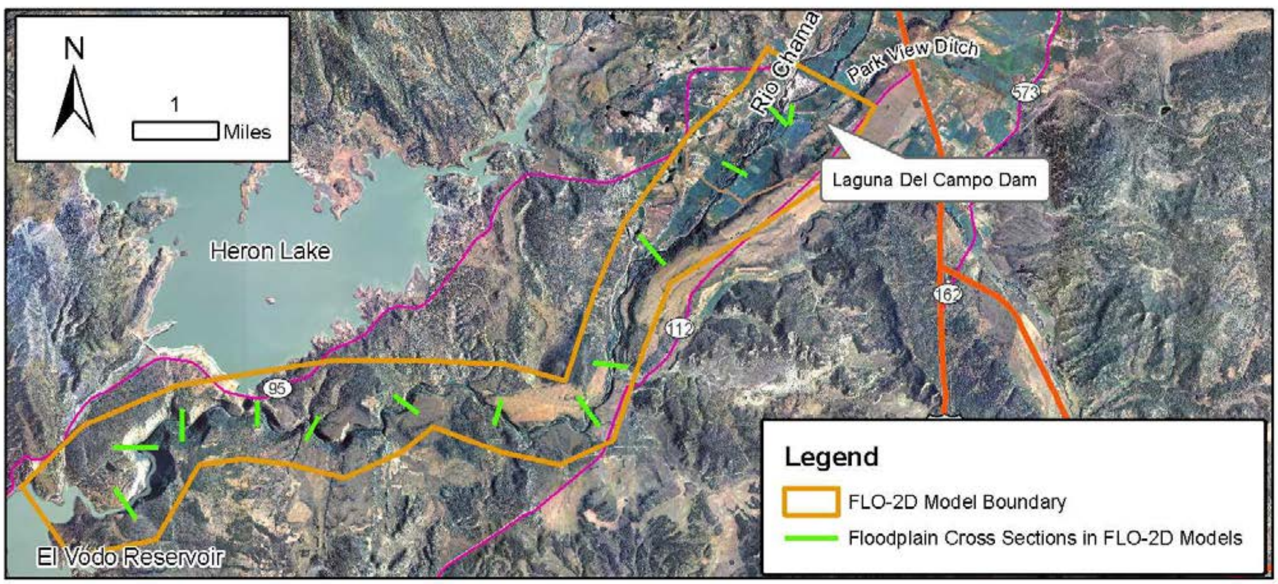

Figure 4-1
FLO-2D Computational Boundary Map

4.4 TWO-DIMENSIONAL HYDRODYNAMIC MODELING

4.4.1 Topographic Data, River Cross Sections and Model Details

FLO-2D is an effective tool for delineating flood hazards or designing flood mitigation. Overland flow is routed in eight directions either as sheetflow using either the kinematic or the diffusive wave approximation to the momentum equation. For the floodplain flood routing, the parameters having the greatest effect on the area of inundation or outflow hydrographs are as follows (FLO-2D Software, Inc., 2007):

- The overland flow path is primarily a function of the topography.

- Inflow hydrograph discharge and volume directly affect the area of inundation.
- The floodplain roughness n-values can range from 0.03 to 0.5 and control the overland floodwave speed.

In this study, the FLO-2D model was developed from available topographic data and aerial images. The FLO-2D topographic surface was generated from 10-meter, USGS digital elevation model (DEM) data. No ground surveys were performed for this study. It should be noted that USGS DEMs often do not adequately resolve roadway embankments, stream crossings and areas of recent development. Therefore, the flood routing results may not provide adequate detail at these features.

The DEM points were imported to the FLO-2D Grid Developer System (GDS) and grid element elevations were interpolated and assigned. In the selection of the grid element size, it is necessary to balance the model resolution with the total number of grid cells. Increasing the resolution increases the total number of grid cells thereby significantly increasing model computation time. Commonly, a grid size between 250 ft to 2,000 ft is used for the floodplain inundation modeling. In this study, in order to enhance the model accuracy based on the obtained DEM resolution, a grid element size of 50 ft was selected and was assumed to adequately resolve the topography. Since 10-meter (33 ft) DEM data was implemented in the model, it was determined that 50-foot grid cells resulted in the highest model resolution with a reasonable computation time. The final system consists of 100,071 elements.

The dam breach outflow hydrographs from Laguna Del Campo Dam obtained from the HEC-HMS model results were assigned to the inflow elements at the dam outlet works where the main dam embankment is located and the northern dike where overtopping flow runs through these potential impacted structures. A total of 14 floodplain cross sections were created and cross the study reach in the model to obtain the simulated flood hydrographs at these locations. The locations of floodplain cross sections are shown on the inundation maps in **Appendix I**. The simulation times are nine hours and 30 hours for PMP events and Sunny Day event, respectively. These simulation times are longer than the inflow hydrograph duration and allow the simulated flood peaks pass through the entire study reach.

The FLO-2D models were built with the default tolerance values, which are (FLO-2D Software, Inc., 2007):

- Tolerance value for the percent change = 0.2 or DEPTOL = 0.2;
- Surface detention value = 0.1 ft or TOL= 0.1;
- Maximum value of the numerical stability coefficient for full dynamic wave flood routing = 1.0 or WAVEMAX = 1.0.

4.4.2 FLO-2D Model Assumptions and Limitations

Based on the computer program limitations, available information, and project scope, the assumptions and limitations of the Laguna Del Campo Dam FLO-2D model are illustrated in this section. The main purpose of this dam breach analysis study is to support the Laguna Del Campo Dam EAP. The upstream end of the FLO-2D model was located at the downstream toe of the dam. The model included the tributary channel from Laguna Del Campo Dam to the confluence with Rio Chama, the reach from the confluence downstream to El Vado Reservoir in Rio Chama. The study reach is approximately 13.2 miles, as shown on **Figure 4-1**.

First, the simulated results are constrained by the FLO-2D program abilities. Very detailed flow hydraulics, such as hydraulic jumps, flow in river bends, around bridge piers, or other detail/complicated hydraulic structures, cannot be simulated with the FLO-2D model. FLO-2D does not distinguish between subcritical or supercritical flow and has no restrictions when computing the transition between the flow regimes. For minor flows, such as the beginning of the inflow hydrograph or the split of flow on a floodplain, the flow discharge is simulated as very shallow sheetflow in the computation grids. Manning's Roughness Coefficients are adjusted based on sheetflow depth. Thinner sheetflow is computed with higher flow roughness and less flow velocity by FLO-2D. These computations make these sheetflow grids become sticking grids and produce very long flow travel times for these sheetflow grids.

Second, the flood inundation simulation is limited by the available data and study level. In this study, the dam flood inundation model was built using the available topographic data, which is USGS DEM. The FLO-2D model is essentially complete except for localized flood detail in some areas. For instance, Tetra Tech Inc. (Tetra Tech, 2005) developed a Below Caballo Dam FLO-2D model for the U.S. Army Corps of Engineers in Albuquerque, New Mexico. Their study results concluded the following:

> *"Accurate flood hazard delineation in local reaches depends on roadway/railroad embankment, wasteways and irrigation system ditches and spoil pile embankments. This detail is not anticipated to significantly affect the movement of the floodwave or alter the maximum water surfaces or discharges. It may impact the area of inundation in a local overbank area.*
>
> *Hydraulic structures are important to local flooding but are not critical to the passage of the floodwave through the system. The bridges, diversion dams and siphons have very limited (almost negligible) upstream storage and therefore accuracy of the rating tables is not critical to the floodwave movement."*

In addition, as aforementioned in **Section 4.4.1**, the FLO-2D user's manual mentions that the inundation area is directly affected by the inflow hydrograph discharge and volume. Therefore, the detailed local structure/ground variation is not anticipated to significantly affect the movement of the floodwave or alter the maximum water surfaces or discharges. This level of evaluating local flooding conditions would require detailed local topographic features and more local hydrologic information, which were not available for this study and were beyond the scope of work.

In the Laguna Del Campo Dam FLO-2D model, a grid element size of 50 ft was assumed to adequately resolve the topography for channel, levee, road embankment, bridge and large residential and commercial structures. For all three dam breach scenarios, the model setup assumed that small hydraulic structures, such as pedestrian bridges, are destroyed. There is no dam flood overtop major road embankments.

Lastly, the Laguna Del Campo Dam FLO-2D model was built with a conservative and practical model setup. The flood inundation was simulated using the FLO-2D floodplain flood hydraulic computation. No sediment transport or debris flow was considered in this study. The runoff losses due to infiltration and evaporation were excluded. For the Laguna Del Campo Dam FLO-2D model, the average Manning's "*n*" value of 0.035 reflecting the existing natural grass channel was assigned to the elements in channel area. The average Manning's "*n*" value of 0.05 reflecting the floodplain with scattered brush (Wurbs and James, 2002) was assigned to the elements in floodplain area. The roughness will be increased with a decrease in the flow depth, therefore, the higher the coefficient, the greater the increase in roughness by the FLO-2D program. This roughness adjustment will slow the progression of the floodwave advancing downstream. In the Laguna Del Campo Dam FLO-2D model, the Manning's "*n*" values vary between 0.035 and 0.089 depending on the computed flow depth.

For detailed numerical methodology and program applications of FLO-2D, refer to the program users' and reference manuals by FLO-2D Software, Inc. Additional details for the Laguna Del Campo Dam FLO-2D model are located in **Appendix H**.

4.4.3 FLO-2D Model Calibration

Dam breach flood inundation is an extreme hazard event and rarely occurs compared to frequency storm floods. The dam breach inundation is primarily dominated by the floodplain overflow and is not like an in-channel flood. In this study, no historical record, or inundation maps observed from past extreme floodplain flood events for the study reach, and no gauged outflow hydrographs from Laguna Del Campo Dam were available for model calibration. Consequently, the FLO-2D models were conservatively built based on the understanding of the study reach and recommended model parameters.

4.5 MODEL SIMULATION RESULTS

The computation run time of each scenario is approximately 68 hours. In the FLO-2D model, the time steps are incremented and decremented during a flood simulation to maintain numerical stability. In this study, the model time steps are between 0.011 and 10.9 seconds. The accuracy of the numerical routing is monitored by volume conservation in the model, which is listed in either the BASE.OUT or SUMMARY.OUT files. The outputs show 100% of volume conservation balance for all analyzed scenarios. There were no error messages found in the model outputs.

Flood inundation maps for Laguna Del Campo Dam are included in **Appendix I**. These maps include the sunny day dam breach, the dam breach during the 50% PMP event, and the dam breach during the 100% PMP event. Mapping extends along the tributary downstream of Laguna Del Campo Dam to the confluence with the Rio Chama, where flows from a potential dam failure are expected to remain within the floodplain area of the Rio Chama.

Floodwave travel velocity, which is shown on the inundation maps as floodwave arrival time and time to peak, is a function of flood volume, wave length, and flow depth. Generally, for the sunny day breach, the floodwave is moving faster than for the PMP floods because the floodwave of the sunny day breach has a relatively short wave length and deep flow depth. While floodwaves are attenuating downstream, the floodwave velocity of the sunny day breach is significantly reduced by the shallow flow depth, which is split over the floodplain. At that time, the PMP floodwaves are moving faster than the sunny day floodwave. In the Laguna Del Campo Dam FLO-2D models, the computed results show that the floodwave arrival times at upstream cross-sections are longer for the 50% PMP and 100% PMP breach scenarios than for the sunny day failure scenario. Similarly, the floodwave arrival times at downstream cross sections are shorter for the 50% PMP and 100% PMP breach scenarios than for the sunny day failure scenario.

It should be noted that the initiation time (t=0) for the dam breach floodwave arrival and peak stage times for the sunny day event and PMP flood events are different. **Table 4-1** shows the definitions of the dam breach floodwave travel times. Based on these definitions, the sunny day dam breach timing starts at breach initiation or the beginning of the simulation time. According to the HEC-HMS reservoir routing outputs, the PMF dam breach timing is initiated (t = 0) at 4.17 hr from the beginning of the analyzed PMP events. This time value was used as a cutoff to compute the floodwave arrival and peak stage times obtained from the FLO-2D outputs. The computed results are shown on the dam flood inundation maps in **Appendix I**.

Table 4-1
Definitions of Dam Breach Floodwave Travel Time

	Dam Breach Floods
Initiation Time (t=0)	Breach initiation at dam embankment
Floodwave arrival time	Beginning of flow increase due to dam failure

The beginning of the dam outflow hydrograph shows very minor discharge. In the FLO-2D model, these minor discharges were simulated as very shallow sheetflow in the computation grids. Flow velocity of sheetflow was computed using the adjusted high Manning's Roughness Coefficients. These computations make these sheetflow grids become sticking grids and produce relatively slow flow velocity for these sheetflow grids.

The extreme flooding event is the dam breach during the 100% PMP event with the inundation limits are greater than those for the sunny day and 50% PMP breaches. All three dam breach hydrographs attenuate significantly as the floodwave progresses downstream. The sunny day, 50% PMP, and 100% PMP peak flow rates vary from approximately 8,000 cfs, 23,800 cfs, and 26,900 cfs, respectively, at Laguna Del Campo Dam to 13 cfs, 6,000 cfs, and 15,800 cfs, respectively, to El Vado Reservoir (about 14 miles downstream of Laguna Del Campo Dam). The inundation area immediately downstream of the dam is expected to have an average maximum width of approximately 250 ft. The remainder of the inundated area along the Rio Chama is expected to have an average maximum width of 160 ft to 1,800 ft.

The dam breach scenario results in the inundation of a private access road about 1,000 ft downstream from Laguna Del Campo Dam, which is expected to overtop as a result of the sunny day, 50% PMP and 100% PMP dam breaches. Additionally, sheetflow introduced by the overflow from the north dike runs down on the natural ground and is collected by Rio Chama. The sheetflow is not expected to significantly inundate residential structures or roads as the flow depth is likely less than 1 foot. The gravel pit operated by Russell Sand and Gravel Co. Inc., houses and other structures downstream of the north dike of Laguna Del Campo Dam may be inundated by sheet flow during PMP events. The inundation extents may be viewed on the maps provided in the EAP and **Appendix I**.

4.6 MODELING LIMITATIONS

All dam breach scenarios herein were modeled based on hypothetical assumptions for the PMP intensity and distributions, which were used to produce a storm event that was expected to produce the largest flood hazard downstream of the dam. The delineated inundation areas indicate the probable maximum inundation areas and provide the local emergency responders guidance of potentially impacted structures. However, the precipitation distribution, reservoir level, and dam breach formation time and geometry are not necessarily the same during an actual precipitation or dam failure event at Laguna Del Campo Dam, and the dam failure could be initiated by any combination of events. The computed flood arrival and maximum stage times provide on the inundation maps may, therefore, differ during an actual event at Laguna Del Campo Dam.

Hydrologic modeling shows that the Local Storm with an EM 1110-2-1411 rainfall distribution is the controlling PMP storm event for Laguna Del Campo Dam. The peak reservoir inflow is expected to be approximately 19,800 cfs with a total inflow volume of approximately 3500 ac-ft.

It was estimated that the 100% PMP (IDF) overtopping breach will result in a peak water surface elevation of 105.9 ft, which is 7.2 ft above the emergency spillway crest. The sunny day breach of Laguna Del Campo Dam is expected to produce a peak outflow of approximately 8,000 cfs. A peak breach outflow of approximately 23,800 cfs is estimated during the 50% PMP, and 26,900 cfs is estimated during the 100% PMP failures of the dam. The dam was modeled to breach at the approximately maximum reservoir level over dam crest during the 50% PMP and 100% PMP dam breach scenarios.

Inundation modeling was conducted along the unnamed tributary from Laguna Del Campo Dam and Rio Chama to El Vado Reservoir. Flood impacts along the majority of the modeled area are minimal. Only a private access road about 1,000 ft downstream from Laguna Del Campo Dam is expected to overtop during dam breach floods. The gravel pit operated by Russell Sand and Gravel Co. Inc., houses and other structures at north of the right bank dike of Laguna Del Campo Dam may be inundated by sheet flow during PMP events.

URS represents that its services were performed within the limits prescribed by the client in a manner consistent with the level of care and skill exercised within the current standard of professional engineering practice of other similar engineering professionals in New Mexico. No other representation to the client, expressed or implied, and no warranty or guarantee is included or intended. URS does not guarantee the performance of the project in any respect; only that the engineering work and judgments rendered meet the standard of care of the profession.

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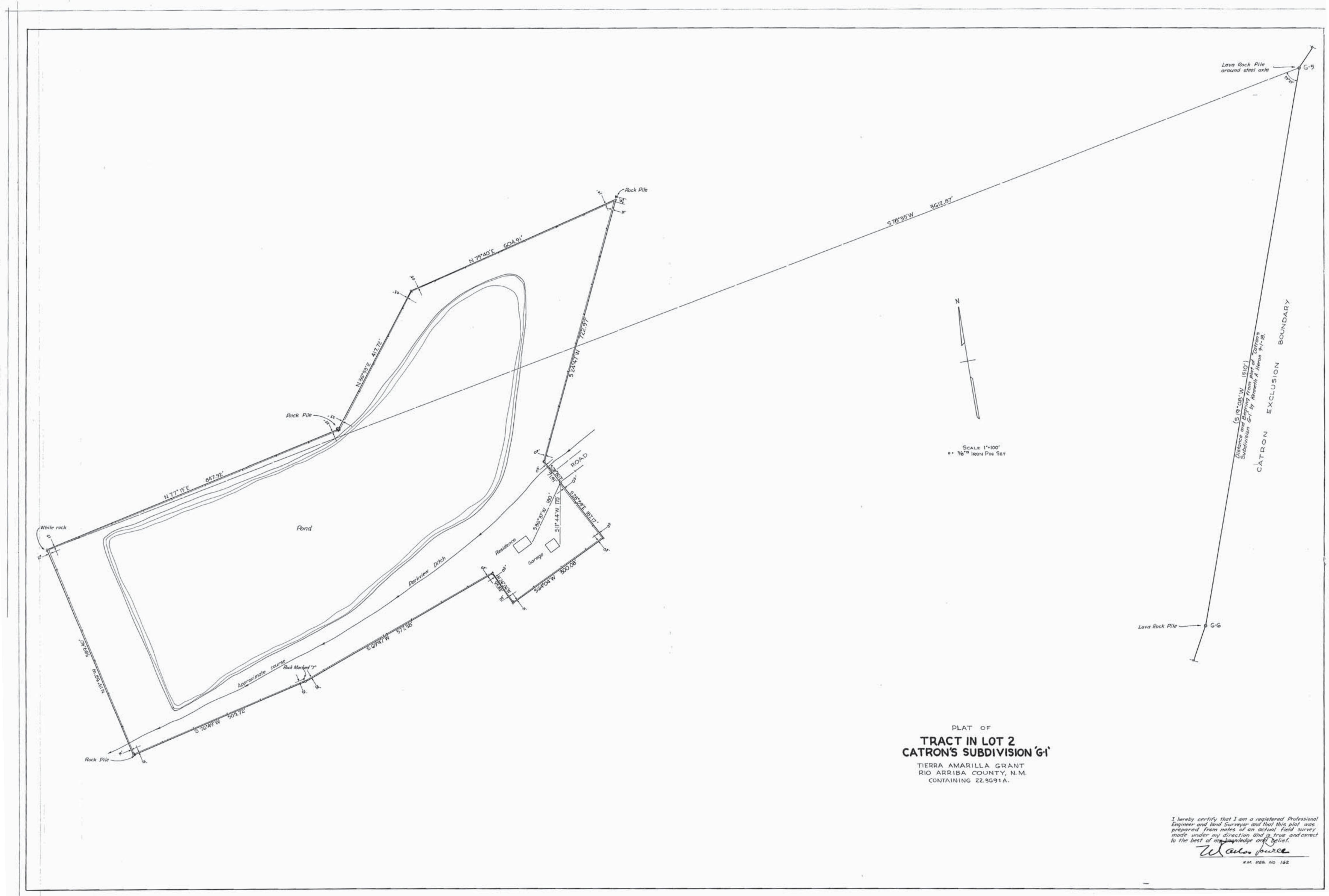
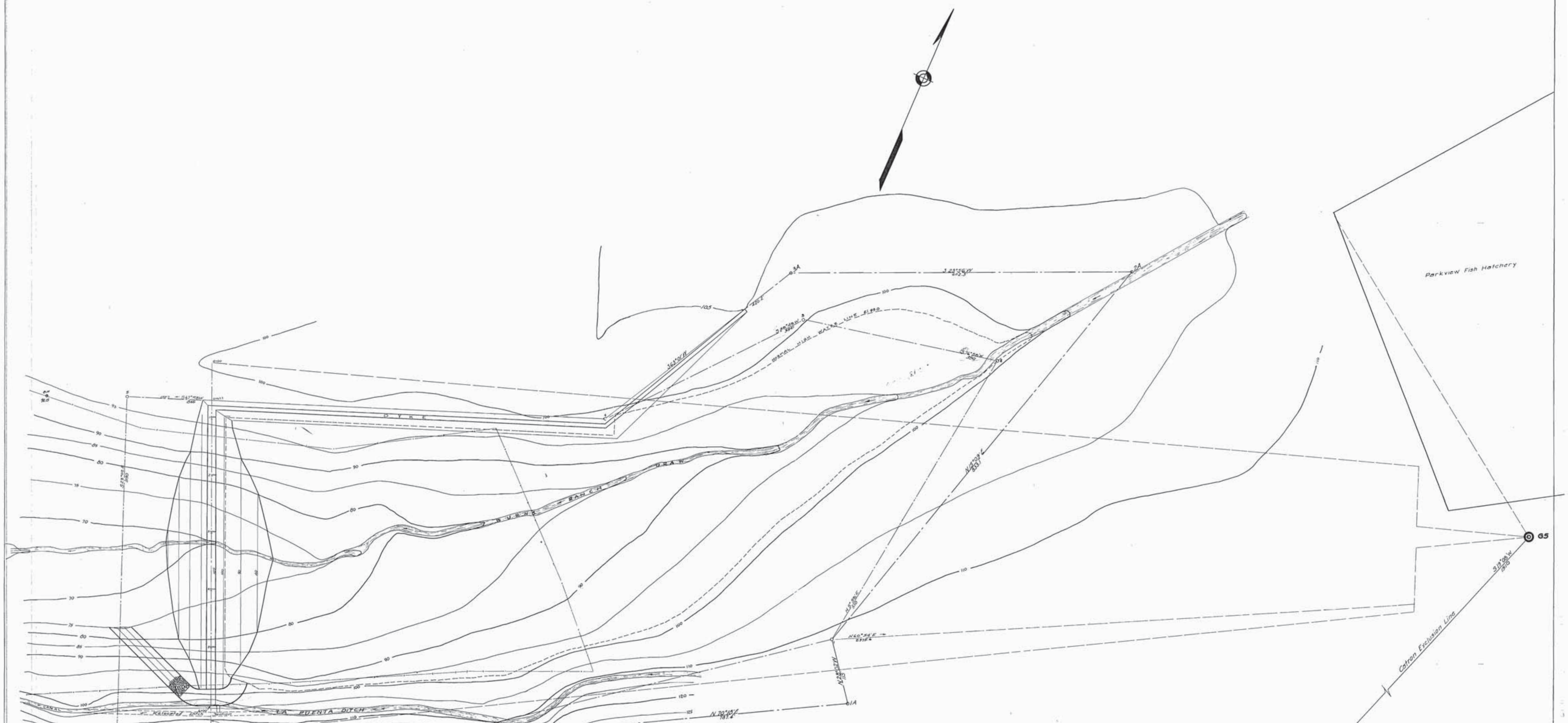
Appendix A

Design Drawings of Laguna Del Campo Dam


Lava Rock Pile around steel axle
G-5
Rock Pile
N 79°40' E 604.91'
S 78°35' W 3612.87'
N 36°35' E 417.72'
S 24°47' W 722.97'
Rock Pile
N 77°15' E 847.92'
White rock
Pond
ROAD
Residence
Garage
Parkview Ditch
S 64°04' W 300.08
S 61°47' W 571.56
Approximate course
Rock Marked "7"
S 76°49' W 505.72'
Rock Pile
N
Scale 1"=100'
o = 3/8" Iron Pin Set
CATRON EXCLUSION BOUNDARY
(S 19°08' W 1510')
Distance and Bearing from plat of "Catron's Subdivision G-1" by Kenneth A. Heron
Lava Rock Pile
G-6
PLAT OF
TRACT IN LOT 2
CATRON'S SUBDIVISION 'G-1'
TIERRA AMARILLA GRANT
RIO ARRIBA COUNTY, N.M.
CONTAINING 22.3691 A.
I hereby certify that I am a registered Professional Engineer and land Surveyor and that this plat was prepared from notes of an actual field survey made under my direction and is true and correct to the best of my knowledge and belief.
N.M. REG. NO 162



RESERVOIR CAPACITY				
ELEV.	AREA IN ACRES	MEAN AREA	ACRE FEET	Σ AC. FT.
73	0			
		0.0649	0.19	
75	0.1297			0.19
		0.5645	2.82	
80	0.9992			3.01
		1.9037	9.52	
85	2.8082			12.53
		3.9715	19.86	
90	5.1349			32.39
		6.4635	32.32	
95	7.7921			64.71
		9.4109	37.64	
99	11.0296			102.35
		15.8425	95.06	
105	20.6554			

FILL QUANTITIES IN DAM					
ELEV.	AREA	SUM	AVERAGE	CU. FT.	CU. YDS.
69	292				
		3976	1988	9940	368
70	3684				
		30428	15214	61070	2262
75	26744				
		62416	31208	156040	5779
80	35672				
		83356	41678	208390	7718
85	47684				
		80860	40430	202150	7487
90	33176				
		61560	30780	153900	5700
95	28384				
		48396	24198	120990	4481
100	20012				
		27812	13906	69530	2575
105	7800				
	203448	398804	199402	982010	36370
	QUANTITIES ARE ABOVE NATURAL GROUND.				

FILL QUANTITIES IN CORE WALL					
ELEV.	AREA	SUM	AVERAGE	CU. FT.	CU. YDS.
70	540				
		15312	7656	38280	1417.8
75	14772				
		34060	17030	85150	3153.7
80	19288				
		38084	19542	97710	3618.9
85	18796				
		37350	18675	93375	3458.3
90	17560				
		31564	15782	78910	2922.6
95	14004				
		24148	12074	60370	2235.9
100	10144				
		13312	6656	33280	1232.6
105	3168				
				487075	18039.8
	QUANTITIES ARE ABOVE NATURAL GROUND				

SUMMARY OF QUANTITIES

Pervious Materials in Dam	16330	Cu Yds.
Impervious Materials in Core Wall (Fill Section)	18040	do
do do do do do (Excavated Section)	11100	do
Unclassified Materials in Dike	4500	do

Unclassified Excavation for Core Wall 11100 Cu Yds.
do do do Spillway 2000 do

Spillway Capacity 1360 cu. ft. per sec.

NEW MEXICO WORKS PROGRESS ADMINISTRATION
ENGINEERING DEPARTMENT
SANTA FE NEW MEXICO

BURNS CANON DAM
PARKVIEW RIO ARRIBA CO
FRED G HEALY
ADMINISTRATOR
SCALE 1"=50'

DRAWN BY Gly
CHECKED BY
DATE Apr 27 1938
PROJECT NO 465-85-2-64
SHEET NO 1

F.P.-No.PV-8

CLAIMANT'S CERTIFICATE

STATE OF NEW MEXICO }
COUNTY OF SANTA FE } ss:

I, Elliott S. Barker, being first duly sworn upon my oath state that I am the Secretary of the State Game Commission, duly organized under the laws of the State of New Mexico, that the foregoing map and statements were made under authority of the State Game Commission, and that I, in their behalf, have read and examined the foregoing statements and representations thereon and state that the same are true to the best of my knowledge and belief.

STATE GAME COMMISSION
Claimant.

By Elliott S. Barker
Secretary.

Subscribed and sworn to before me this 24th day of July, 1937

Minnie B. Carter
Notary Public

My commission expires June 3, 1939

ENGINEER'S CERTIFICATE

STATE OF NEW MEXICO }
COUNTY OF RIO ARRIBA } ss:

I, Kenneth A. Heron, being first duly sworn upon my oath, state that I am the engineer who made the map of the BROOD POND NO. 3, PARKVIEW FISH HATCHERY, and that such map was made from field notes taken from actual bona fide surveys made in the field by me, and from plans furnished by Geo. M. Neel, and that the same true and correct to the best of my knowledge and belief.

Kenneth A. Heron
Engineer.

Subscribed and sworn to before me this 20th day of July, 1937.

A. J. Barr
Notary Public.

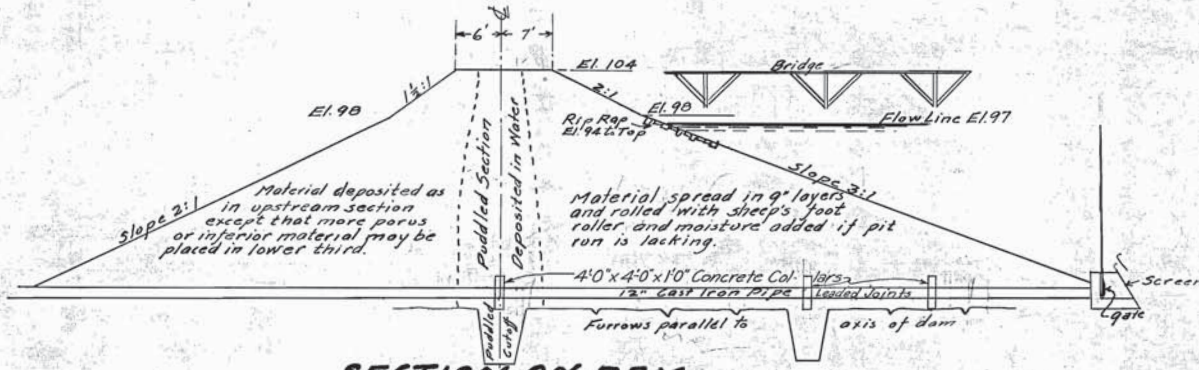
My commission expires 2-1-40

STATE ENGINEER'S CERTIFICATE

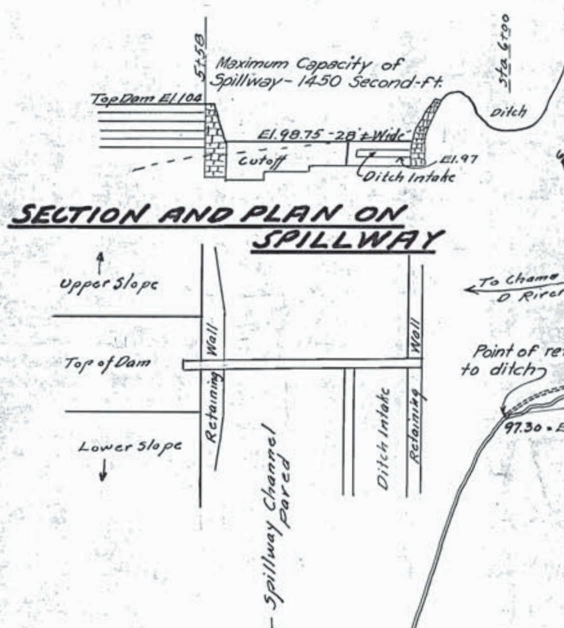
STATE OF NEW MEXICO }
COUNTY OF SANTA FE } ss:

I hereby certify that the above map and statements have been examined by me, and were duly accepted for filing on the 27 day of July 1937.

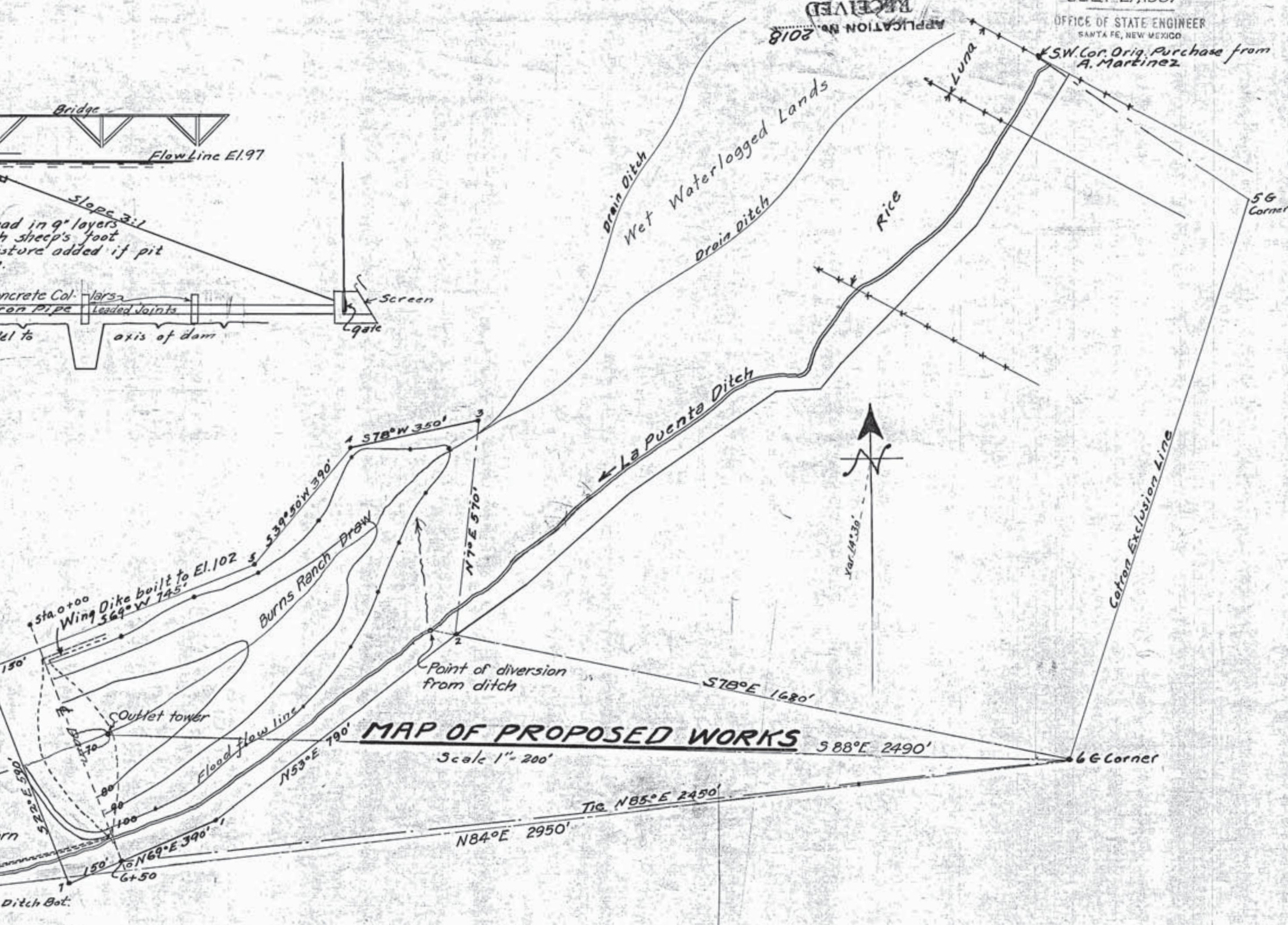
Thos. M. McClure
State Engineer.



SECTION ON DAM

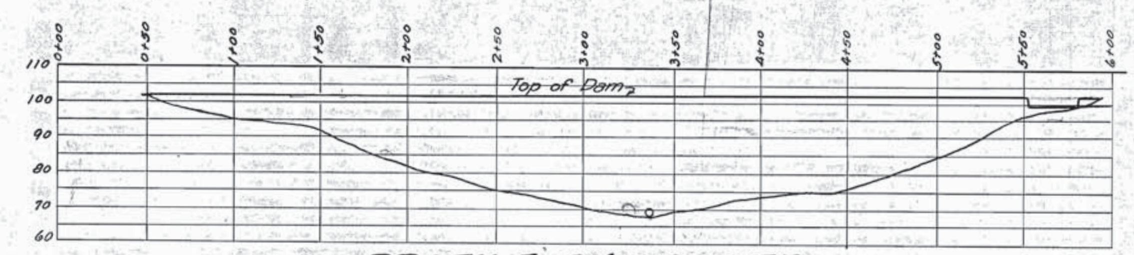


SECTION AND PLAN ON SPILLWAY



MAP OF PROPOSED WORKS

Scale 1" = 200'



PROFILE ON DAMSITE

Looking Upstream

The claimant, the State Game Commission, has caused to be located by a competent surveyor and engineer, the works shown hereon and makes these several statements relative thereto, and offers this map and statements for acceptance and due filing in compliance with the laws of the State of New Mexico:

1. The "6G" corner on the boundary of the Parkview Mesa Tract, Catron Exclusions, Tierra Amarilla Grant bears N85°E 2450' from Sta. 6+50 on the ℄ of the proposed dam.
2. All of the lands shown hereon are on the Tierra Amarilla Grant, Rio Arriba County, N.M.
3. Maximum height of dam is 36.0';
   Length of dam is 500';
   Top of dam is 13.0 wide; bottom 190' wide;
   Dam to be an earth fill;
   Slopes to be as shown hereon;
4. Maximum storage capacity of 96.7 acre feet is hereby claimed continously, when there is an unappropriated supply of water to fill the reservoir.
5. The cost of the works is estimated at $30,000.00;
6. Drain ditches 4' wide and 2' deep are proposed as shown hereon.
7. The pond or reservoir is to be used for fish breeding purposes.
8. The reservoir tract is to be acquired from the present owner, T. D. Burns.
9. Surplus water to the amount of 6.0 cfs. originating on the property of the claimant one half mile Northeast of the proposed pond is wasted into La Puenta Ditch. It is proposed to divert this into the pond and divert all surplus water back into the La Puenta Ditch.
10. Soil on Midsection of dam site is surface gravel underlaid at 2 ft. depth with tight impervious clay. Walls of canyon mostly clay with some gravel imbedded in clay matrix.

APPLICATION No. 2181
RECEIVED
JULY 27, 1937
OFFICE OF STATE ENGINEER
SANTA FE, NEW MEXICO

Reservoir Contents				
Elev.	Area acres	M.A.	Contents a.F.	Total Contents A.F.
70	0.2			
		1.1	11.0	
80	2.0			11.0
		3.8	38.0	
90	5.6			49.0
		6.8	47.7	
97	8.0			96.7
		8.8	26.3	
100	9.5			123.0

Map of

BROOD POND NO. 3, PARKVIEW FISH HATCHERY

-of-

STATE GAME COMMISSION, APPLICANT

Located in Rio Arriba County, N. M.

Scale as Shown

July 20, 1937

Kenneth A. Heron
Engineer.

APPLICATION No. 2181
RECEIVED
JULY 27, 1937
OFFICE OF STATE ENGINEER
SANTA FE, NEW MEXICO

APPLICATION No. 2018
RECEIVED
JULY 27, 1937
OFFICE OF STATE ENGINEER
SANTA FE, NEW MEXICO

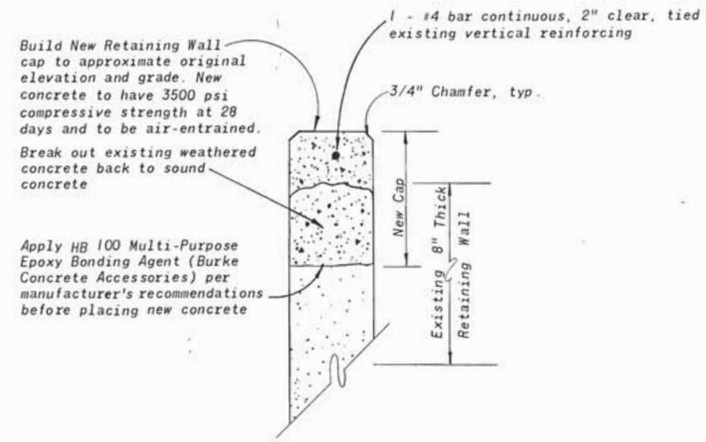
REPAIRS TO BROOD POND NO. 3 SPILLWAY

PARKVIEW FISH HATCHERY — RIO ARRIBA COUNTY, N.M.

NOTE: Fish Screen not shown.

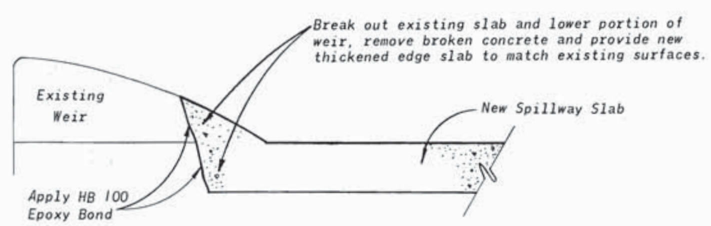
SCHEDULE OF QUANTITIES

Spillway Slab Concrete	12 cubic yards
Epoxy	60 gallons
Welded Wire Mesh	440 square feet
Expansion Joint Dowels	11 each



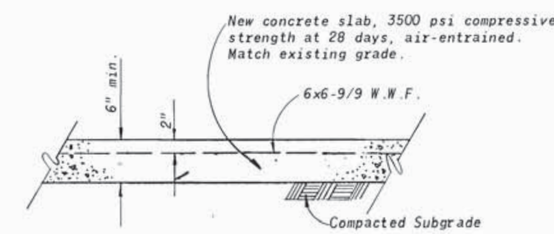
NEW RETAINING WALL CAP

1 1/2" = 1'-0"



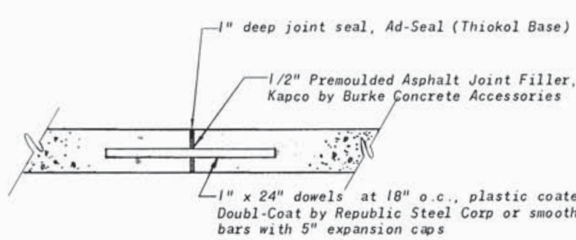
NEW SPILLWAY SLAB AT WEIR

No Scale



NEW SPILLWAY SLAB

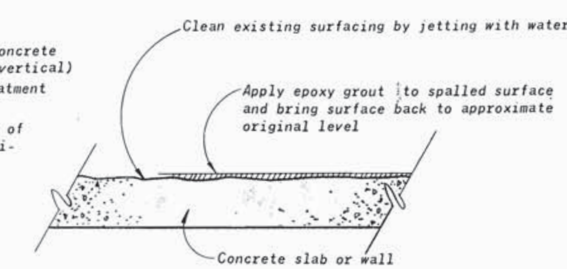
1" = 1'-0"



TYPE 'A' EXPANSION JOINT DETAIL

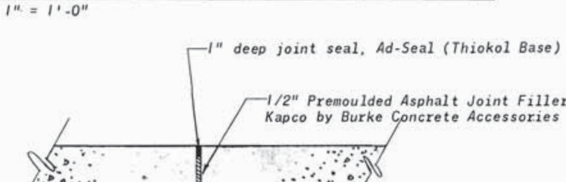
1" = 1'-0"

NOTES:
1. All existing distressed concrete surfaces (horizontal and vertical) shall receive surface treatment application.
2. Epoxy grout shall consist of Hunt Process, HB 100 Multi-Purpose Epoxy Bonding Agent mixed with sand with a minimum ratio of adhesive (epoxy) to aggregate of 1:4 by volume.



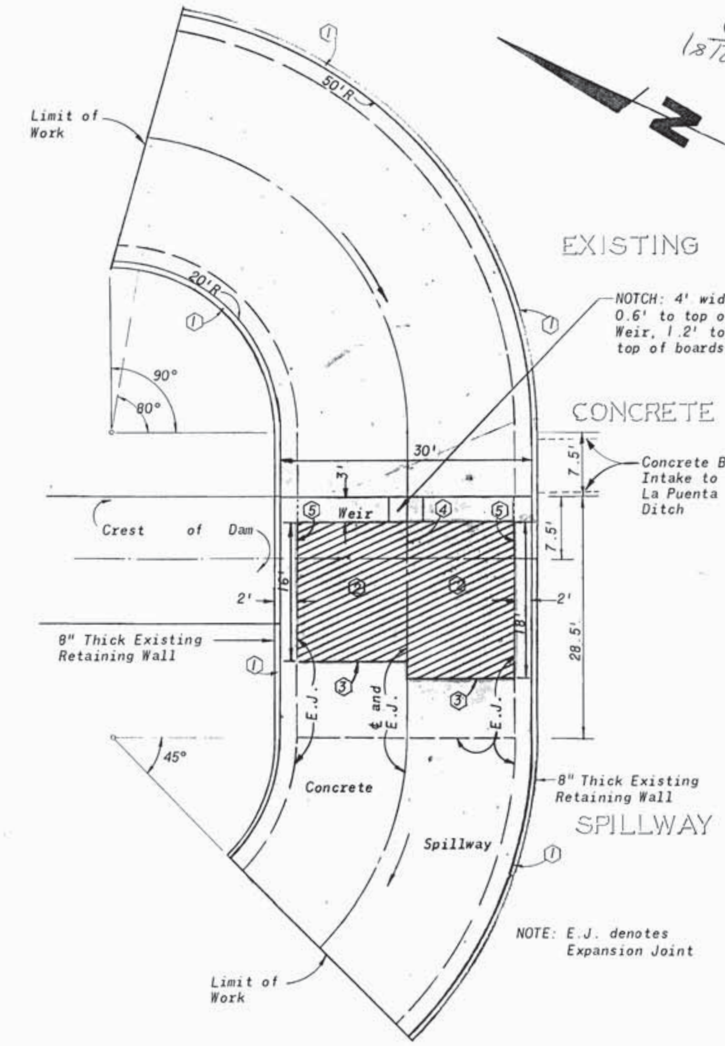
SURFACE TREATMENT DETAIL

No Scale



TYPE 'B' EXPANSION JOINT DETAIL

1" = 1'-0"



PLAN OF SPILLWAY

1" = 10'

1. Build new retaining wall cap - see detail this sheet.
2. Cross-hatched area indicates concrete slab to be removed and re-constructed per details this sheet.
3. Existing mortar filled construction joint to be westerly limits of new concrete slab.
4. Build Type 'A' Expansion Joint along spillway ℄.
5. Build Type 'B' Expansion Joint along retaining wall footing.

ENGINEER'S CERTIFICATE

STATE OF NEW MEXICO
COUNTY OF BERNALILLO

I, Thomas O. Isaacson, being first duly sworn upon my oath, state that I am a registered professional engineer, qualified in civil engineering and that the accompanying plans and specifications consisting of one sheet of plans was prepared under my supervision and direction.

License No. 3895

Thomas O. Isaacson
Thomas O. Isaacson
Registered Professional Engineer

Subscribed and sworn to before me this 9th day of July, 1979.
My commission expires: 11-23-79

Notary Public

REGISTERED PROFESSIONAL ENGINEER AND LAND SURVEYOR — NEW MEXICO — No. 3895 — THOMAS O. ISAACSON

CCIC Chambers Campbell Isaacson Chaplin Inc.

JOB NO. 379-4
DATE JUNE 1979
SHEET 1 OF 1

Appendix B

Photographs of Laguna Del Campo Dam

Laguna Del Campo Dam: Photos Taken 12/13/2010

Photo B-1 Upstream slope looking left from right end of main embankment

Photo B-2 Main reservoir, looking downstream from cross dike

Photo B-3 Inflow from abandoned diversion of No Name Ditch into upper reservoir, looking left

Photo B-4 No Name Ditch inflow into upper part of reservoir

Photo B-5 No Name Ditch, looking upstream

Photo B-6 Gate operator looking left from main embankment

Photo B-7 Discharge channel looking downstream from dam crest

Photo B-8 Emergency spillway weir looking left from dam crest

Photo B-9 Fish screen upstream of weir in emergency spillway channel

Photo B-10 Chute downstream of emergency spillway weir

Appendix C

Flood Hydrology Manual (Cudworth, 1989) - Select Pages

CWV

Flood Hydrology Manual

A Water Resources Technical Publication

by

Arthur G. Cudworth, Jr.

Surface Water Branch
Earth Sciences Division

FIRST EDITION 1989

Available from:

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U.S. Department of the Interior
Bureau of Reclamation
Denver Office

(2) *S-graph technique* [56,57].—Unit hydrographs developed from recorded events are converted to dimensionless form as follows:

a. A summation hydrograph is initially developed by algebraically adding the ordinates of a continuous series of identical unit hydrographs, each successively out of phase by one unit period. The lag time for this particular technique is determined by reading from the plotted summation hydrograph, the elapsed time from the beginning of rainfall to the time when 50 percent of the ultimate discharge is reached.

b. The dimensionless hydrograph is then developed from the summation hydrograph by converting the time base (abscissa) to time in percent of lag time and converting the ordinate values to discharge as a percent of the ultimate discharge.

(e) Development of Synthetic Unit Hydrographs.—In chapter 2, considerable attention was given to the specific observations that should be made during a field inspection of a drainage basin. Observations made relative to the basin's drainage network or hydraulic system form the primary basis for establishing an appropriate K_n value [10] to be used in estimating the synthetic unit hydrograph lag time. In assigning a K_n value for a particular basin, consideration should also be given to K_n values developed from analyses of observed flood hydrographs for basins that are similar with respect to general topography, to channel and flood plain characteristics, and to drainage network density.

Once the value of K_n has been estimated, the length of the longest watercourse, L, and the length along the longest watercourse to a point opposite the centroid of the drainage basin, L_{ca}, are measured. A suitable topographic map such as a USGS quadrangle map is usually used for these measurements. The slope of the longest watercourse, S, is also determined using contour data from the topographic map. The drainage basin's physical parameters K_n, L, L_{ca}, and S are then entered into the general lag equation (1):

$$L_g = 26\,K_n \left(\frac{LL_{ca}}{S^{0.5}} \right)^{0.33} \qquad (4)$$

where:

L_g = lag time, in hours;
L = distance of longest watercourse, in miles;
L_{ca} = distance from gauging station to a point opposite centroid of drainage basin, in miles;
S = overall slope of L measured from gauging station or point of interest to drainage basin divide, in feet per mile; and
K_n = a trial value based on an estimate of the weighted, by stream length, average Manning's n value for the principal watercourses in the drainage basin.

Equation (4) yields the synthetic unit hydrograph lag time in hours. The results of applying this equation are considered adequate for either the dimensionless unit hydrograph or S-graph approach:

$$(\text{Lag Time})\text{S-Graph} \approx (\text{Lag time} + \text{Semiduration})\text{Dimensionless Graph}$$

To aid in determining an appropriate lag time, many flood hydrograph reconstitutions have been examined. These reconstitutions represent flood runoff from natural basins throughout the conterminous United States west of the Mississippi River and from urbanized basins for several locations throughout the States. Data for urbanized basins are included in this manual because of the increased interest in the flood hydrology of such areas, particularly with respect to the impact on runoff from various levels of development.

As a result of the examination of these reconstitutions, 162 flood hydrographs considered representative of surface runoff from rainfall events were selected for analysis relative to regionalized trends in the lag time relationships and the time versus variation of discharge relationships. Those hydrographs not included were considered to represent either interflow runoff or runoff that included significant contributions from snowmelt. The 162 hydrographs were then segregated on a regional and topographic basis, as shown on figures 4-6 through 4-11. The supporting data for these figures are listed in tables 4-1 through 4-6. These tables include the station index number, station name and location, drainage area (in some cases, only the area contributing to flood runoff), basin factor $LL_{ca}/S,^{0.5}$ unit hydrograph lag time determined from the flood hydrograph reconstitution, computed K_n value, and the C_t constant in equation (2) which is equal to 26 K_n. These data may be used as a guide during the field reconnaissance to establish an appropriate K_n value for the drainage basin being studied. As previously stated, it is of considerable value to conduct a field reconnaissance of the basins represented in the data set to gain an understanding of the physical conditions that are indicative of a particular K_n value.

Figure 4-6 and the data in table 4-1 represent conditions on the Great Plains west of the Mississippi River and east of the foothills of the Rocky Mountains. The relationships shown on figure 4-6 reflect K_n values from 0.069 to as low as 0.030, which result in lag equation coefficients C_t of 1.8 and 0.77, respectively. The upper limit values generally reflect basins with considerable overland flow before reaching moderately well-defined watercourses. Many upper reach watercourses are swales, and the well-defined drainage networks are limited to the lower parts of the basins. Overbank flow conditions reflect relatively high Manning's n values. The lower limit values generally reflect a well-defined drainage network reaching points near the basin boundary, the overland flow occurs for fairly short distances before entering a well-defined watercourse, and the overbank conditions reflect relatively low Manning's n values.

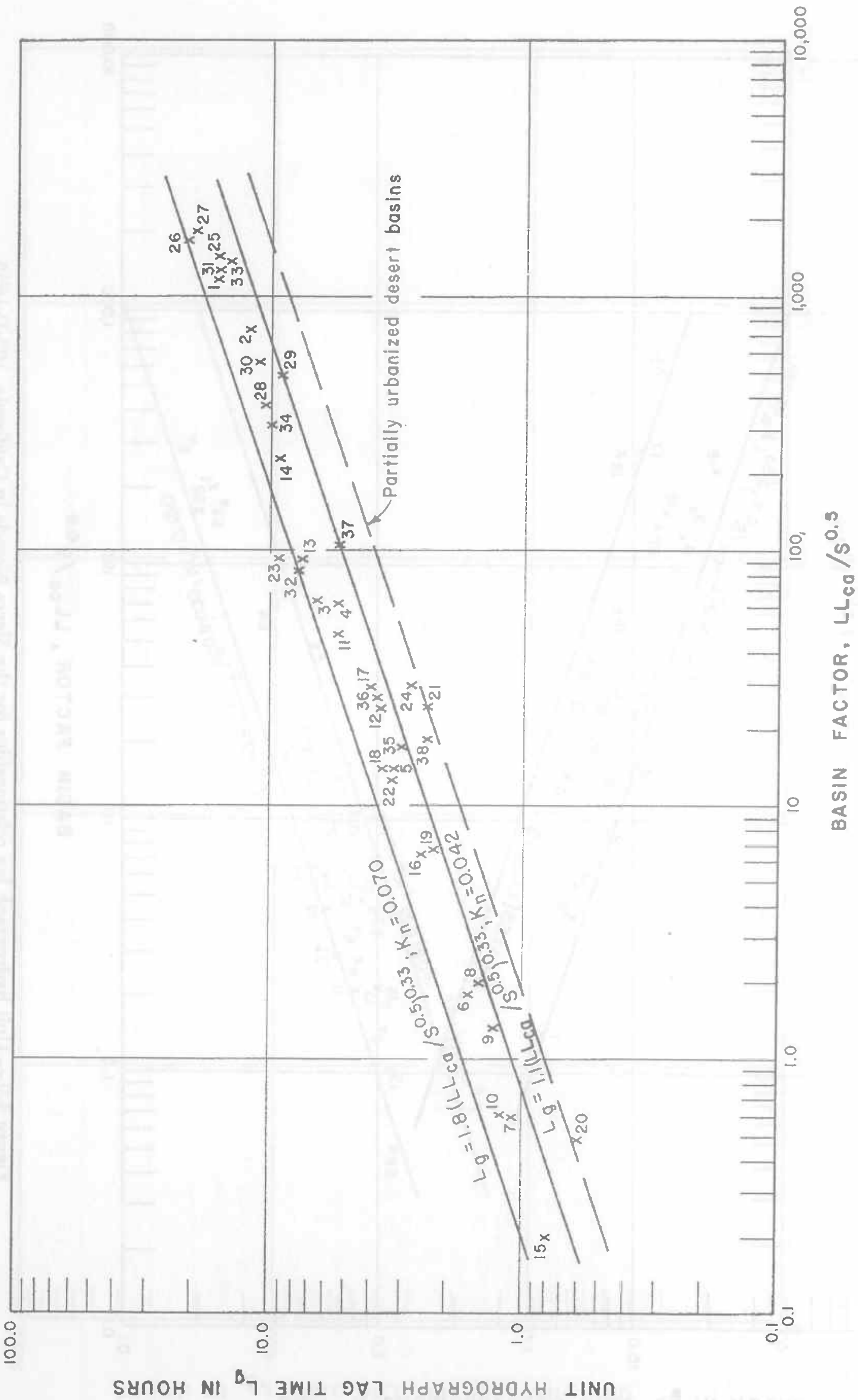


Figure 4-8.—Unit hydrograph lag relationships for the Southwest Desert, Great Basin, and Colorado Plateau. 103-D-1852.

Table 4-3.—Unit hydrograph lag data for the Southwest Desert, Great Basin, and Colorado Plateau.

Index No.	Station and location	Drainage area, mi²	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Salt River at Roosevelt, AZ	4341.0	1261.0	16.0	0.058	1.51
2	Verde R. above E. Verde and below Jerome, AZ	3190.0	760.0	12.0	.052	1.35
3	Tonto Cr. above Gun Cr., AZ	678.0	66.3	6.5	.063	1.64
4	Agua Fria R. nr. Mayor, AZ	590.0	63.2	5.4	.053	1.38
5	San Gabriel R. at San Gabriel Dam, CA	162.0	14.4	3.3	.053	1.38
6	West Fk. San Gabriel R. at Cogswell Dam, CA	40.4	1.8	1.6	.051	1.33
7	Santa Anita Cr. at Santa Anita Dam, CA	10.8	0.6	1.1	.050	1.30
8	Sand Dimas Cr. at San Dimas Dam, CO	16.2	2.0	1.5	.046	1.20
9	Eaton Wash at Eaton Wash Dam, CA	9.5	1.3	1.3	.046	1.20
10	San Antonio Cr. nr. Claremont, CA	16.9	0.6	1.2	.055	1.43
11	Santa Clara R. nr. Saugus, CA	355.0	48.2	5.6	.060	1.56
12	Temecula Cr. at Pauba Canyon, CA	168.0	24.1	3.7	.050	1.30
13	Santa Margarita R. nr. Fallbrook, CA	645.0	99.2	7.3	.062	1.61
14	Santa Margarita R. at Ysidora, CA	740.0	228.0	9.5	.061	1.59
15	Live Oak Cr. at Live Oak Dam, CA	2.3	0.2	0.8	.052	1.35
16	Tujunga Cr. at Big Tujunga Dam, CA	81.4	6.5	2.5	.052	1.35
17	Murrieta Cr. at Temecula, CA	220.0	28.9	4.0	.051	1.33
18	Los Angeles R. at Sepulveda Dam, CA	152.0	14.3	3.5	.056	1.46
19	Pacoima Wash at Pacoima Dam, CA	27.8	6.8	2.4	.049	1.27
20	East Fullerton Cr. at Fullerton Dam, CA	3.1	0.5	0.6	.029	0.75
21	San Jose Cr. at Workman Mill Rd. CA	81.3	24.8	2.4	.032	0.83
22	San Vincente Cr. at Foster, CA	75.0	12.8	3.2	.053	1.38
23	San Diego R. nr. Santee, CA	380.0	95.4	9.2	.078	2.03
24	Deep Cr. nr. Hesperia, CA	137.0	28.1	2.8	.036	0.94
25	Bill Williams R. at Planet, AZ	4730.0	1476.0	16.2	.056	1.46
26	Gila R. at Conner No. 4 Damsite, AZ	2840.0	1722.0	21.5	.071	1.85
27	San Francisco R. at Jct. with Blue R., AZ	2000.0	1688.0	20.6	.068	1.77
28	Blue R., nr. Clifton, AZ	790.0	352.0	10.3	.057	1.48
29	Moencopi Wash nr. Tuba City, AZ	2490.0	473.0	9.2	.046	1.20
30	Clear Cr. nr. Winslow, AZ	607.0	570.0	11.2	.053	1.38
31	Puerco R. nr. Admana, AZ	2760.0	1225.0	15.9	.058	1.51
32	Plateau Cr. nr. Cameo, CO	604.0	89.9	7.9	.069	1.79
33	White R. nr. Watson, UT	4020.0	1473.0	15.7	.054	1.40
34	Paria R. at Lees Ferry, AZ	1570.0	296.0	10.2	.060	1.56
35	New River at Rock Springs, AZ	67.3	16.5	3.1	.047	1.22
36	New River at New River, AZ	85.7	26.3	3.7	.048	1.25
37	New R. at Bell Road nr. Phoenix, AZ	187.0	108.0	5.3	.043	1.12
38	Skunk Cr. nr. Phoenix, AZ	64.6	18.7	2.4	.035	0.91

approaching the ultimate infiltration rate. The ultimate or final infiltration rate is theoretically equal to the saturated hydraulic conductivity because tension gradients are no longer present and the hydraulic gradient is due solely to gravity.

When the precipitation rate is less than the soil profile's infiltration capacity, infiltration rates are identical to the precipitation rates and no water is available for surface runoff. Since the infiltrating water alters the water content in the soil profile, the infiltration capacity also changes. At some point during a precipitation event, precipitation rates may exceed the infiltration capacity of the soil, which results in ponding and/or surface runoff. Continued rainfall will then produce the characteristic decay or decrease in infiltration rates. This phenomenon, as it relates to severe flood occurrences, can be represented by a decay curve function where the infiltration capacity rather than rainfall rates control the infiltration rates. In 1940, Horton [58] proposed the following mathematical relationship to represent this function:

$$f = f_c + (f_o - f_c)\, e^{-kt} \tag{5}$$

where:

f = resulting infiltration rate at time t, in hours;
f_c = minimum or ultimate infiltration rate, in inches per hour;
f_o = initial rate of infiltration capacity, in inches per hour;
e = base of natural logarithm;
k = a constant dependent primarily on soil type and vegetation; and
t = time from start of rainfall, in hours.

In the development of the PMF, the hydrologic engineer is primarily concerned with the magnitude of f_c in equation (5).

Many attempts have been made to measure infiltration rates using devices known as infiltrometers. When infiltration rates developed from the infiltrometer tests are compared with those from observed flood hydrograph reconstitutions, the test results from the infiltrometer are almost always higher. Bureau hydrologic engineers consider rates resulting from reconstitution studies to be more valid because they tend to reflect the integrated infiltration rates for the various soil conditions over the entire drainage basin.

The SCS (Soil Conservation Service) has proposed the subdivision of soils into four groups relative to their respective infiltration capacities or ultimate infiltration rates. The ultimate or minimum infiltration rates of these four groups have been found by the Bureau to be in reasonably close agreement with the rates resulting from observed flood hydrograph reconstitution studies. When more than one group of soils is present in a drainage basin, an average value for the basin should be calculated based on weighted areas. The four groups as generally defined by the SCS are:

(1) *Group A soils* (low runoff potential).—Soils that have high infiltration rates even when thoroughly wetted, and consisting mostly of well- to excessively well-drained sands or gravels. These soils have a high rate of water transmission. Ultimate infiltration rates for these soils have been found to range from 0.3 to 0.5 inch per hour.

(2) *Groups B soils.*—Soils having moderate infiltration rates when thoroughly wetted, and consisting mostly of moderately deep to deep, moderately well- to well-drained soils with moderately fine to moderately coarse textures, which would include sandy loams and shallow loess. These soils may also include moderate organic matter. Ultimate infiltration rates for these soils range from 0.15 to 0.30 inch per hour.

(3) *Group C soils.*—Soils having slow infiltration rates when thoroughly wetted, and consisting mostly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission, and include many clay loams, shallow sandy loams, soils low in organic matter, and soils usually high in clay content. The minimum or ultimate infiltration rates for these soils range from 0.05 to 0.15 inch per hour.

(4) *Group D soils* (high runoff potential).—Soils having very slow infiltration rates when thoroughly wetted, and consisting mostly of clay soils with high swelling potential, soils with a permanent high-water table, soils with a claypan (e.g., desert pavement) or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of transmission, and include heavy plastic clays and certain saline soils. Minimum infiltration rates range from 0 to 0.05 inch per hour.

Hydrologic analyses leading to PMF estimates should be based on the assumption that minimum or ultimate infiltration rates prevail throughout the duration of the PMS. This assumption is based on consideration of conditions that have been shown to exist prior to extreme storm events. Examination of historical conditions have shown that it is entirely reasonable to expect one or more storms antecedent (prior) to the extreme event due to meteorologic persistence. Accordingly, it is reasonable to assume that any antecedent storm has satisfied any soil moisture deficiencies and initial losses, and that infiltration rates would be at the minimum or ultimate rate at the onset of the PMS.

(k) Base Flow and Interflow.—The base flow and interflow components to a flood hydrograph are graphically shown on figure 4-14. The base flow component generally consists of the water reaching a basin's watercourses after flowing a considerable distance underground as ground water. The base flow is generally depicted as a recession curve, which indicates a gradually decreasing rate of flow. This flow continues to decrease until the water surface in the stream is in a state of equilibrium

Appendix D

HMR 55A Materials

URS **CALCULATION COVER SHEET** Quality

Project Name:	*Laguna Del Campo Dam*	**Project Number:**	*22242013*
Project Location:	*Rio Arriba, NM*	**Client Name:**	*US Fish and Wildlife Service*
PM Name:	*Gregg Batchelder-Adams*	**PIC Name:**	*John France*

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages *(including cover sheet): 19*

Discipline: *Dam EAP*

Title of Calculation: *Probable Maximum Precipatation (PMP) - HMR-55A in Laguna Del Campo Dam, NM*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Brad Rastall*

DESCRIPTION & PURPOSE

To estimate the PMP events for Laguna Del Campo Dam watershed

BASIS / REFERENCE / ASSUMPTIONS

NOAA HMR-55A (1988)

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	01.07.2011	[ ]	[ ]	[ ]
1		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

Shih — Originator Signature — Date: 01.07.2011

Bradley Rastall — Checker Signature — Date: 1/7/11

Project Manager Signature — Date: 1/8/11

Distribution:

Project Central File – Quality file folder

Other Specify: ____

Purpose:

1. Determine the probable maximum precipitation (PMP) for the general and local storm events in the Laguna Del Campo Dam watershed using HMR 55A.
2. Obtain the frequency-duration-depth of the frequency storms from NOAA Atlas 14.

References:

NOAA, USACE, and USBR. *Hydrometeorological Report No. 55A*. Silver Spring, MD. June 1988.

NOAA, U.S. Department of Commerce, National Weather Service, *NOAA Atlas 14 - Precipitation-Frequency Atlas of the United States, Silver Spring, MD. 2004, revised 2006.*

USGS 1/3-Arc Second National Elevation Dataset, 2009.

USGS High Resolution State Orthoimagery for New Mexico 2005

Files:

General Storm Estimate:

The steps taken to calculate the general storm PMP, as described in HMR 55A, are in the table below. Each step will be discussed individually.

Step	Description							
1	CATCHMENT AREA (mile^2)		5.7					
2	DURATION (HR)	0	1	6	24	72		
	PMP (in.)	0	4.5	9.3	17.0	22		
	(Duration-Area PMP Maps)							
3	PMP ESTIMATES FOR INTERMEDIATE DURATIONS							
	DURATION (HR)	1	6	12	18	24	30	36
	PMP (in.)	4.5	9.3	13.0	15.5	17.0	17.7	18.5
	DURATION (HR)	42	48	54	60	66	72	
	PMP (in.)	19.0	19.5	20.0	20.8	21.3	22.0	
4	INCREMENTAL PMP ESTIMATES							

Duration (hr)	Incremental Precipitation (in)	Cumulative Precipitation (in)
0	0	0
1	4.5	4.5
6	4.8	9.3
12	3.7	13.0
18	2.5	15.5
24	1.5	17.0
30	0.7	17.7
36	0.8	18.5
42	0.5	19.0
48	0.5	19.5
54	0.5	20.0
60	0.8	20.8
66	0.5	21.3
72	0.7	22.0

Step #1: The basin catchment area is 5.75 square miles, calculated using Geographic Information System (GIS) software. USGS national elevation database (1/3-Arc Second) was used to perform the watershed delineation. Figure 1 shows the delineated watershed and aerial image.

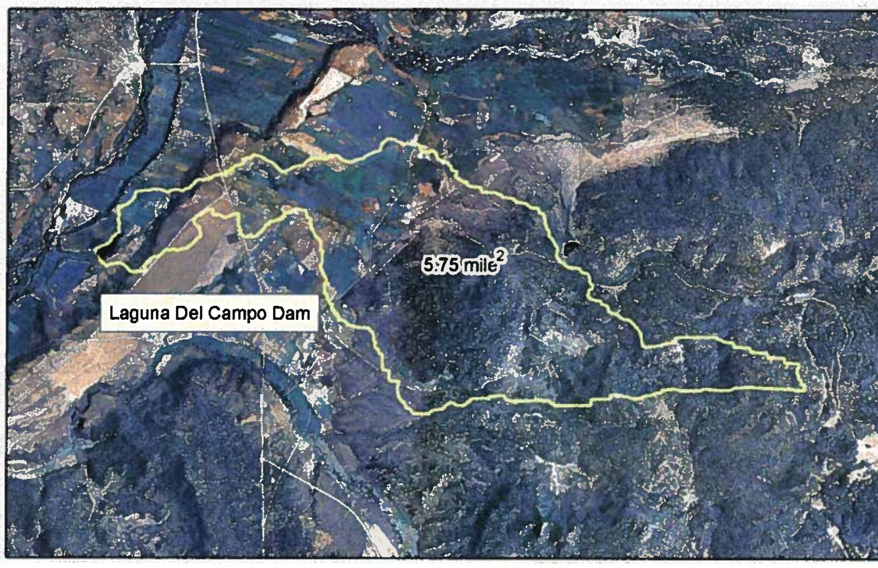


Figure 1. Delineated upstream catchment area of Laguna Del Campo Dam

Step #2: Refer to Figure 2. The Laguna Del Campo Dam (Laguna Dam) watershed is located east of Continental Divide and in Colorado Plateaus Region. NOAA Hydrometeorological Report No. 55A (HMR 55A) shall be utilized to estimate the Probable Maximum Precipitation (PMP) for the Laguna Dam. Unadjusted PMP values were obtained from HMR 55A plates Ic, IIc, IIIc, and IVc (attached) for durations of 1, 6, 24, and 72 hours, respectively. The location of the Laguna Dam is shown on each plate.

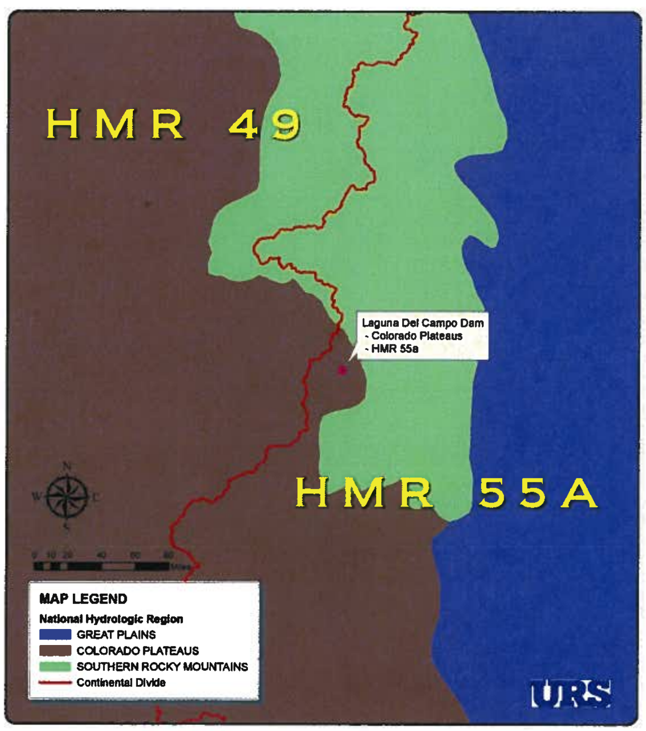


Figure 2. Hydrological region of Laguna Del Campo Dam watershed

Step #3: Since the catchment area is less than 10 square mile, the areal reduction factors are not required for PMP depth adjustment. A depth duration curve (Figure 3) was created based upon the obtained PMP depth from HMR-55A (solid marker symbol). This plot is shown below with duration in hours on the x-axis and cumulative precipitation in inches on the y-axis.

Step #4: PMP values were interpolated using the depth duration curve to estimate precipitation amounts every 6 hours. The values indicated with a hollow marker symbol (Figure 3) were obtained from the interpolation process. The final HMR 55A general storm PMP curve is shown on Figure 3 with duration in hours on the x-axis and precipitation in inches on the y-axis.

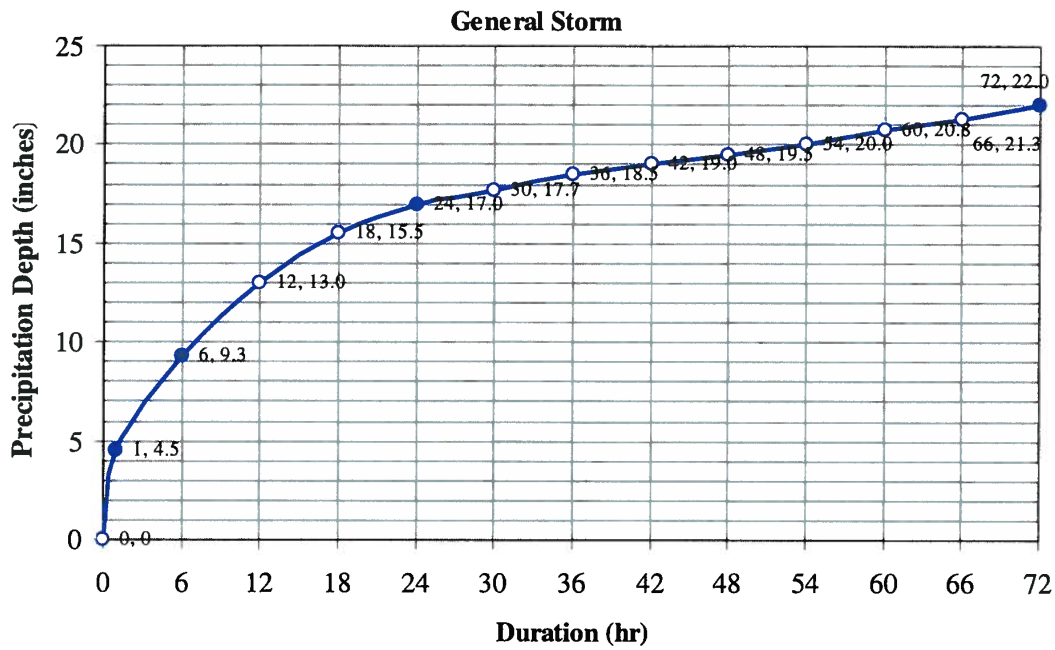


Figure 3. HMR 55A General Storm PMP curve of Laguna Del Campo Dam watershed

Rainfall distributions based on central-peaking and 2/3 peaking were prepared to present the rainfall depth distribution of the site general PMP event. Table 1 and Figures 4 and 5 show the 72-hour general PMP depth distribution in the Laguna Dam catchment.

Table 1. 72hr general PMP distribution in the Laguna Dam catchment.

Time (hr)	Incremental Precipitation (inch)			Time (hr)	Incremental Precipitation (inch)	
	Central-Peaking	2/3-Peaking			Central-Peaking	2/3-Peaking
0	0.00	0		4.50	0.31	4.50
1	0.04	0.04		1.20	0.33	1.20
2	0.04	0.04		0.93	0.38	0.93
3	0.06	0.04		0.72	0.43	0.72
4	0.06	0.06		0.63	0.48	0.63
5	0.06	0.06		0.54	0.54	0.54
6	0.07	0.06		0.46	0.63	0.46
7	0.08	0.06		0.38	0.68	0.38
8	0.08	0.07		0.33	0.80	0.33
9	0.08	0.08		0.29	0.93	0.29
10	0.09	0.08		0.23	1.20	0.23
11	0.09	0.08		0.20	1.44	0.20
12	0.09	0.08		0.18	4.50	0.18
13	0.11	0.09		0.18	0.96	0.18
14	0.11	0.09		0.17	0.72	0.17
15	0.11	0.09		0.15	0.56	0.15
16	0.12	0.09		0.15	0.46	0.15
17	0.13	0.11		0.13	0.38	0.13
18	0.13	0.11		0.13	0.29	0.13
19	0.13	0.11		0.13	0.21	0.13
20	0.13	0.11		0.12	0.18	0.12
21	0.15	0.12		0.11	0.17	0.11
22	0.15	0.12		0.11	0.15	0.11
23	0.17	0.13		0.11	0.13	0.11
24	0.18	0.13		0.09	0.13	0.09
25	0.20	0.13		0.09	0.13	0.09
26	0.21	0.13		0.09	0.11	0.09
27	0.28	0.15		0.08	0.11	0.08
28	0.31	0.15		0.08	0.09	0.08
29	0.38	0.15		0.08	0.09	0.08
30	0.43	0.17		0.07	0.08	0.07
31	0.48	0.18		0.06	0.08	0.06
32	0.56	0.18		0.06	0.07	0.06
33	0.68	0.20		0.06	0.06	0.06
34	0.80	0.20		0.04	0.06	0.04
35	0.96	0.23		0.04	0.04	0.04
36	1.44	0.28				

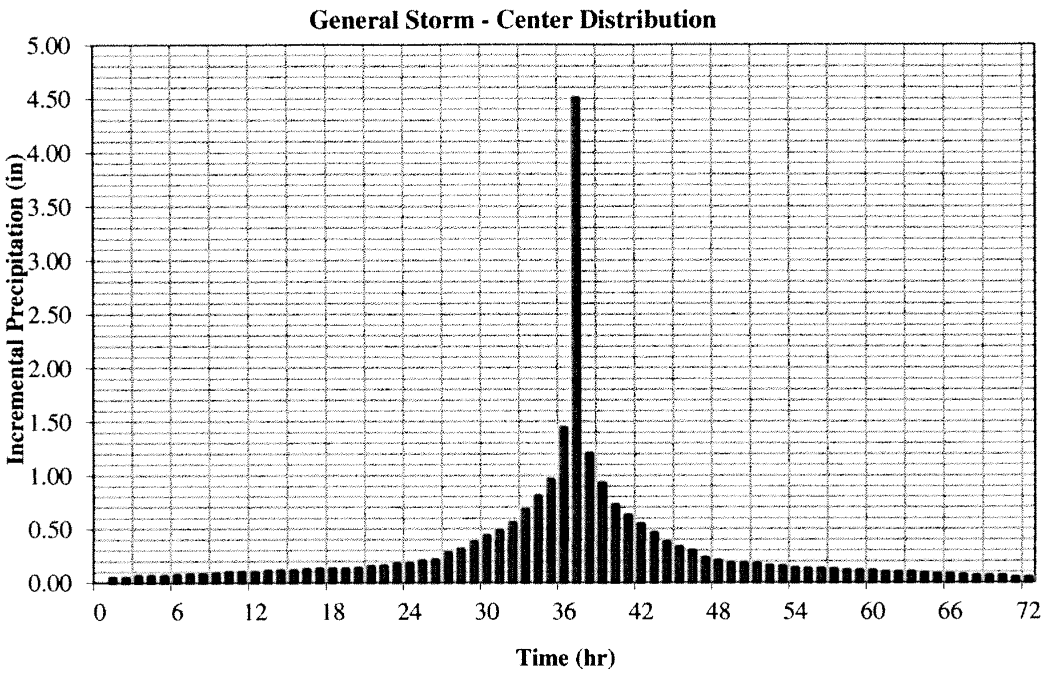


Figure 4. 72hr general PMP distribution in the Laguna Dam catchment (Central-Peaking).

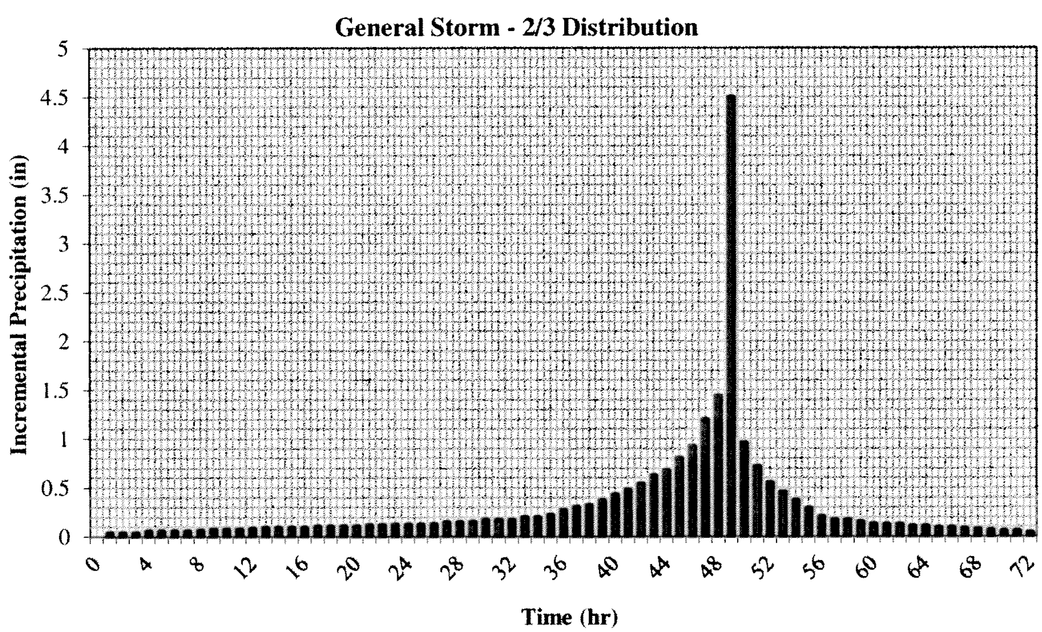


Figure 5. 72hr general PMP distribution in the Laguna Dam catchment (2/3-Peaking).

Local Storm Estimate:

The steps taken to calculate the local storm PMP are in the table below. Each step will be discussed individually.

Steps	Description	Quantities		
1	INDEX ESTIMATE AT 5000ft ELEV.			
	REFER TO PLATE VI c.	10.8	inches	
	(1-hr 1mile^2 Local PMP Maps)			
2	ADJUSTMENT FOR MEAN ELEVEATION OF DRAINAGE			
	REFER TO FIGURE 4.12 (p.80)	77.2	degrees F	
	REFER TO FIGURE 14.3 (p.219)	86.0%	AT ELEV. OF 7500 ft.	
3	INDEX PMP ESTIMATE AT MEAN ELVEVATION OF DRAINAGE	9.3	inches	
4	DEPTH DURATION CURVE			
	REFER TO TABLE 12.4 (p.200)	DURATION	PERCENT OF 1HR	PMP EST.
		0.25	0.68	6.3
		0.5	0.86	8.0
		0.75	0.94	8.7
		1	1	9.3
		2	1.16	10.8
		3	1.23	11.4
		4	1.28	11.9
		5	1.32	12.3
		6	1.35	12.5
5	AREAL REDUCTION FACTOR			
	REFER TO FIGURE 12.12 (p.203)	DURATION	REDUCTION FACTOR	
		0.25	82.0%	
		0.5	85.0%	
		0.75	87.0%	
		1	88.0%	
		2	90.0%	
		3	91.0%	
		4	92.0%	
		5	93.0%	
		6	93.5%	
6	PMP ESTIMATES (inches)			

Duration (hr)	Incr. PMP (in)	Cum PMP (in)
0	0.0	0
0.25	5.2	5.2
0.5	1.6	6.8
0.75	0.8	7.6
1	0.6	8.2
2	1.5	9.7
3	0.7	10.4
4	0.5	10.9
5	0.5	11.4

		6	0.3	11.7
7	INCREMENTAL PMP AMOUNTS			

Duration (hr)	Cum PMP (in)
0	0.0
0.25	5.2
0.50	6.8
0.75	7.6
1	8.2
2	9.7
3	10.4
4	10.9
5	11.4
6	11.7

<u>Step #1:</u> The index precipitation depth estimate for the basin was obtained using Plate VIc (attached).

<u>Step #2:</u> Adjustments for the mean elevation of the basin (7,500 feet) were determined using HMR 55A graphs. HMR-55A Figure 4.12 was used to determine the maximum dew point for August (August gives the largest dew point). The mean basin elevation and the dew point, obtained from Figure 4.12, were used on HMR-55A Figure 14.3 to determine the elevation adjustment percentage. These figures are attached.

<u>Step #3:</u> The elevation adjustment percentage (from Step #2) was then multiplied by the index estimate precipitation (from Step #1) to obtained an index estimate at the mean elevation of the drainage. This product is equal to 9.3 inches.

<u>Step #4:</u> A depth duration curve was then calculated. The precipitation percentages for each listed duration in Table HMR-55A 12.4 was multiplied by the index estimate at the mean elevation of drainage (9.3 inches) to develop the depth duration curve.

Table 12.4.--Percent of 1-hr local-storm PMP for selected durations for 6-/1-hr ratio of 1.35 (HMR No. 49)

Duration (hr)	Percent of 1 hr
1/4	.68
1/2	.86
3/4	.94
1	1.00
2	1.16
3	1.23
4	1.28
5	1.32
6	1.35

<u>Step #5:</u> Areal reduction factors for each duration of the local storm were estimated using HMR-55A Figure 12.12 (attached) based upon the watershed area.

<u>Step #6:</u> The Areal reduction factors (from Step #5) were multiplied by the depth duration amounts for each duration (from Step #4) to estimate the final PMP values. The final HMR 55A local storm PMP curve is shown on Figure 5 with duration in hours on the x-axis and precipitation in inches on the y-axis.

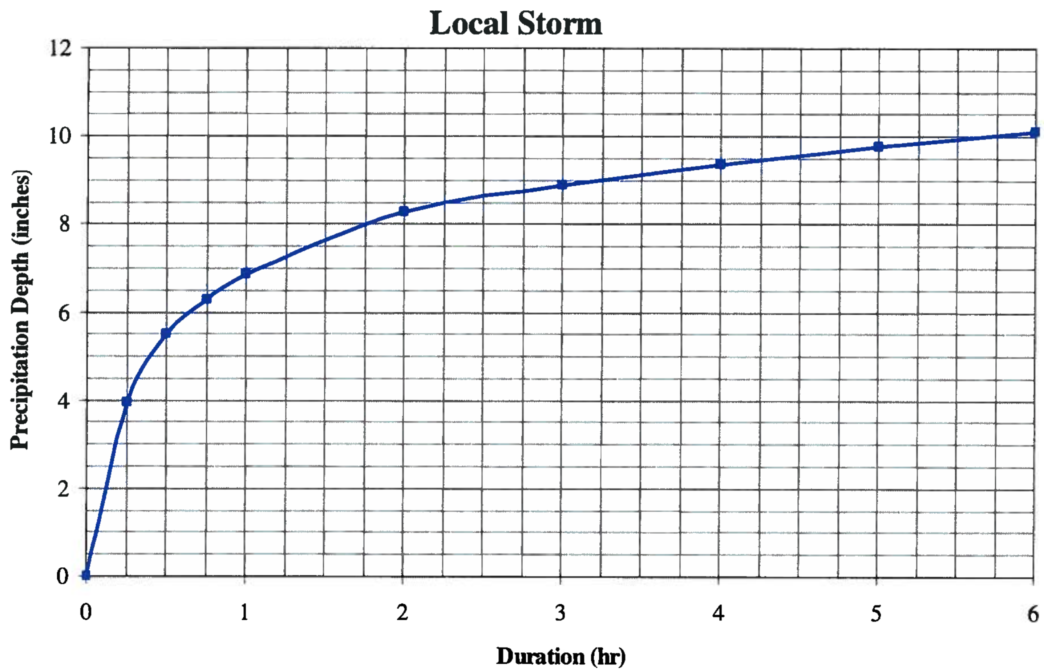


Figure 5. HMR 55A Local Storm PMP curve of Laguna Del Campo Dam watershed

For local PMP, New Mexico Office of the State Engineer suggests two rainfall distributions obtained from HMR 5 and USACE EM1110-2-1411. Both distributions should be evaluated to identify the critical event. The rainfall distributions of local PMP are shown Figures 6 and 7, and Tables 2 and 3.

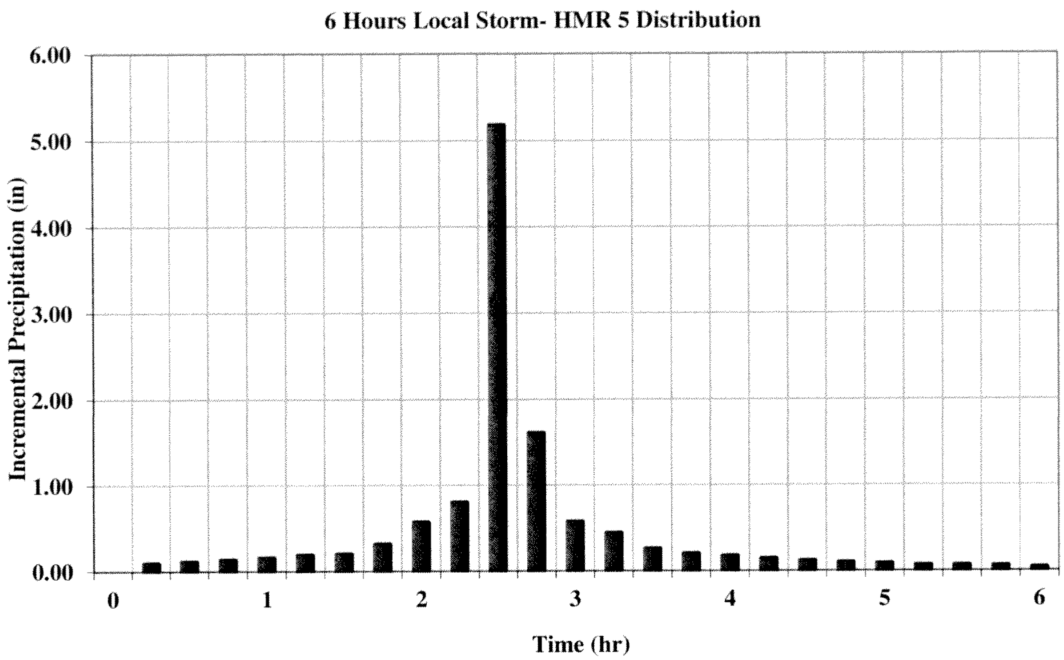


Figure 6. 6hr local PMP distribution in the Laguna Dam catchment (HMR 5 Distributed)

Time (hrs)	HMR No. 5 Distributed (inch)		Time (hrs)	HMR No. 5 Distributed (inch)
0	0.00		3.25	0.44
0.25	0.09		3.50	0.26
0.50	0.11		3.75	0.20
0.75	0.14		4.00	0.17
1.00	0.16		4.25	0.15
1.25	0.19		4.50	0.12
1.50	0.20		4.75	0.10
1.75	0.32		5.00	0.09
2.00	0.57		5.25	0.07
2.25	0.81		5.50	0.07
2.50	5.18		5.75	0.06
2.75	1.61		6.00	0.04
3.00	0.58			

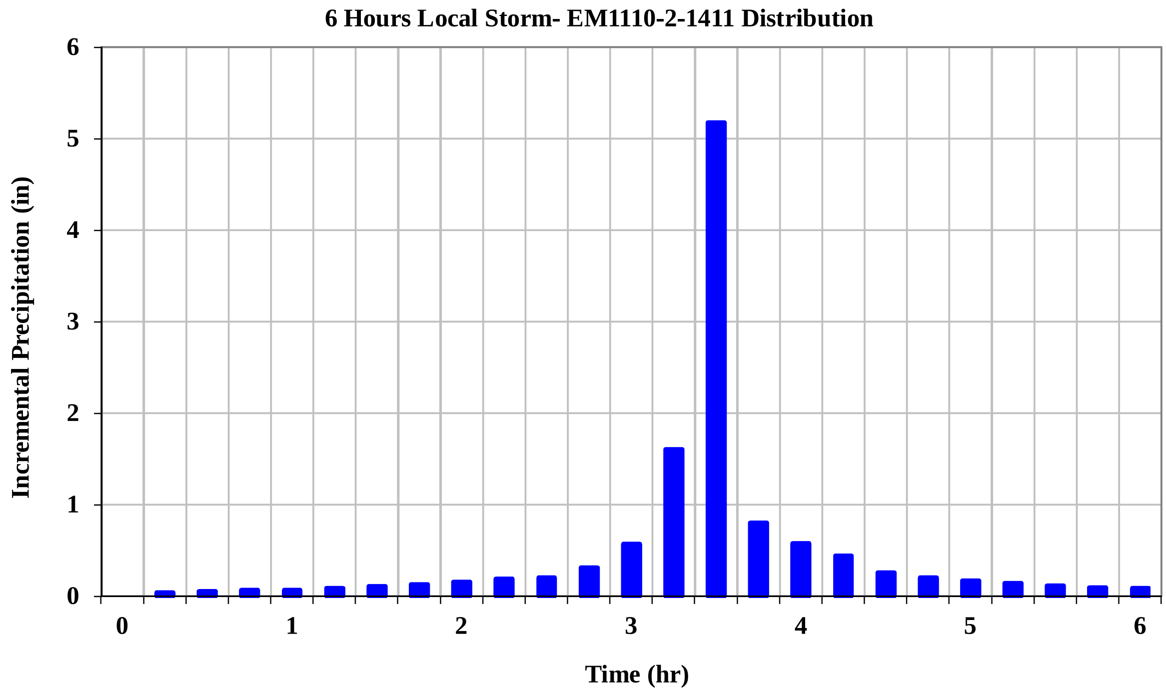
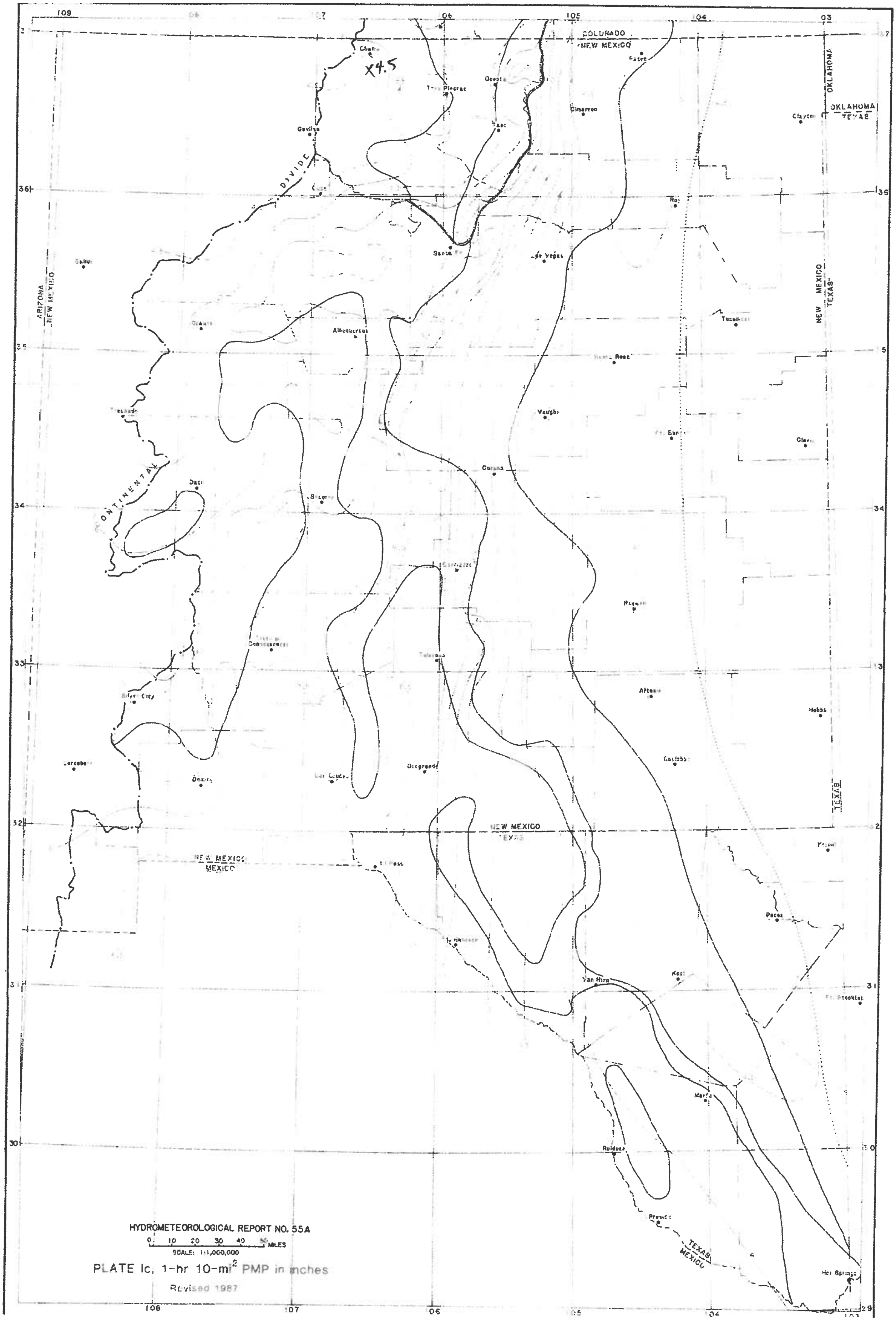


Figure 7. 6hr local PMP distribution in the Laguna Dam catchment (EM1110-2-1411 Distributed)

Time (hrs)	EM1110-2-1411 Distributed (inch)		Time (hrs)	EM1110-2-1411 Distributed (inch)
0	0		3.25	1.61
0.25	0.04		3.50	5.18
0.50	0.06		3.75	0.81
0.75	0.07		4.00	0.58
1.00	0.07		4.25	0.44
1.25	0.09		4.50	0.26
1.50	0.11		4.75	0.20
1.75	0.14		5.00	0.17
2.00	0.16		5.25	0.15
2.25	0.19		5.50	0.12
2.50	0.20		5.75	0.10
2.75	0.32		6.00	0.09
3.00	0.57			

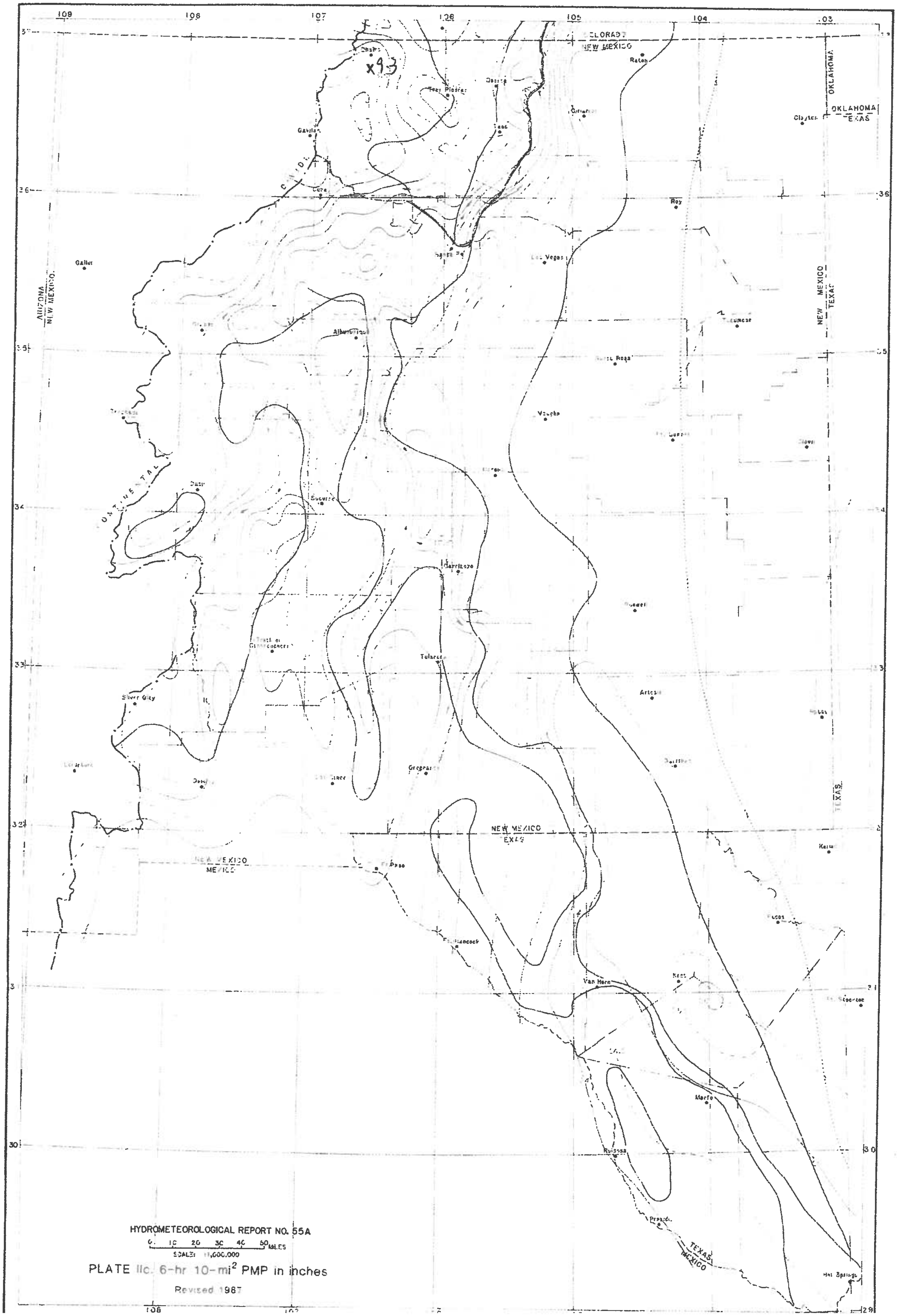


HYDROMETEOROLOGICAL REPORT NO. 55A

SCALE: 1:1,000,000

PLATE Ic, 1-hr 10-mi^2 PMP in inches

Revised 1987



HYDROMETEOROLOGICAL REPORT NO. 55A

PLATE IIc. 6-hr 10-mi^2 PMP in inches

Revised 1987

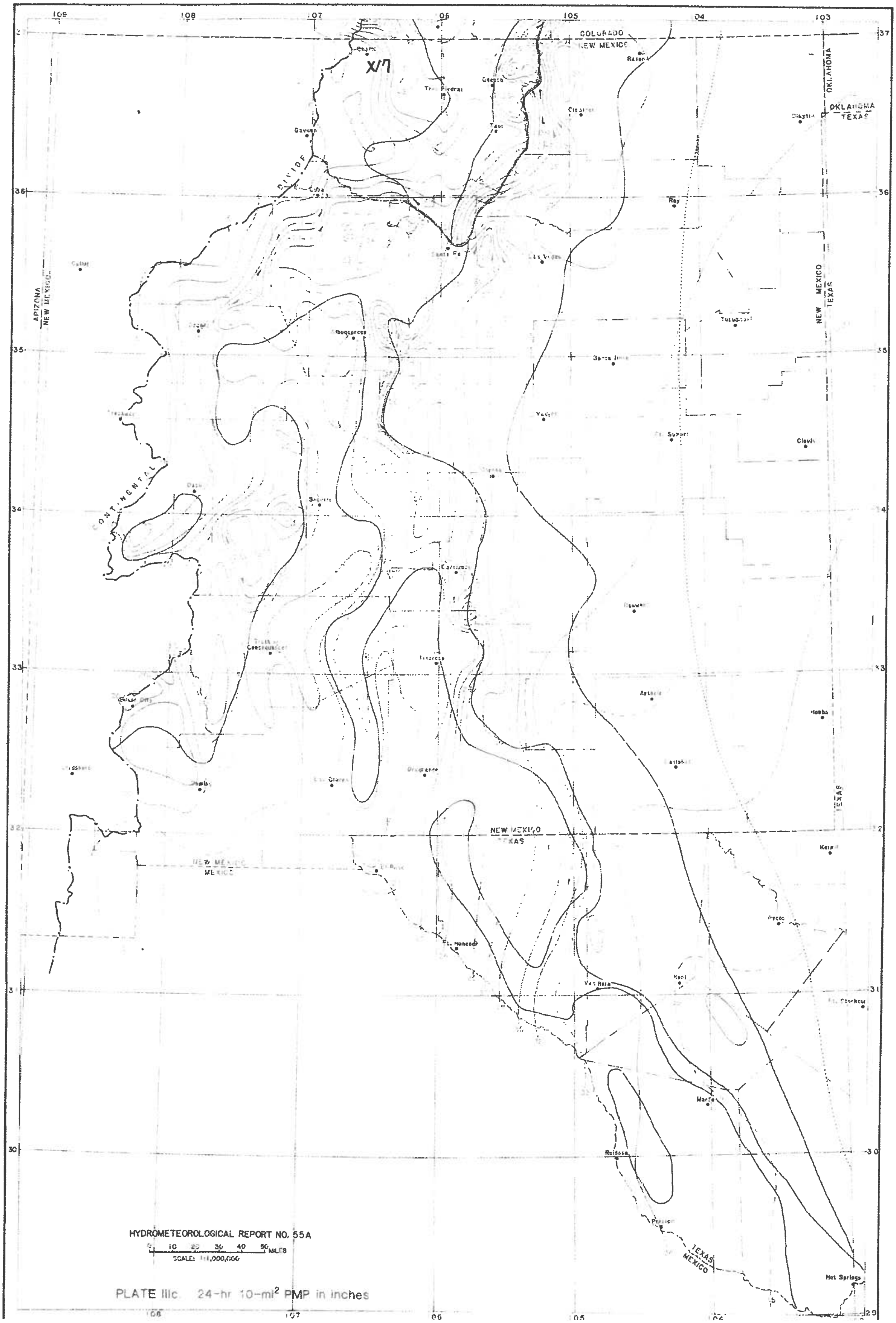


PLATE IIIc 24-hr 10-mi^2 PMP in inches

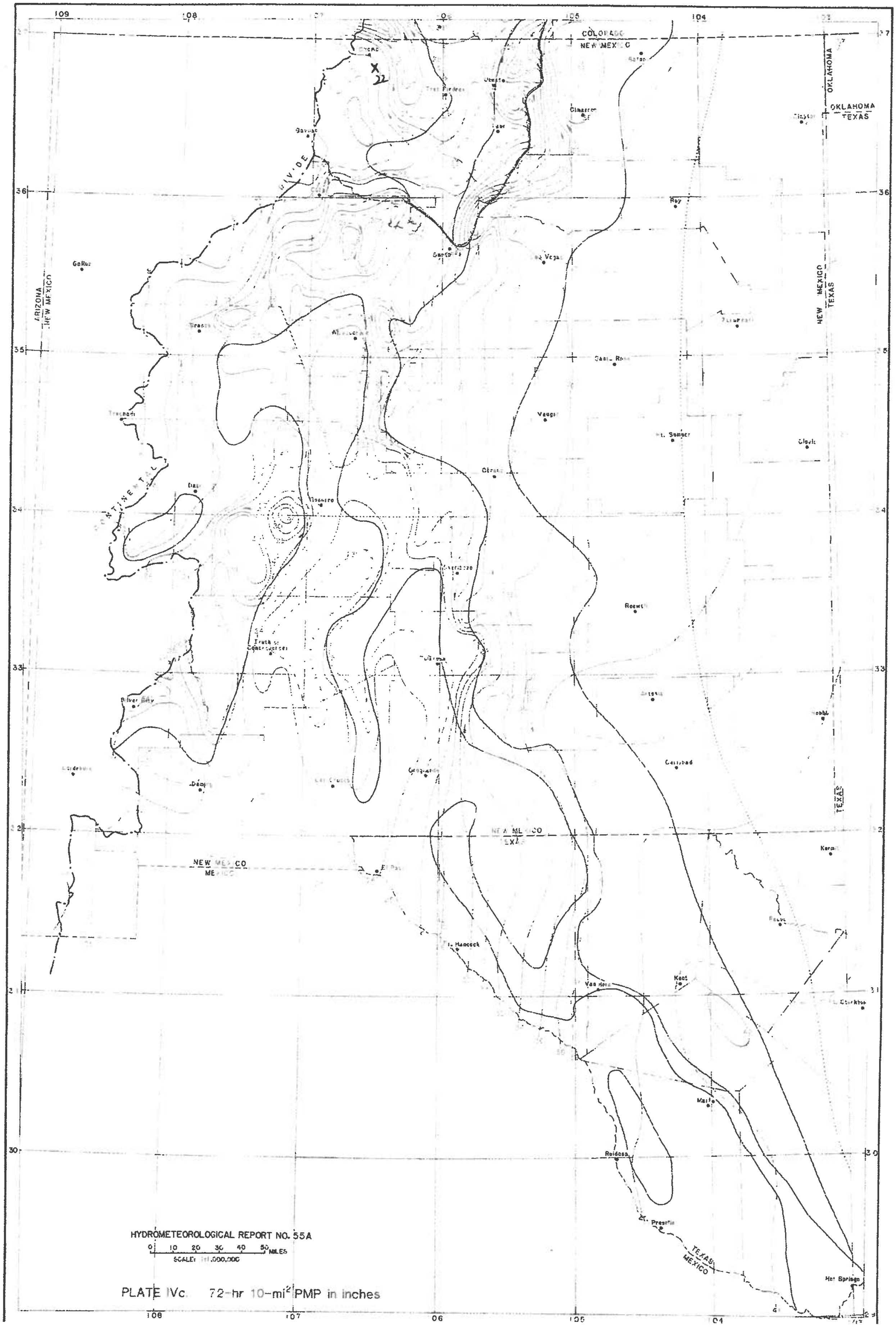
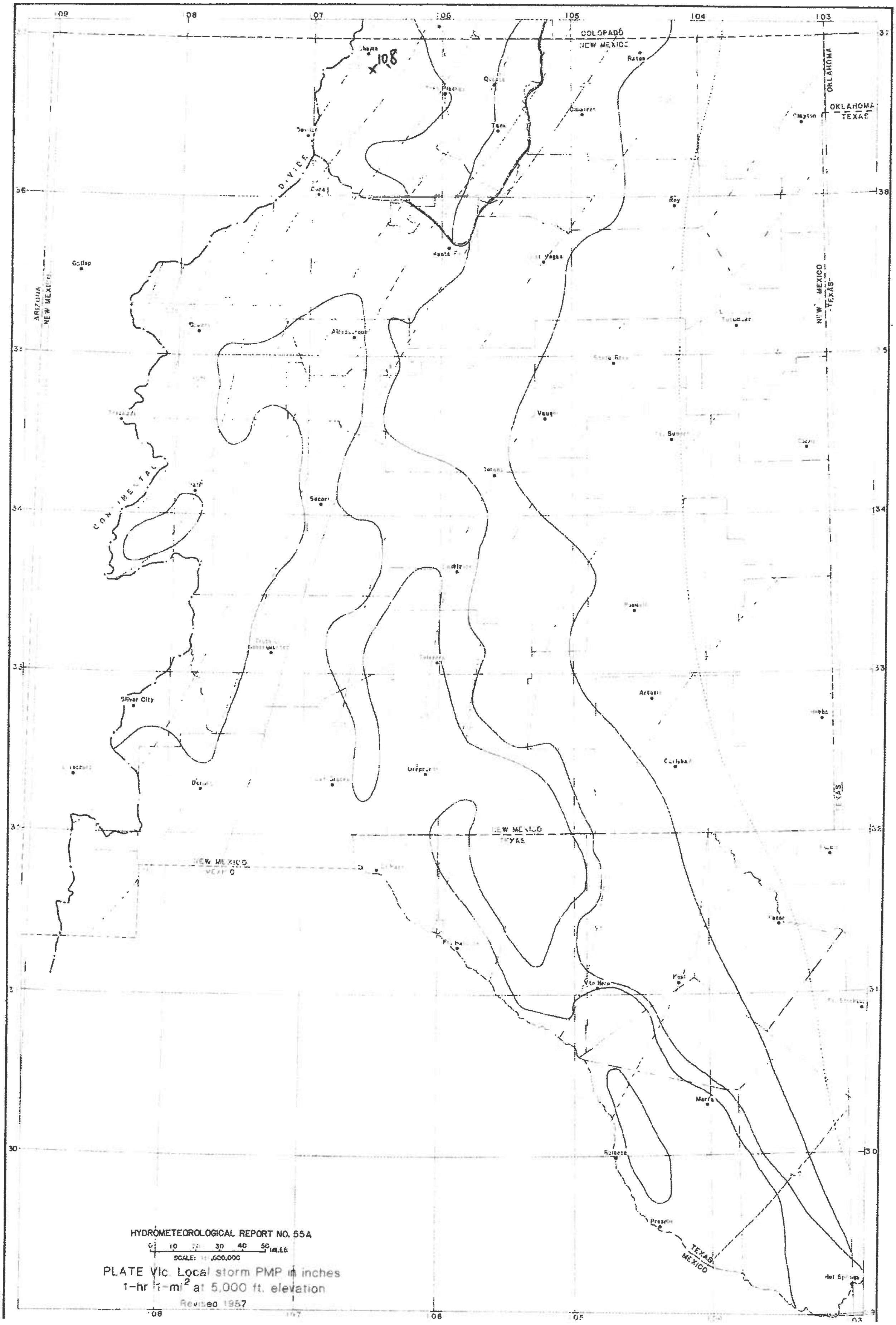


PLATE IVc. 72-hr 10-mi^2 PMP in inches



HYDROMETEOROLOGICAL REPORT NO. 55A

SCALE: 1:1,600,000

0 10 20 30 40 50 MILES

PLATE VIc. Local storm PMP in inches
1-hr 1-mi^2 at 5,000 ft. elevation
Revised 1957

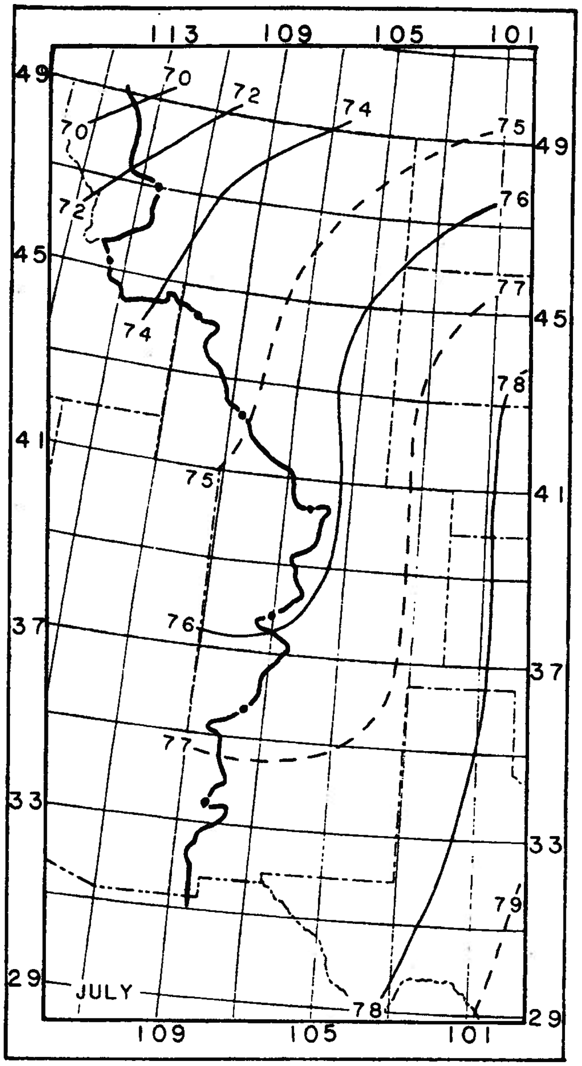


Figure 4.11.--Maximum persisting 12-hr 1000-mb dew points (°F) for July.

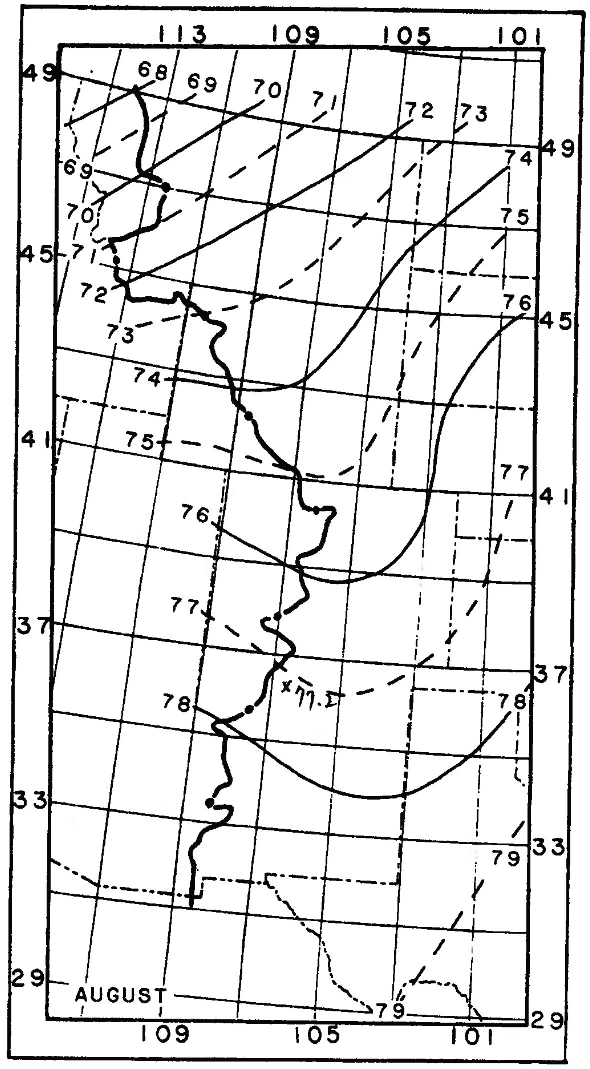


Figure 4.12.--Maximum persisting 12-hr 1000-mb dew points (°F) for August.

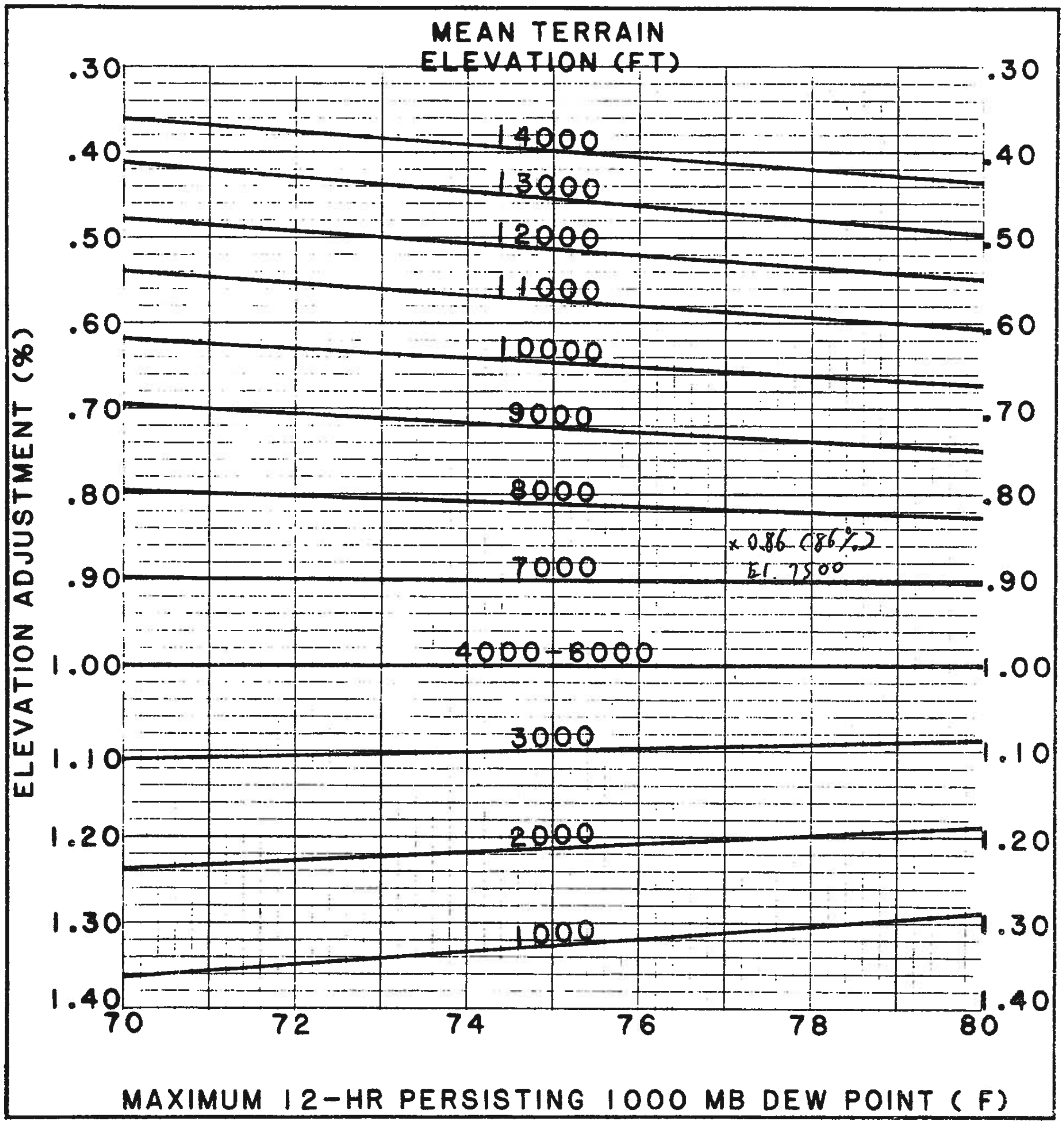
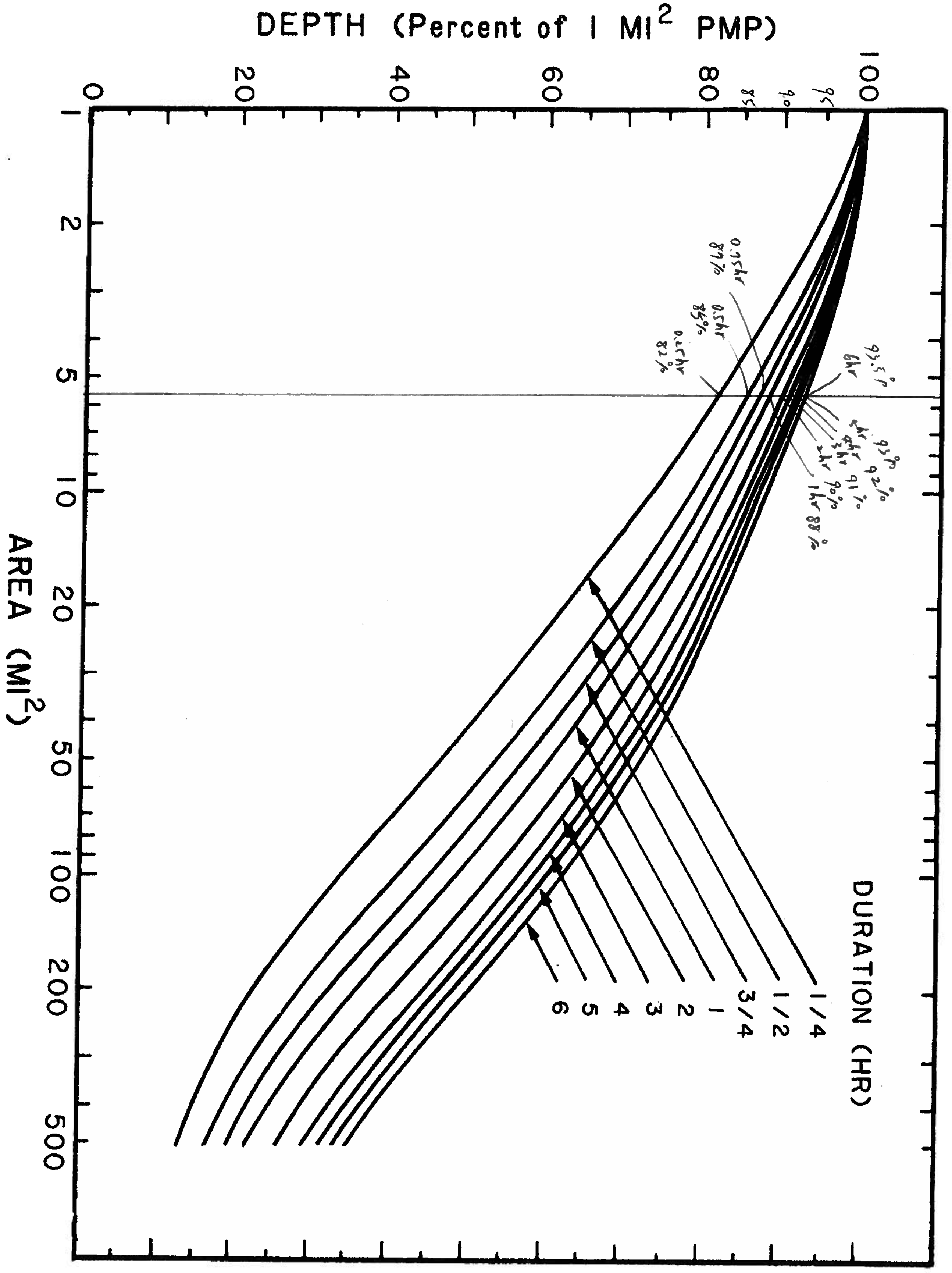


Figure 14.3.--Adjustment for elevation for local-storm PMP based on procedures developed in the report and maximum persisting 12-hr 1000-mb dew point (F).


DEPTH (Percent of 1 MI² PMP)
0
20
40
60
80
100
AREA (MI²)
1
2
5
10
20
50
100
200
500
DURATION (HR)
1/4
1/2
3/4
1
2
3
4
5
6

Appendix E

Laguna Del Campo Reservoir Elevation-Storage-Discharge Relationship Calculations

URS **CALCULATION COVER SHEET** Quality

Project Name:	Laguna Del Campo Dam	**Project Number:**	22242013
Project Location:	Rio Arriba, NM	**Client Name:**	US Fish and Wildlife Service
PM Name:	Birgit Dixon	**PIC Name:**	

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages *(including cover sheet)*: *5*

Discipline: *Dam EAP*

Title of Calculation: *Elevation-Area-Storage Relationship of Laguna Dam*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Brad Rastall*

DESCRIPTION & PURPOSE

To identify the elevation-area-storage relationship for Laguna Dam

BASIS / REFERENCE / ASSUMPTIONS

Burns Canon Dam Map (1938)

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	[ ]	[ ]	[ ]	[ ]
1		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

[signature] 1/19/2011
Originator Signature **Date**

Bradley W Rastall 1/19/11
Checker Signature **Date**

[signature] 1/20/11
Project Manager Signature **Date**

Distribution:

Project Central File – Quality file folder

Other Specify: _____

Purpose

1. To develop the stage-area-storage relationship from the given as-built construction plans for Laguna Del Campo Dam (Laguna Dam).

Given

- Map of Burns Canon Dam (Laguna Del Campo Dam) prepared by New Mexico Works Progress Administration (NMWPA), Engineering Department, New Mexico in 1938. (Attached)

  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\5.0_Reference\from NM OSE\Laguna Del Campo Dam\Dwgs - Laguna.pdf

- Construction Plan of Repairs to Brood Pond No. 3 (Laguna Del Campo Dam) Spillway prepared by Chambers, Campbell, Isaacson, Chaplin, Inc. in 1979.

  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\5.0_Reference\from USFWS\Laguna Del Campo Dam\Laguna Del Campo - 1979 Spillway Repair.pdf

Files

- Elevation-Area-Storage Table

  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Res_Stage-Storage \ Stage_Storage_Laguna.xls

- Calculation Cover Sheet and Summary

  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Res_Stage-Storage \

  CCS_Elev-Area-Storage_Laguna.doc

  Elev-Area-Storage_Laguna.doc

Summary

- Existing Pond Stage-Storage Curve
The given Burns Canon Dam Map (1938, NMWPA) shows a Reservoir Capacity Table and dam geometry information. These reservoir information and dam elevations were used to generate the stage-area-storage relationship for Laguna Dam. The reservoir calculation sheet and relevant information were attached to this analysis.

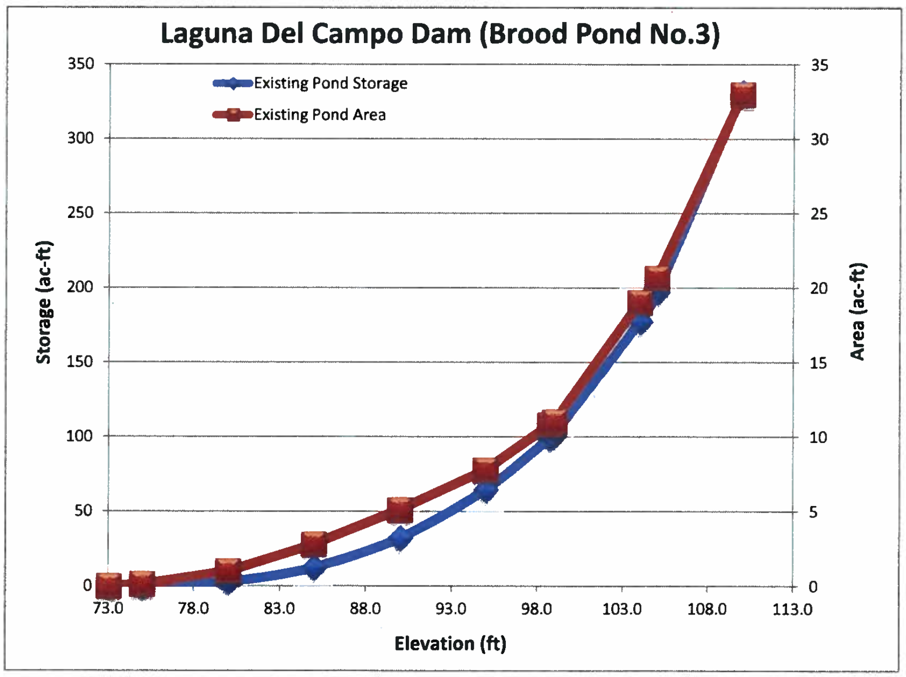
Detention/Pond Name: Laguna Del Campo Dam (Brood Pond No.3)

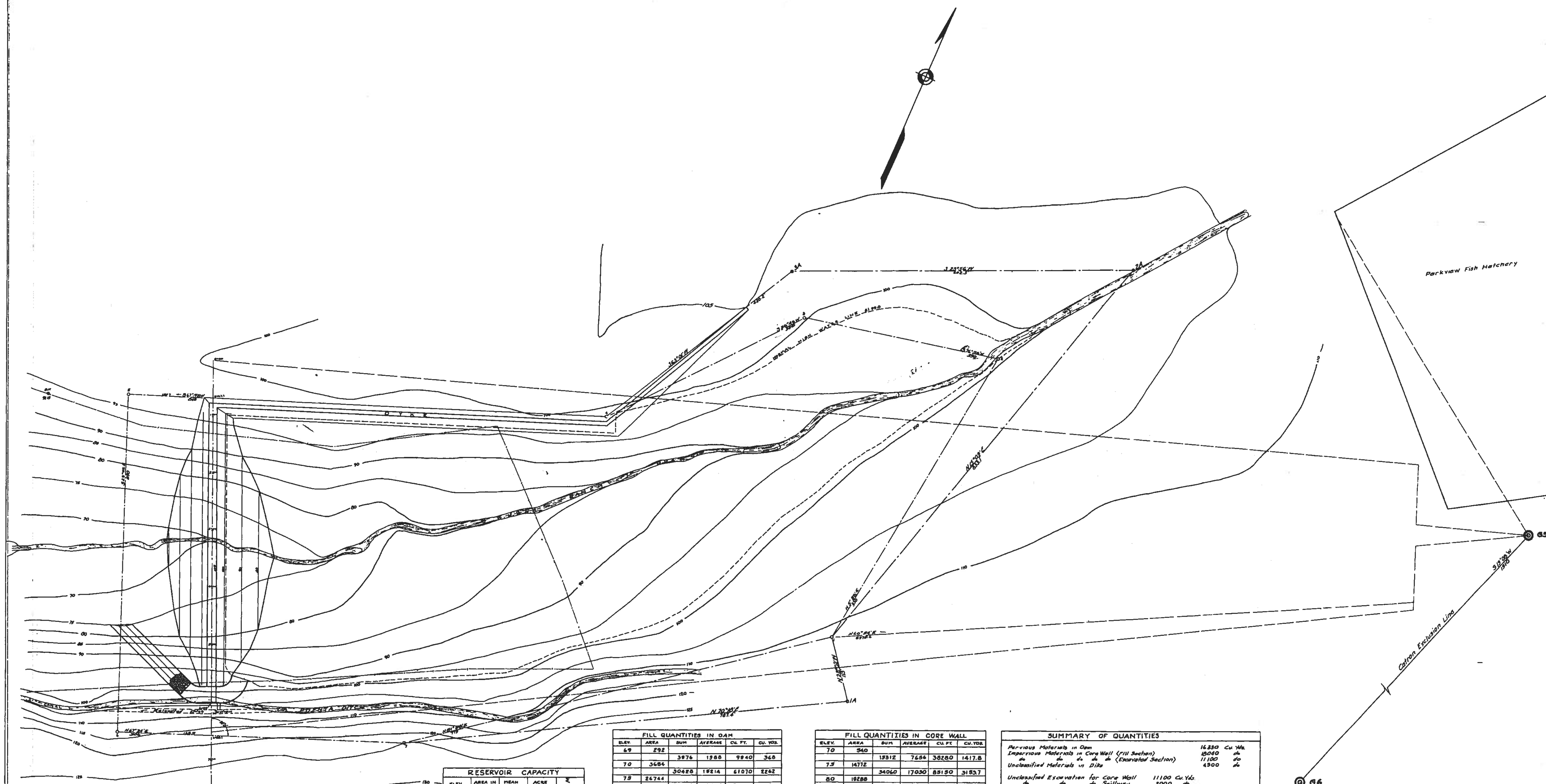
Vertical Datum: N/A

Elevation Convertor to NAVD 88= N/A ft

POND VOLUME CALCULATIONS

ID	Elevation (ft)	Area (ft^2)	Depth (ft)	Acres	Existing Pond Volume Incremental Vol. (ft^3)	Accumulated Vol. (ft^3)	Accumulated Vol. (Ac-Ft)	
1	73.0	0	0.0	0.0000	-		0.00	
2	75.0	5,650	2.0	0.1297	5,650	5,650	0.13	
3	80.0	43,525	7.0	0.9992	122,937	128,587	2.95	
4	85.0	122,325	12.0	2.8082	414,626	543,213	12.47	
5	90.0	223,676	17.0	5.1349	865,004	1,408,216	32.33	
6	95.0	339,424	22.0	7.7921	1,407,750	2,815,967	64.65	
7	98.75	471,635	25.8	10.8273	1,520,736	4,336,703	99.56	Spillway Crest
8	99.0	480,449	26.0	11.0296	119,011	4,455,713	102.29	
9	104.0	829,866	31.0	19.0511	3,275,788	7,731,501	177.49	Dam Crest
10	105.0	899,749	32.0	20.6554	864,808	8,596,309	197.34	
11	110.0	1,437,480	37.0	33.0000	5,843,073	14,439,382	331.48	





RESERVOIR CAPACITY				
ELEV.	AREA IN ACRES	MEAN AREA	ACRE FEET	Σ AC. FT.
73	0			
		0.0649	0.19	
75	0.1297			0.19
		0.5645	2.82	
80	0.9992			3.01
		1.9037	9.52	
85	2.8082			12.53
		3.9715	19.86	
90	5.1349			32.39
		6.4635	32.32	
95	7.7921			64.71
		9.4109	37.64	
99	11.0296			102.35
		15.8425	95.06	
105	20.6554			

FILL QUANTITIES IN DAM					
ELEV.	AREA	SUM	AVERAGE	CU. FT.	CU. YDS.
69	292				
		3976	1988	9940	368
70	3684				
		30428	15214	61070	2262
75	26744				
		62416	31208	156040	5779
80	35672				
		83356	41678	208390	7718
85	47684				
		80860	40430	202150	7487
90	33176				
		61560	30780	153900	5700
95	28384				
		48396	24198	120990	4481
100	20012				
		27812	13906	69530	2575
105	7800				
	203448	398804	199402	982010	36370
	QUANTITIES ARE ABOVE NATURAL GROUND.				

FILL QUANTITIES IN CORE WALL					
ELEV.	AREA	SUM	AVERAGE	CU. FT.	CU. YDS.
70	540				
		15312	7656	38280	1417.8
75	14772				
		34060	17030	85150	3153.7
80	19288				
		38084	19542	97710	3618.9
85	18796				
		37350	18675	93375	3458.3
90	17560				
		31564	15782	78910	2922.6
95	14004				
		24148	12074	60370	2235.9
100	10144				
		13312	6656	33280	1232.6
105	8168				
				487075	18039.8
	QUANTITIES ARE ABOVE NATURAL GROUND				

SUMMARY OF QUANTITIES

Pervious Materials in Dam	16330	Cu. Yds.
Impervious Materials in Core Wall (Fill Section)	18040	do
do do do do do (Excavated Section)	11100	do
Unclassified Materials in Dike	4500	do

Unclassified Excavation for Core Wall 11100 Cu. Yds.
do do do Spillway 2000 do

Spillway Capacity 1360 cu. ft. per sec.

NEW MEXICO WORKS PROGRESS ADMINISTRATION ENGINEERING DEPARTMENT SANTA FE NEW MEXICO	DRAWN BY
	CHECKED BY
BURNS CANON DAM PARKVIEW RIO ARRIBA CO FRED G HEALY SCALE 1"=50' ADMINISTRATOR	DATE Apr 23 1938
	PROJECT NO 465-85-2-64
	SHEET NO 1

F.P.-No.PV-8

APPLICATION No. 2181
RECEIVED
JULY 27 1937
OFFICE OF STATE ENGINEER
SANTA FE, NEW MEXICO

CLAIMANT'S CERTIFICATE

STATE OF NEW MEXICO } ss:
COUNTY OF SANTA FE }

I, Elliott S. Barker, being first duly sworn upon my oath state that I am the Secretary of the State Game Commission, duly organized under the laws of the State of New Mexico, that the foregoing map and statements were made under authority of the State Game Commission, and that I, in their behalf, have read and examined the foregoing statements and representations thereon and state that the same are true to the best of my knowledge and belief.

STATE GAME COMMISSION
Claimant.

By Elliott S Barker
Secretary.

Subscribed and sworn to before me this 24th day of July, 1937

Minnie B. Carter
Notary Public

My comission expires June 3, 1939

ENGINEER'S CERTIFICATE

STATE OF NEW MEXICO } ss:
COUNTY OF RIO ARRIBA }

I, Kenneth A. Heron, being first duly sworn upon my oath, state that I am the engineer who made the map of the BROOD POND No. 3, PARKVIEW FISH HATCHERY, and that such map was made from field notes taken from actual bona fide surveys made in the field by me, and from plans furnished by Geo. M. Neel, and that the same true and correct to the best of my knowledge and belief.

Kenneth A. Heron
Engineer.

Subscribed and sworn to before me this 20th day of July, 1937.

A. J. Barr
Notary Public.

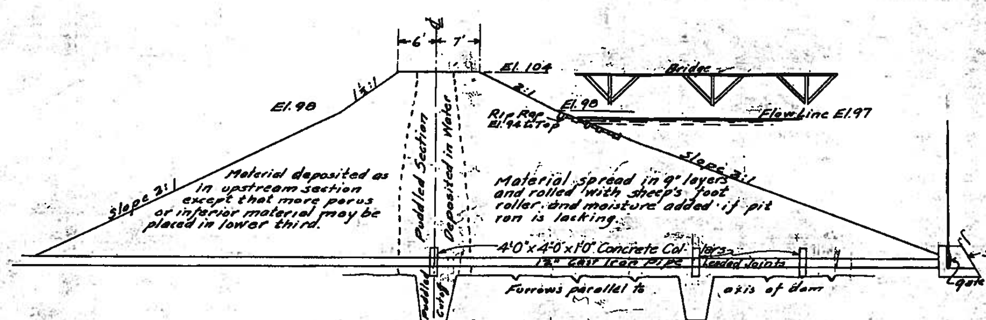
My commission expires 2-1-40

STATE ENGINEER'S CERTIFICATE

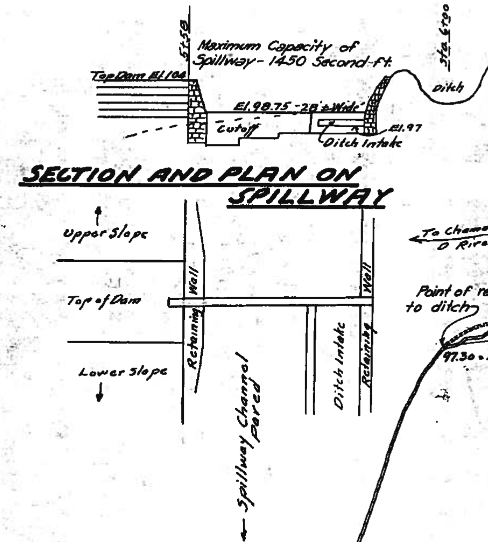
STATE OF NEW MEXICO } ss:
COUNTY OF SANTA FE }

I hereby certify that the above map and statements have been examined by me, and were duly accepted for filing on the 27 day of July 1937.

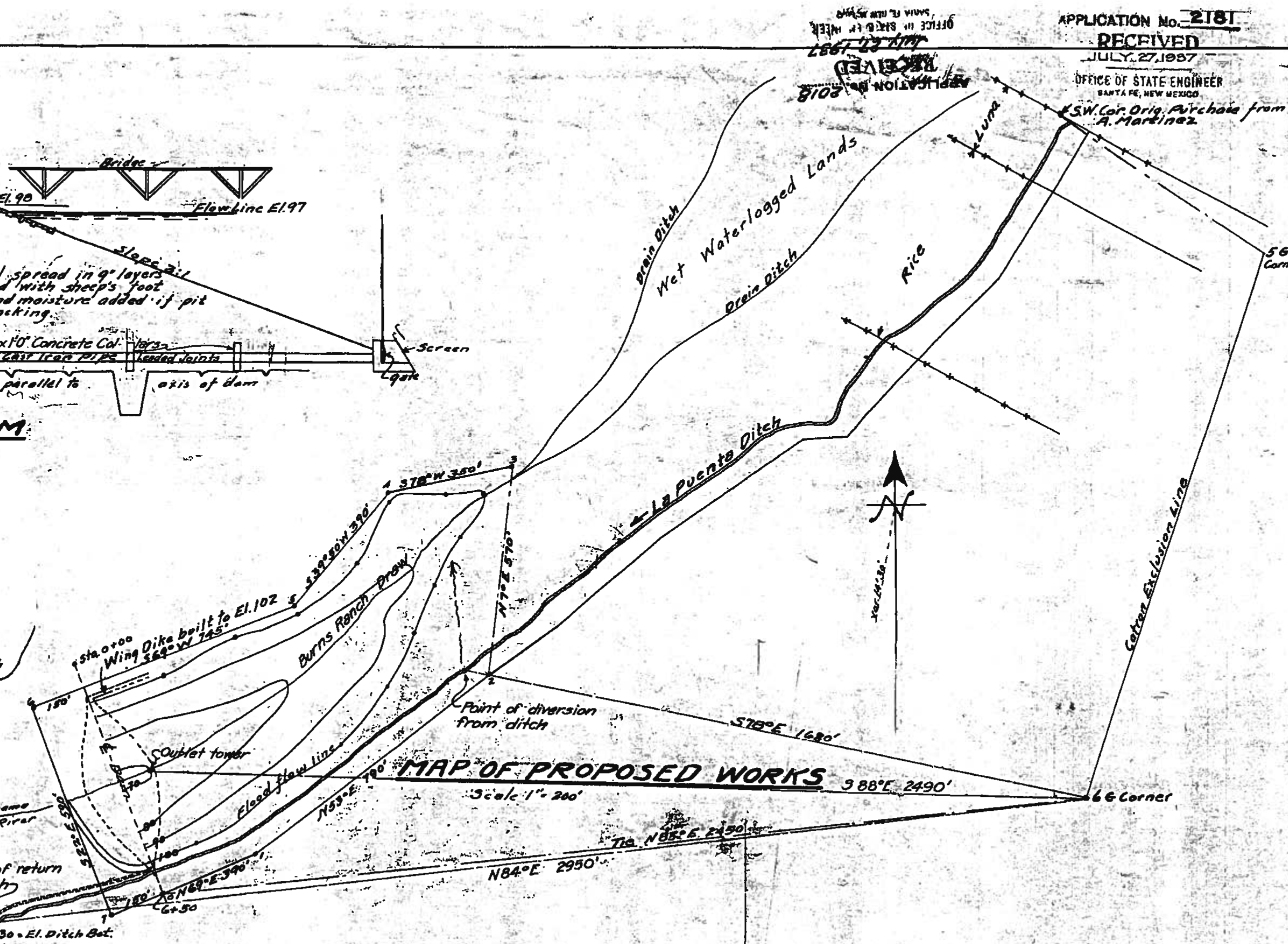
Thos. M. McClure
State Engineer.



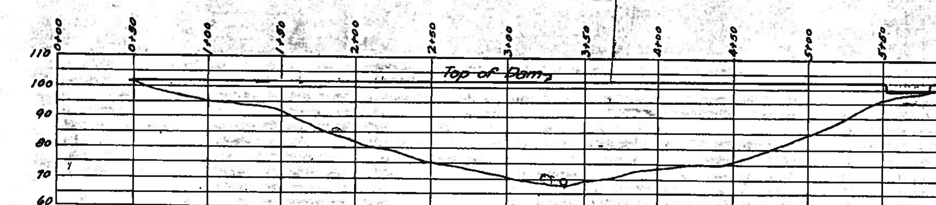
SECTION ON DAM



SECTION AND PLAN ON SPILLWAY



MAP OF PROPOSED WORKS
Scale 1" = 200'



PROFILE ON DAMSITE
Looking Upstream

The claimant, the State Game Commission, has caused to be located by a competent surveyor and engineer, the works shown hereon and makes these several statements relative thereto, and offers this map and statements for acceptance and doc filing in compliance with the laws of the State of New Mexico:

1. The "6G" corner on the boundary of the Parkview Mesa Tract, Catron Exclusions, Tierra Amarilla Grant, bears N88°E 2480' from Sta. 6+50 on the ℄ of the proposed dam.

2. All of the lands shown hereon are on the Tierra Amarilla grant, Rio Arriba County, N.M.

3. Maximum height of dam is 36.0';
Length of dam is 500;
Top of dam is 13.0 wide; bottom 190' wide;
Dam to be an earth fill;
Slopes to be as shown hereon;

4. Maximum storage capacity of 96.7 acre feet is hereby claimed continously, when there is an unappropriated supply of water to fill the reservoir.

5. The cost of the works is estimated at $ 30,000.00;

6. Drain ditches 4' wide and 2' deep are proposed as shown hereon.

7. The pond or reservoir is to be used for fish breeding purposes.

8. The reservoir tract is to be acquired from the present owner, T. D. Burns.

9. Surplus water to the amount of 6.0 c.f.s. originating on the property of the claimant one half 1 mile Northeast of the proposed pond is wasted into La Puenta Ditch. It is proposed to divert this into the pond and divert all surplus water back into the La Puenta Ditch.

10. Soil on Midsection of dam site is surface gravel underlaid at 2 ft. depth with tight impervious clay. Walls of canyon mostly clay with some gravel imbedded in clay matrix.

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Reservoir Contents

Elev.	Area Acres	M.A.	Contents A.F.	Total Contents A.F.
70	0.2			
		1.1	11.0	
80	2.0			11.0
		3.8	38.0	
90	5.6			49.0
		6.8	47.7	
97	8.0			96.7
		8.8	26.3	
100	9.5			123.0

Map of
BROOD POND No. 3 PARKVIEW FISH HATCHERY
-of-
STATE GAME COMMISSION, APPLICANT
Located in Rio Arriba County, N. M.

Scale as Shown
July 20, 1937

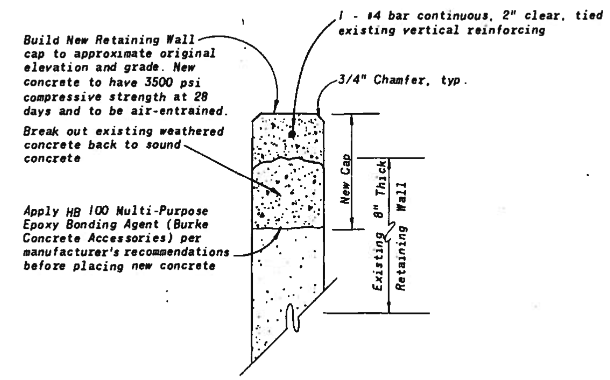
Kenneth A. Heron
Engineer.

REPAIRS TO BROOD POND NO. 3 SPILLWAY

PARKVIEW FISH HATCHERY — RIO ARRIBA COUNTY, N.M.

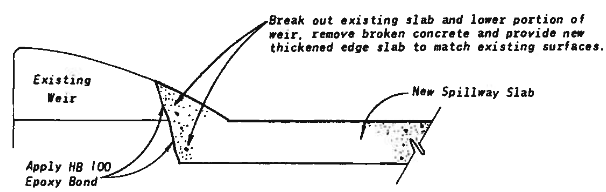
SCHEDULE OF QUANTITIES

Spillway Slab Concrete	12 cubic yards
Epoxy	60 gallons
Welded Wire Mesh	440 square feet
Expansion Joint Dowels	11 each



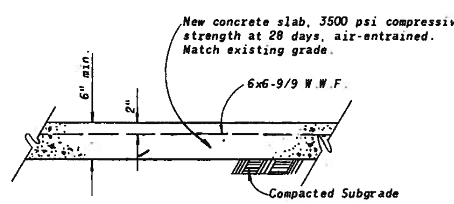
NEW RETAINING WALL CAP

1 1/2" = 1'-0"



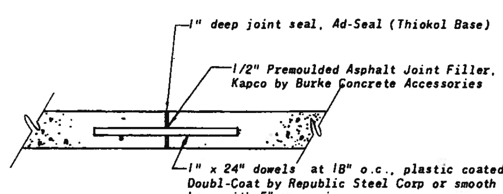
NEW SPILLWAY SLAB AT WEIR

No Scale



NEW SPILLWAY SLAB

1" = 1'-0"

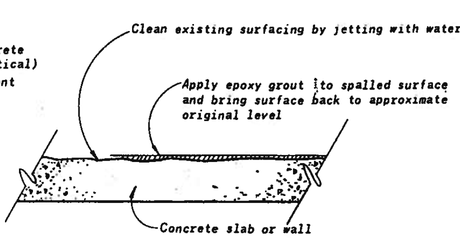


TYPE 'A' EXPANSION JOINT DETAIL

1" = 1'-0"

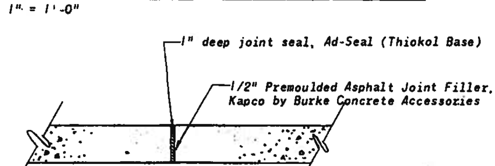
NOTES:

1. All existing distressed concrete surfaces (horizontal and vertical) shall receive surface treatment application.
2. Epoxy grout shall consist of Hunt Process, HB 100 Multi-Purpose Epoxy Bonding Agent mixed with sand with a minimum ratio of adhesive (epoxy) to aggregate of 1:4 by volume.



SURFACE TREATMENT DETAIL

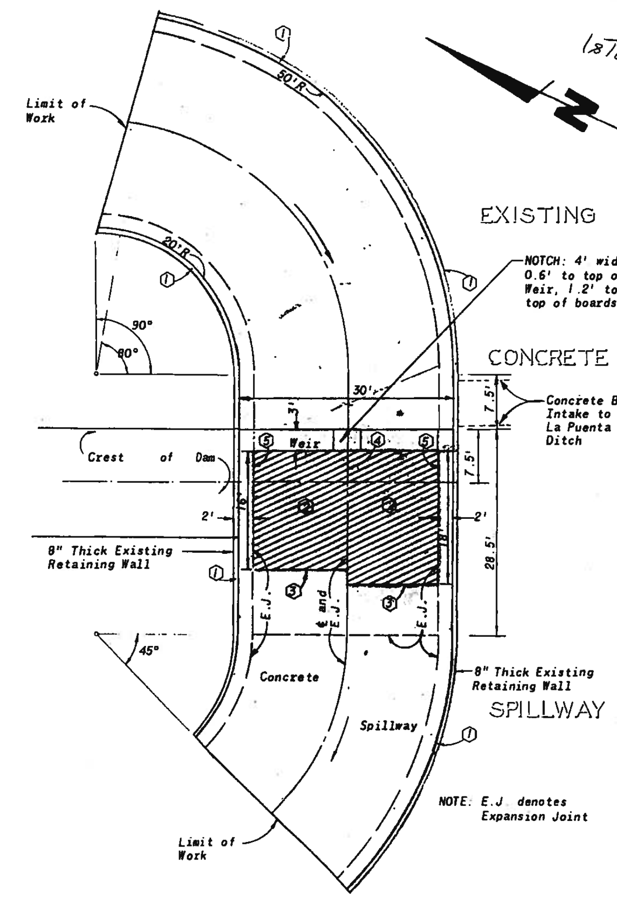
No Scale



TYPE 'B' EXPANSION JOINT DETAIL

1" = 1'-0"

NOTE: Fish Screen not shown.



PLAN OF SPILLWAY

1" = 10'

1. Build new retaining wall cap - see detail this sheet.
2. Cross-hatched area indicates concrete slab to be removed and re-constructed per details this sheet.
3. Existing mortar filled construction joint to be westerly limits of new concrete slab.
4. Build Type 'A' Expansion Joint along spillway ℄.
5. Build Type 'B' Expansion Joint along retaining wall footing.

ENGINEER'S CERTIFICATE

STATE OF NEW MEXICO
COUNTY OF BERNALILLO

I, Thomas O. Isaacson, being first duly sworn upon my oath, state that I am a registered professional engineer, qualified in civil engineering and that the accompanying plans and specifications consisting of one sheet of plans was prepared under my supervision and direction.

License No. 3895

Thomas O. Isaacson
Registered Professional Engineer

Subscribed and sworn to before me this 9th day of July, 1979.
My commission expires: 11-22-81

Notary Public

REGISTERED PROFESSIONAL ENGINEER AND LAND SURVEYOR — NO. 3895 — NEW MEXICO — THOMAS O. ISAACSON

Chambers, Campbell, Isaacson, Chaplin, Inc.

JOB NO 379-4
DATE JUNE 1979
SHEET 1 OF 1

APPLICATION NO. 2187
RECEIVED
OFFICE OF STATE ENGINEER
SANTA FE, NEW MEXICO

URS CALCULATION COVER SHEET *Quality*

Project Name:	*Laguna Del Campo Dam*	**Project Number:**	*22242013*
Project Location:	*Rio Arriba, NM*	**Client Name:**	*US Fish and Wildlife Service*
PM Name:	*Birgit Dixon*	**PIC Name:**	

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages *(including cover sheet)*: *5*

Discipline: *Dam EAP*

Title of Calculation: *Dam Stage-Discharge Rating Curve of Laguna Dam*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Brad Rastall*

DESCRIPTION & PURPOSE

To estimate the dam stage-discharge relationship for Laguna Dam

BASIS / REFERENCE / ASSUMPTIONS

•Flow Master

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	[ ]	[ ]	[ ]	[ ]
1	RATING CURVE OVERFLOW AT DAM	☐	☐	☐	[illegible]	[illegible]	[illegible]	[illegible]
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

[signature] 02.08.2011

Originator Signature **Date**

[signature] 2/8/11

Checker Signature **Date**

[signature] 2/9/11

Project Manager Signature **Date**

Distribution:

Project Central File – Quality file folder

Other Specify: _____

OBJECTIVES:

To estimate dam flow rating table for Laguna Del Campo Dam (Laguna Dam) based on the available information.

GIVEN:

- Construction Plan of Burns Canyon Dam by New Mexico State Game Commission in 1937 (Attached).

REFERENCES:

- *Bentley FlowMaster 2008, Bentley System Inc.*
- *United States Department of Interior, Bureau of Reclamation (USBR), 1987. Design of Small Dams. Water Resources Technical Publication. Third Edition.*

FILES:

- FlowMaster Model
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Dam Rating Curve\FMS\Weir.fm8
- Summary Table & Figure
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Dam Rating Curve\Weir Flow.xlsx
- Calculation Cover Sheet and Memo
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Dam Rating Curve\
  CCS_Dam_RatingCurves_Laguna.doc
  Summary of Dam_RatingCurves_Laguna.doc

SUMMARY:

The Laguna Del Campo Dam flow rating curve is a stage-discharge relationship of a combined weir flow. Flow can run through the lowflow opening at the weir structure in the emergency spillway, overtop the ogee spillway crest, dam crest, as well as the west dike. The study weir geometries were taken from the as-built construction plans (attached). The lengths of dam crest and dike are 500 ft and 1030 ft in order. A discharge coefficient of 3.5 was used for calculating discharge over the ogee weir. This discharge coefficient is in the range of values suggested by Reclamation (USBR, 1987) as shown on **Figure 1**. Flow overtopping the dam and north dike crest was considered broad crested weir flow and calculated using the Bentley Flow Master computer program. In the Bentley Flow Master computer program, the discharge coefficient of 3.09 is estimated for a broad-crest weir based on the input parameters. Flow rating table for each weir was estimated using FlowMaster program. The calculated rating tables were attached to this analysis. The combined dam flow rating table and chart were attached to this calculation.

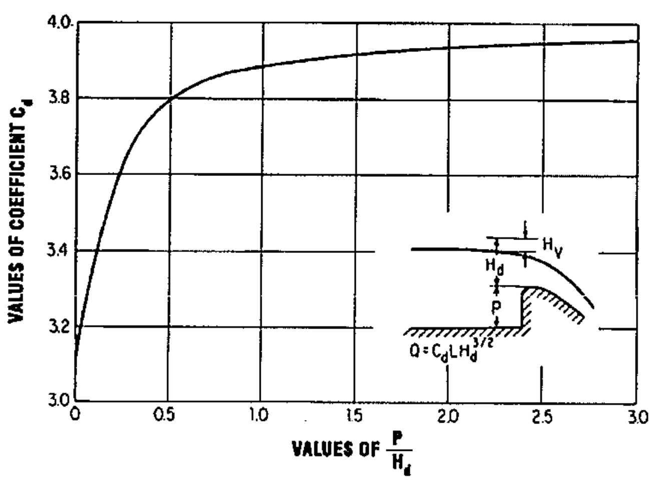
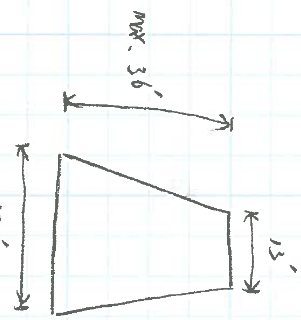


Figure 1
Discharge Coefficients for Vertical-Faced Ogee-Crest (from USBR, 1987)

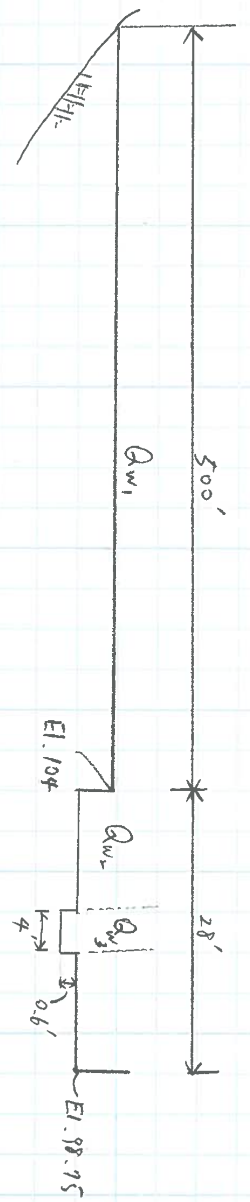
URS

Job: Laguna Del Campo Dam EAP	Project No. 22242013	Page ___ of ___	
		Sheet ___ of ___	
Description: Dam Weir Flow	Computed by: max shih	Date: 02.02.2011	
	Checked by:	Date:	

Reference



Reference: Construction Plan (1937)
Burns Canyon Dam.



Dam flow = $Q_{w_1} + Q_{w_2} + Q_{w_3}$

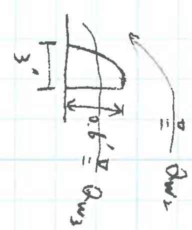
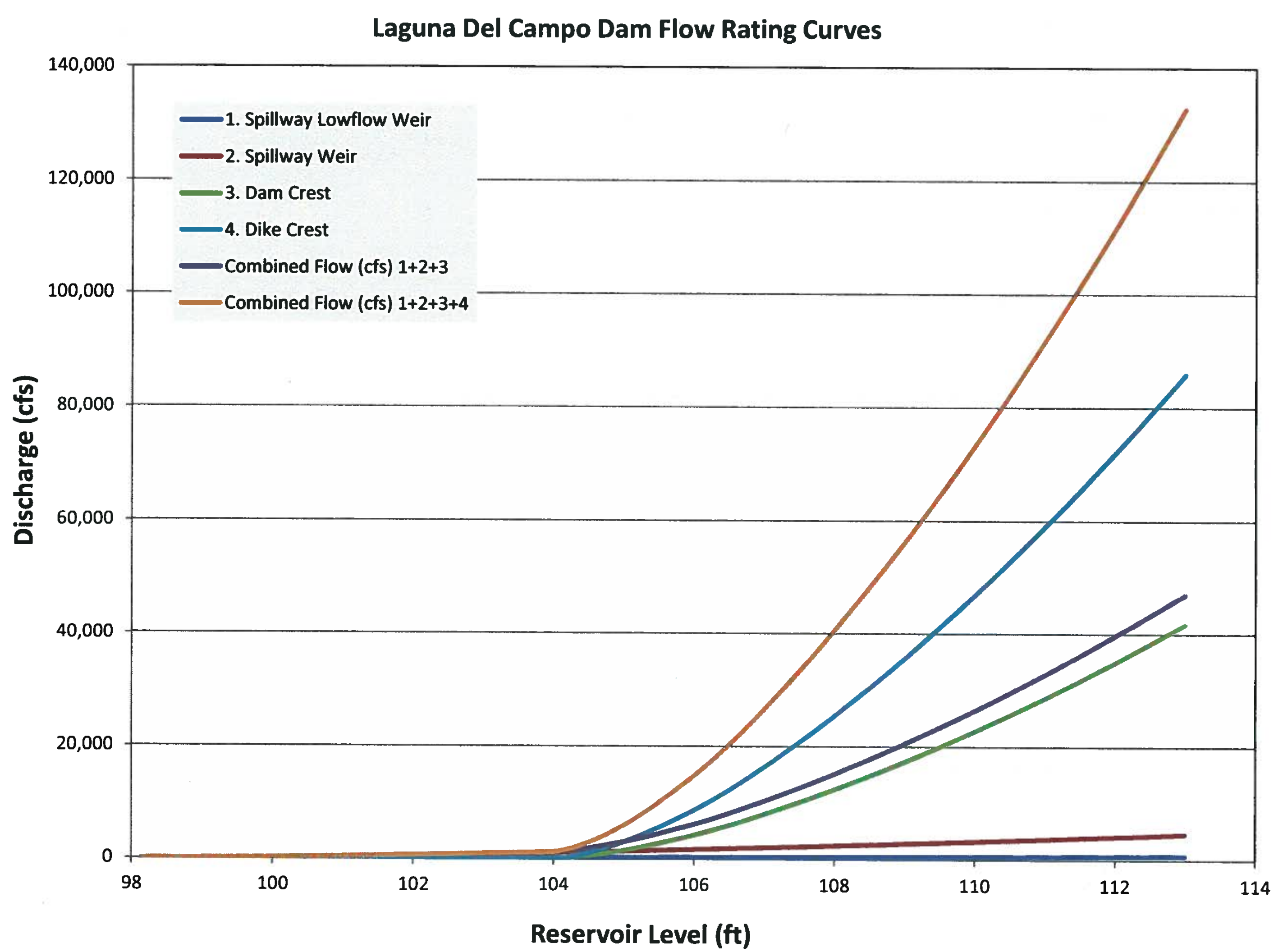


Table A. Laguna Del Campo Dam Flow Rating Table

Reservoir Level (ft)	Weir Flow (cfs)				Combined Flow (cfs) 1+2+3+4	Combined Flow (cfs) 1+2+3
	1. Spillway Lowflow Weir	2. Spillway Weir	3. Dam Crest	4. Dike Crest		
98.15	0	0	0	0	0	0
98.25	0	0	0	0	0	0
98.5	2	0	0	0	2	2
98.75	6	0	0	0	6	6
99	10	11	0	0	20	20
99.25	14	30	0	0	44	44
99.5	19	55	0	0	74	74
99.75	25	84	0	0	109	109
100	31	117	0	0	148	148
100.25	38	154	0	0	192	192
100.5	44	194	0	0	239	239
100.75	52	238	0	0	289	289
101	59	284	0	0	343	343
101.25	67	332	0	0	399	399
101.5	76	383	0	0	459	459
101.75	84	436	0	0	521	521
102	93	492	0	0	585	585
102.25	103	550	0	0	653	653
102.5	112	610	0	0	722	722
102.75	122	672	0	0	794	794
103	132	736	0	0	868	868
103.25	142	802	0	0	944	944
103.5	153	870	0	0	1022	1022
103.75	164	939	0	0	1103	1103
104	175	1010	0	0	1185	1185
104.25	186	1083	163	335	1768	1432
104.5	198	1158	476	981	2813	1832
104.75	209	1235	899	1851	4193	2342
105	221	1313	1414	2913	5861	2948
105.25	234	1392	2011	4143	7780	3637
105.5	246	1473	2681	5523	9924	4400
105.75	259	1556	3416	7038	12268	5231
106	272	1640	4168	8586	14665	6079
106.25	285	1725	4998	10297	17305	7008
106.5	298	1812	5906	12167	20184	8017
106.75	311	1901	6879	14171	23262	9091
107	325	1990	7901	16275	26492	10216
107.25	339	2082	8957	18452	29830	11378
107.5	353	2174	10041	20684	33252	12568
107.75	367	2268	11154	22978	36768	13790

108	382	2363	12318	25376	40439	15063
108.25	396	2460	13524	27858	44238	16379
108.5	411	2557	14734	30352	48055	17703
108.75	426	2656	15979	32917	51978	19061
109	441	2757	17257	35549	56004	20455
109.25	457	2858	18567	38248	60130	21882
109.5	472	2961	19909	41013	64355	23342
109.75	488	3065	21282	43841	68675	24834
110	504	3170	22685	46731	73089	26358
110.25	520	3276	24117	49681	77594	27913
110.5	536	3383	25579	52692	82190	29498
110.75	552	3492	27068	55761	86873	31112
111	569	3602	28586	58887	91644	32756
111.25	585	3712	30131	62070	96499	34429
111.5	602	3824	31703	65308	101437	36129
111.75	619	3937	33301	68600	106458	37858
112	636	4051	34925	71946	111560	39613
112.25	654	4167	36575	75345	116741	41396
112.5	671	4283	38250	78796	122000	43204
112.75	689	4400	39950	82297	127337	45039
113	707	4519	41675	85849	132749	46900


Laguna Del Campo Dam Flow Rating Curves
1. Spillway Lowflow Weir
2. Spillway Weir
3. Dam Crest
4. Dike Crest
Combined Flow (cfs) 1+2+3
Combined Flow (cfs) 1+2+3+4
Discharge (cfs)
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
98
100
102
104
106
108
110
112
114
Reservoir Level (ft)

Rating Table for Dam Crest Weir

Project Description

Solve For	Discharge

Input Data

Headwater Elevation		110.00	ft
Crest Elevation		104.00	ft
Tailwater Elevation		90.00	ft
Crest Surface Type	Gravel		
Crest Breadth		13.00	ft
Crest Length		500.00	ft

Headwater Elevation (ft)	Discharge (ft^3/s)	Velocity (ft/s)
98.00		
98.25		
98.50		
98.75		
99.00		
99.25		
99.50		
99.75		
100.00		
100.25		
100.50		
100.75		
101.00		
101.25		
101.50		
101.75		
102.00		
102.25		
102.50		
102.75		
103.00		
103.25		
103.50		
103.75		

Rating Table for Dam Crest Weir

Input Data

Headwater Elevation (ft)	Discharge (ft³/s)	Velocity (ft/s)
104.00		
104.25	162.84	1.30
104.50	476.08	1.90
104.75	898.55	2.40
105.00	1414.00	2.83
105.25	2011.22	3.22
105.50	2681.21	3.57
105.75	3416.34	3.90
106.00	4168.00	4.17
106.25	4998.43	4.44
106.50	5906.39	4.73
106.75	6878.97	5.00
107.00	7900.67	5.27
107.25	8957.37	5.51
107.50	10040.90	5.74
107.75	11154.40	5.95
108.00	12318.26	6.16
108.25	13523.53	6.36
108.50	14734.16	6.55
108.75	15978.91	6.73
109.00	17256.85	6.90
109.25	18567.16	7.07
109.50	19909.06	7.24
109.75	21281.80	7.40
110.00	22684.72	7.56
110.25	24117.19	7.72
110.50	25578.59	7.87
110.75	27068.38	8.02
111.00	28586.02	8.17
111.25	30131.01	8.31
111.50	31702.87	8.45
111.75	33301.15	8.59
112.00	34925.42	8.73
112.25	36575.27	8.87
112.50	38250.32	9.00
112.75	39950.18	9.13

Rating Table for Dam Crest Weir

Input Data

Headwater Elevation (ft)	Discharge (ft^3/s)	Velocity (ft/s)
113.00	41674.50	9.26

Rating Table for Spillway Ogee Weir

Project Description

Solve For	Discharge

Input Data

Headwater Elevation	110.00	ft
Crest Elevation	98.75	ft
Weir Coefficient	3.50	US
Crest Length	24.00	ft

Headwater Elevation (ft)	Discharge (ft³/s)	Velocity (ft/s)
98.00		
98.25		
98.50		
98.75		
99.00	10.50	1.75
99.25	29.70	2.47
99.50	54.56	3.03
99.75	84.00	3.50
100.00	117.39	3.91
100.25	154.32	4.29
100.50	194.46	4.63
100.75	237.59	4.95
101.00	283.50	5.25
101.25	332.04	5.53
101.50	383.07	5.80
101.75	436.48	6.06
102.00	492.16	6.31
102.25	550.02	6.55
102.50	609.99	6.78
102.75	672.00	7.00
103.00	735.97	7.22
103.25	801.86	7.42
103.50	869.60	7.63
103.75	939.15	7.83
104.00	1010.46	8.02
104.25	1083.49	8.21

Rating Table for Spillway Ogee Weir

Input Data

Headwater Elevation (ft)	Discharge (ft^3/s)	Velocity (ft/s)
104.50	1158.19	8.39
104.75	1234.54	8.57
105.00	1312.50	8.75
105.25	1392.03	8.92
105.50	1473.11	9.09
105.75	1555.70	9.26
106.00	1639.78	9.42
106.25	1725.33	9.59
106.50	1812.31	9.74
106.75	1900.70	9.90
107.00	1990.49	10.05
107.25	2081.65	10.20
107.50	2174.16	10.35
107.75	2268.00	10.50
108.00	2363.15	10.64
108.25	2459.60	10.79
108.50	2557.33	10.93
108.75	2656.31	11.07
109.00	2756.54	11.21
109.25	2858.01	11.34
109.50	2960.68	11.48
109.75	3064.56	11.61
110.00	3169.63	11.74
110.25	3275.87	11.87
110.50	3383.27	12.00
110.75	3491.81	12.12
111.00	3601.50	12.25
111.25	3712.31	12.37
111.50	3824.23	12.50
111.75	3937.26	12.62
112.00	4051.38	12.74
112.25	4166.58	12.86
112.50	4282.85	12.98
112.75	4400.19	13.10
113.00	4518.58	13.21

Rating Table for Spillway Lowflow Weir

Project Description

Solve For	Discharge

Input Data

Headwater Elevation	110.00	ft
Crest Elevation	98.15	ft
Tailwater Elevation	90.00	ft
Crest Surface Type	Paved	
Crest Breadth	3.00	ft
Crest Length	4.00	ft

Headwater Elevation (ft)	Discharge (ft³/s)	Velocity (ft/s)
98.00		
98.25	0.37	0.93
98.50	2.47	1.77
98.75	5.70	2.37
99.00	9.67	2.84
99.25	14.25	3.24
99.50	19.37	3.59
99.75	24.99	3.90
100.00	31.07	4.20
100.25	37.58	4.47
100.50	44.48	4.73
100.75	51.77	4.98
101.00	59.41	5.21
101.25	67.40	5.44
101.50	75.71	5.65
101.75	84.34	5.86
102.00	93.28	6.06
102.25	102.51	6.25
102.50	112.03	6.44
102.75	121.82	6.62
103.00	131.89	6.80
103.25	142.22	6.97
103.50	152.80	7.14
103.75	163.64	7.31

Rating Table for Spillway Lowflow Weir

Input Data

Headwater Elevation (ft)	Discharge (ft³/s)	Velocity (ft/s)
104.00	174.72	7.47
104.25	186.03	7.62
104.50	197.59	7.78
104.75	209.37	7.93
105.00	221.38	8.08
105.25	233.61	8.23
105.50	246.05	8.37
105.75	258.71	8.51
106.00	271.58	8.65
106.25	284.66	8.79
106.50	297.94	8.92
106.75	311.42	9.05
107.00	325.10	9.18
107.25	338.97	9.31
107.50	353.03	9.44
107.75	367.29	9.56
108.00	381.73	9.69
108.25	396.35	9.81
108.50	411.16	9.93
108.75	426.14	10.05
109.00	441.31	10.17
109.25	456.65	10.28
109.50	472.16	10.40
109.75	487.85	10.51
110.00	503.70	10.63
110.25	519.73	10.74
110.50	535.92	10.85
110.75	552.27	10.96
111.00	568.79	11.07
111.25	585.47	11.17
111.50	602.31	11.28
111.75	619.31	11.38
112.00	636.46	11.49
112.25	653.77	11.59
112.50	671.23	11.69
112.75	688.85	11.80

Rating Table for Spillway Lowflow Weir

Input Data

Headwater Elevation (ft)	Discharge (ft^3/s)	Velocity (ft/s)
113.00	706.62	11.90

Rating Table for Dike Crest Weir

Project Description

Solve For	Discharge

Input Data

Headwater Elevation		110.00	ft
Crest Elevation		104.00	ft
Tailwater Elevation		90.00	ft
Crest Surface Type	Gravel		
Crest Breadth		13.00	ft
Crest Length		1030.00	ft

Headwater Elevation (ft)	Discharge (ft^3/s)	Velocity (ft/s)
104.00		
104.25	335.44	1.30
104.50	980.73	1.90
104.75	1851.01	2.40
105.00	2912.83	2.83
105.25	4143.12	3.22
105.50	5523.29	3.57
105.75	7037.65	3.90
106.00	8586.08	4.17
106.25	10296.76	4.44
106.50	12167.17	4.73
106.75	14170.67	5.00
107.00	16275.38	5.27
107.25	18452.18	5.51
107.50	20684.25	5.74
107.75	22978.07	5.95
108.00	25375.62	6.16
108.25	27858.47	6.36
108.50	30352.37	6.55
108.75	32916.55	6.73
109.00	35549.12	6.90
109.25	38248.36	7.07
109.50	41012.66	7.24
109.75	43840.51	7.40

Rating Table for Dike Crest Weir

Input Data

Headwater Elevation (ft)	Discharge (ft^3/s)	Velocity (ft/s)
110.00	46730.53	7.56
110.25	49681.41	7.72
110.50	52691.90	7.87
110.75	55760.87	8.02
111.00	58887.20	8.17
111.25	62069.87	8.31
111.50	65307.90	8.45
111.75	68600.36	8.59
112.00	71946.36	8.73
112.25	75345.06	8.87
112.50	78795.65	9.00
112.75	82297.37	9.13
113.00	85849.47	9.26

ꟼILLWAY

\ COUNTY, N.M.

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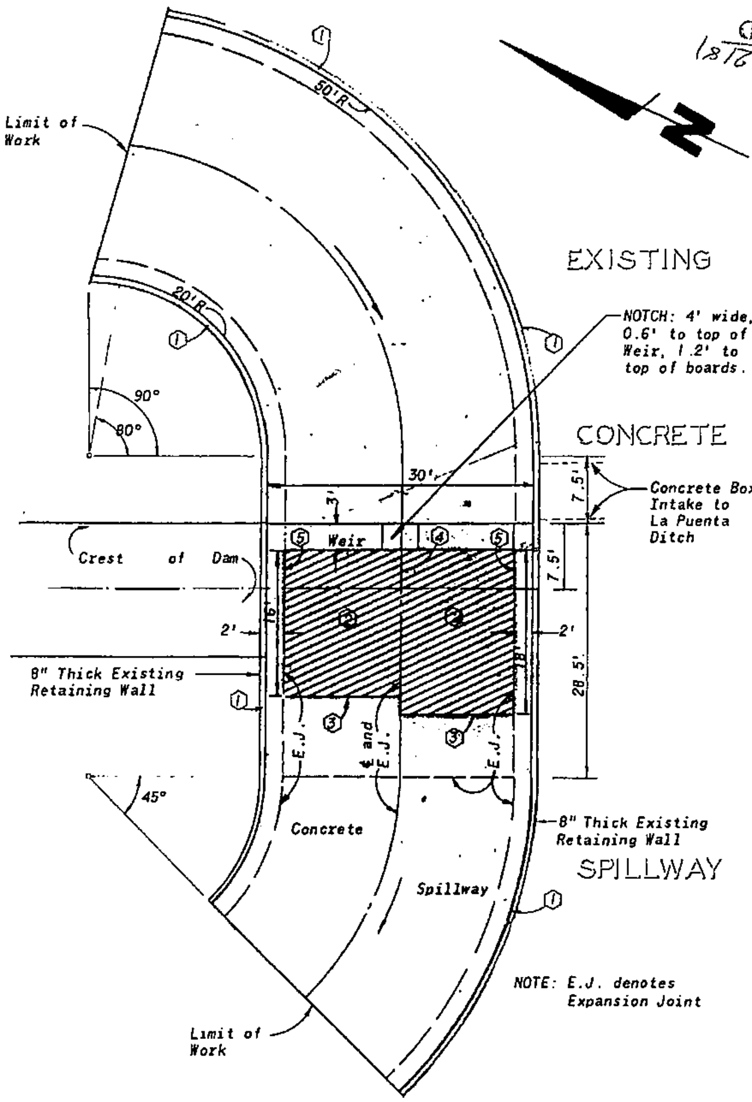
NOTE: Fish Screen not shown.

OFFICE OF STATE ENGINEER
SANTA FE, NEW MEXICO
RECEIVED
APPLICATION NO. 2181



PLAN OF SPILLWAY

1" = 10'

1. Build new retaining wall cap - see detail this sheet.
2. Cross-hatched area indicates concrete slab to be removed and re-constructed per details this sheet.
3. Existing mortar filled construction joint to be westerly limits of new concrete slab.
4. Build Type 'A' Expansion Joint along spillway ℄.
5. Build Type 'B' Expansion Joint along retaining wall footing.

:d
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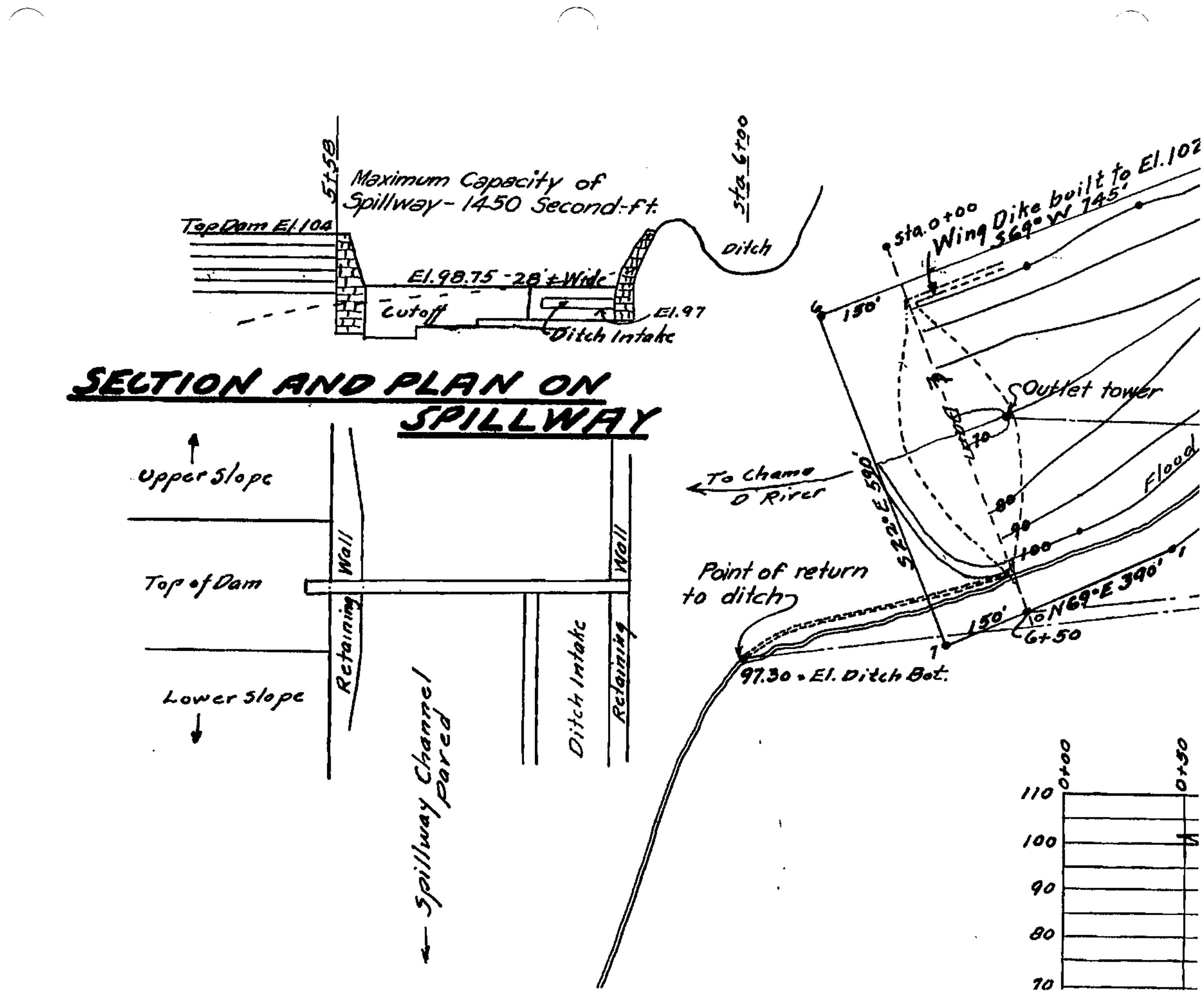
ENGINEER'S CERTIFICATE

STATE OF NEW MEXICO
COUNTY OF BERNALILLO

REGISTERED PROFESSIONAL ENGINEER AND LAND SURVEYOR NO. 3895 NEW MEXICO


SECTION AND PLAN ON SPILLWAY
Maximum Capacity of Spillway - 1450 Second-ft.
Top Dam El. 104
5+58
sta 6+00
Ditch
El. 98.75 - 28' ± Wide
Cutoff
El. 97
Ditch Intake
Upper Slope
Top of Dam
Lower Slope
Retaining Wall
Ditch Intake
Retaining Wall
Spillway Channel paved
Sta. 0+00
Wing Dike built to El. 102
S69° W 145'
150'
Outlet tower
Dam
70
80
90
100
Flood
To Chame O River
S22° E 590'
Point of return to ditch
150'
N69° E 390'
6+50
97.30 = El. Ditch Bot.
110
100
90
80
70
0+00
0+50

and that such map was made from field notes taken from actual bona fide surveys made in the field by me, and from plans furnished by Geo. M. Neel, and that the same true and correct to the best of my knowledge and belief.

Kenneth A. Heron
Engineer.

Subscribed and sworn to before me this 20th day of [illegible] 1937

A.J. Barr
Notary Public.

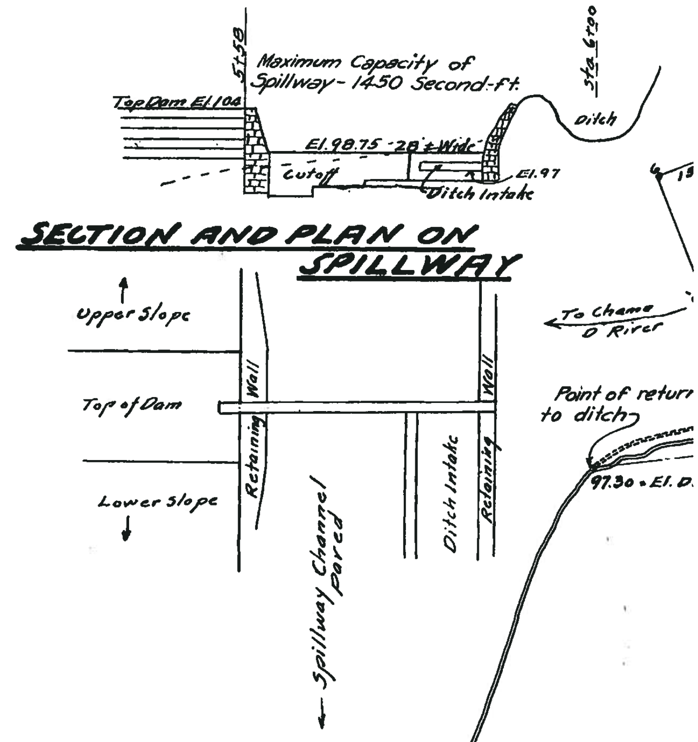
My commission expires 2-1-40

STATE ENGINEER'S CERTIFICATE

STATE OF NEW MEXICO }
COUNTY OF SANTA FE } ss.

I hereby certify that the above map and statements have been examined by me, and were duly accepted for filing on the 27 day of July 1937.

Thomas M. McClure
State Engineer.



The claimant, the State Game Commission, has caused to be located by a competent surveyor and engineer, the works shown hereon and makes these several statements relative thereto, and offers this map and statements for acceptance and due filing in compliance with the laws of the State of New Mexico:

1. The "6G" corner on the boundary of the Parkview Mesa Tract, Catron Exclusions, Tierra Amarilla Grant, bears N88° E 2450' from Sta. 6+50 on the ℄ of the proposed dam.

2. All of the lands shown hereon are on the Tierra Amarilla grant, Rio Arriba County, N.M.

3. Maximum height of dam is 36.0';
Length of dam is 500';
Top of dam is 13.0' wide; bottom 190' wide;
Dam to be an earth fill;
Slopes to be as shown hereon;

4. Maximum storage capacity of 96.7 acre feet is hereby claimed continously, when there is an unappropriated supply of water to fill the reservoir.

5. The cost of the works is estimated at $30,000.00;

6. Drain ditches 4' wide and 2' deep are proposed as shown hereon.

7. The pond or reservoir is to be used for fish breeding purposes.

8. The reservoir tract is to be acquired from the present owner, T. D. Burns.

9. Surplus water to the amount of 6.0 c.f.s. originating on the property of the claimant one half mile Northeast of the proposed pond is wasted into La Puenta Ditch. It is proposed to divert this into the pond and divert all surplus water back into the La Puenta Ditch.

10 Soil on Midsection of dam site is surface gravel underlaid at 2 ft depth with tight impervious clay. Walls of canyon mostly clay with some gravel imbedded in clay matrix.

[illegible] No 2181

JULY 27, 1937

URS **CALCULATION COVER SHEET** Quality

Project Name:	Laguna Del Campo Dam	**Project Number:**	22242013
Project Location:	Rio Arriba, NM	**Client Name:**	US Fish and Wildlife Service
PM Name:	Birgit Dixon	**PIC Name:**	

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic — File Name:

(Select as appropriate) ☒ Hard-copy — Unique Identification:

Number of pages *(including cover sheet)*: *5*

Discipline: *Dam EAP*

Title of Calculation: *Outlet Works Stage-Discharge Rating Curve of Laguna Dam*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Brad Rastall*

DESCRIPTION & PURPOSE

To identify the outlet works stage-discharge relationship for Laguna Dam

BASIS / REFERENCE / ASSUMPTIONS

- *Design of Small Dams, US Bureau of Reclamation 1987*

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	[ ]	[ ]	[ ]	[ ]
1		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

Originator Signature — **Date** 03/01/11

Brad Rastall

Checker Signature — **Date** 3/2/11

Project Manager Signature — **Date** 3/3/11

Distribution:

Project Central File – Quality file folder

Other Specify: ____

OBJECTIVES:

To determine the stage-discharge rating curve for Laguna Del Campo Dam (Laguna Dam) outlet works based on the available information.

GIVEN:

- Construction Plan of Burns Canyon Dam by New Mexico State Game Commission in 1937 (Attached).
- Dam Safety Inspection Report for Brood Pond Dam No.3 (Laguna Del Campo Dam) on December 26, 1978.

REFERENCES:

- *Design of Small Dams*, US Bureau of Reclamation 1987. (Chapter 10 Outlet Works attached in Appendix B)

FILES:

- Calculation Sheets of Elevation-Discharge rating curve
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\OutletWorks\Outlet-Rating_Laguna.xls
- Calculation Cover Sheet and Memo
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\OutletWorks\
  CCS_Outlet Works_Laguna.doc
  Summary of Outlet Works_Laguna.doc

SUMMARY:

Available information and Assumptions:
The given one sheet construction plan is the only information providing the elevations of dam outlet works and attached to this calculation. According to the 1978 dam inspection report (attached), the outlet works has been reconstructed as a 2ft by 2ft cast concrete conduit with a control at the upstream toe. Since the intake gate is normally kept closed, an energy loss coefficient of 1.0 was assumed to reflect the energy loss through the intake structure and entrance to the main culvert. The elevations were assumed as the same as the original construction plan. The outlet works conduit was presumed flat horizontally in the calculation. Free flow condition is utilized at the downstream outlet.

Approach
A stage-discharge rating curve was estimated based on construction plans of the Laguna Dam outlet works. These calculation sheets and relevant documents are attached to this calculation. The calculations of energy losses were estimated based on Chapter 10 Outlet Works in Design of Small Dams (USBR, 1987). The referred materials and equations are described in the following sections.

1. Entrance Loss, H_e

$$H_e = K_e \frac{V^2}{2g}$$

where V is flow velocity through the entrance; g is gravity acceleration; and K_e is entrance loss coefficient and is presumed 1.0 in the analysis.

2. Friction Loss, H_f

Friction losses by conduits were estimated using Eq. 10 on Page 456 of Dam of Small Dams (DSD).

$$H_f = K_f \frac{V^2}{2g}$$

$$K_f = 29.1 \cdot n^2 \left(\frac{L}{r^{4/3}} \right)$$

where n is Manning's roughness; L is conduit length; r is hydraulic radius, 2'*2'/(2'*4)=0.5 ; V is flow velocity; and g is gravity acceleration.

3. Outflow Velocity Head, H_o

The dam outlet was assumed as a free flowing exit. The velocity head of free flowing at the conduit exit was expressed as

$$H_o = K_o \frac{V_o^2}{2g}$$

where K_o is exist loss coefficient and is unity herein; and V_o is the outflow velocity.

4. Required Upstream Water Head, H_T, based on outlet control mechanism.

The required upstream water head within the Bear Canyon Reservoir is to sum of all energy losses and terminal flow velocity head of outflow. It can be shown as

$$H_T = \sum H_{loss} + H_o$$

where H_{loss} are the energy loss sources from contraction, structure entrance and expansion, conduit friction, gates, outlet, etc.

The stage-discharge relationship of dam outlet works was shown in the attached calculation sheet.

Laguna Del Campo Dam - Outlet Works Rating Curve
URS Project No. 22242013

By:	Max Shih
Date:	03.02.2011
Checked:	Brad Rastall

Scenario:

Intake Tower Bottom Gate Elevation (ft):	73
Main Culvert Size	2'(R)X2'(S) Cast Concrete Box Culvert
Outlet Central Line Elevation (ft):	73.5
Normal Pool Elevation (ft):	98.5

Pipe and Valve Sizes:

	Diameter (in) /Dimension	Area (ft^2)	Mannings n	Length Pipe (ft)	Loss Coeff
Entrance	2'(R)X2'(S)	4.0			0.80
Main Culvert	2'(R)X2'(S)	4.0	0.012	146.5	1.55
Outlet	2'(R)X2'(S)	4.0			1.00

Loss Coefficient and Headloss Estimate:

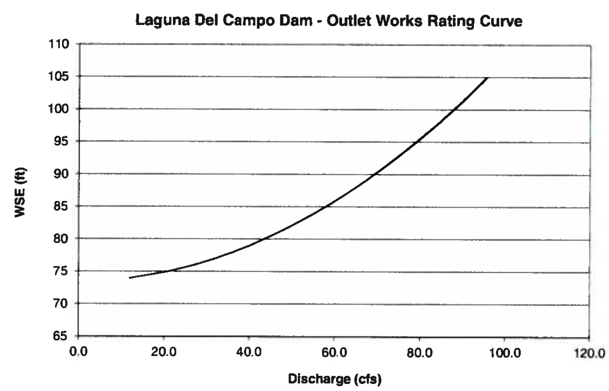
Item	Element	Area (ft^2)	Area Ratio Squared	Loss type	Loss Coeff	Loss Coeff times area ratio	Peak Velocity (ft/s)	Head Loss (ft)
1	Entrance	4.0	1.00	Gate	1.00	1.00	21.3	7.05
2	Culvert	4.0	1.00	Friction	1.55	1.55	21.3	10.90
3	Exit	4.0	1.00	Exit	1.00	1.00	21.3	7.05
	Totals					3.55		25.00

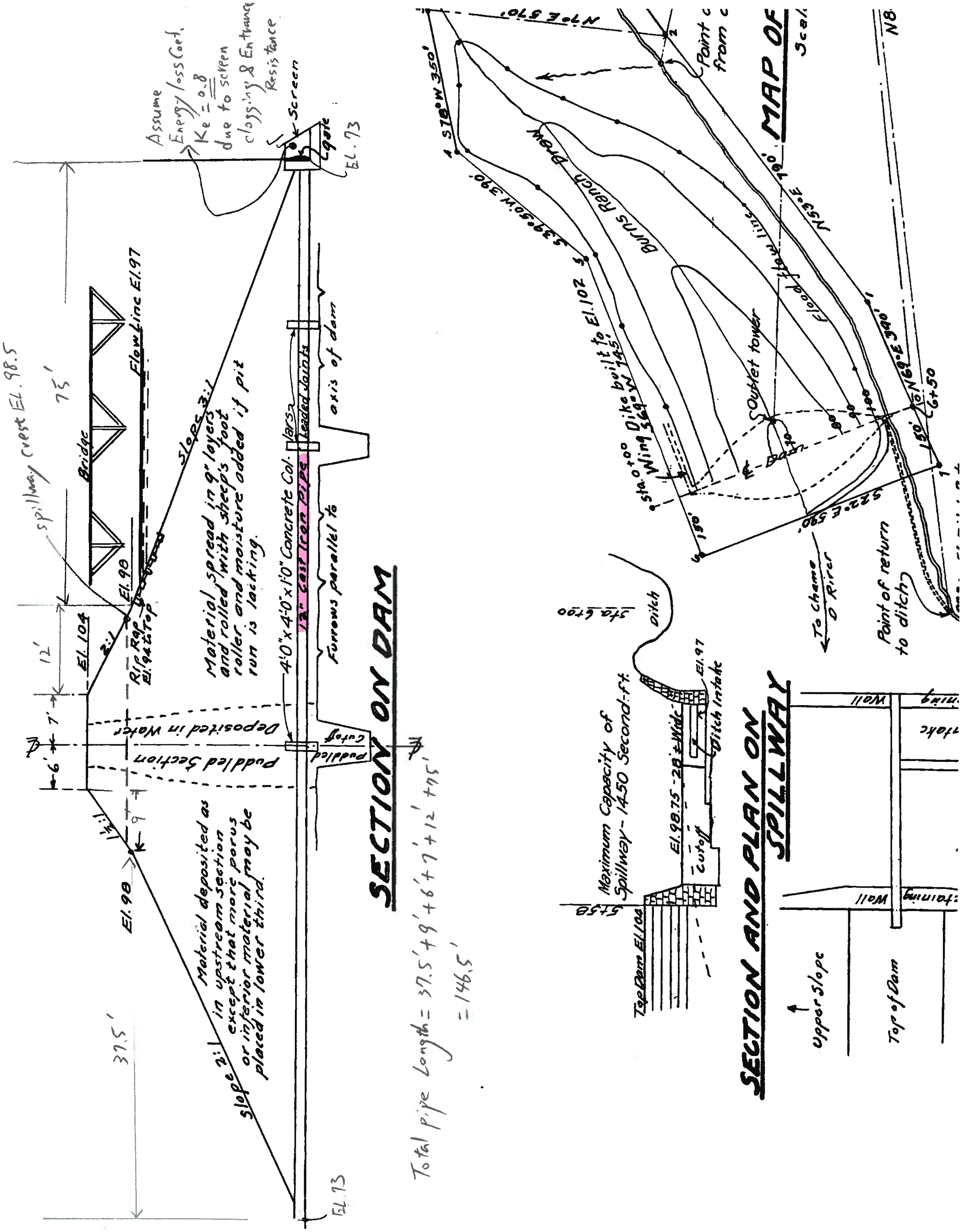
Estimate Peak Flow:

Total Head (ft):	25
Peak Flow (cfs)	85.2

Rating Curve:

WSE (ft)	Water Head (ft)	Discharge (cfs)	
74	0.5	12.1	
75	1.5	20.9	
76	2.5	26.9	
77	3.5	31.9	
78	4.5	36.2	
79	5.5	40.0	
80	6.5	43.5	
81	7.5	46.7	
82	8.5	49.7	
83	9.5	52.5	
84	10.5	55.2	
85	11.5	57.8	
86	12.5	60.3	
87	13.5	62.6	
88	14.5	64.9	
89	15.5	67.1	
90	16.5	69.2	
91	17.5	71.3	
92	18.5	73.3	
93	19.5	75.3	
94	20.5	77.2	
95	21.5	79.0	
96	22.5	80.8	
97	23.5	82.6	
98	24.5	84.4	
98.75	25.25	85.6	Spillway Crest
99	25.5	86.1	
100	26.5	87.7	
101	27.5	89.4	
102	28.5	91.0	
103	29.5	92.6	
104	30.5	94.1	Dam Crest
105	31.5	95.7	
106	32.5	97.2	
107	33.5	98.7	
108	34.5	100.1	
109	35.5	101.6	
110	36.5	103.0	




SECTION ON DAM
Total pipe Length = 37.5' + 9' + 6' + 7' + 12' + 75' = 146.5'
spillway crest El. 98.5
Assume Energy loss Coef. Ke = 0.8 due to screen clogging & Entrance Resistance
Screen
Gate
El. 73
Bridge
Flow Line El. 97
Slope 3:1
El. 98
Rip Rap
El. 104
2:1
Material spread in 9" layers and rolled with sheep's foot roller and moisture added if pit run is lacking.
4'0" x 4'0" x 1'0" Concrete Col-lars
12" Cast Iron Pipe
Leaded Joints
axis of dam
Furrows parallel to
Deposited in Water
Puddled Section
Puddled Cutoff
Material deposited as in upstream section except that more porus or inferior material may be placed in lower third.
Slope 2:1
El. 98
37.5'
75'
12'
7'
6'
9'
SECTION AND PLAN ON SPILLWAY
Maximum Capacity of Spillway - 1450 Second-Ft.
Top Dam El. 104
El. 98.75 - 28' Wide
Ditch Intake
El. 97
Cutoff
ditch
sta. 6+00
5+58
Upper Slope
Top of Dam
Wall
MAP OF
Burns Ranch Draw
Outlet tower
Dam
Wing Dike built to El. 102
Flood flow line
Point of return to ditch
To Chama River
Point
N 1° E 570'
S 78° W 350'
S 39° 50' W 390'
N 53° E 190'
N 69° E 390'
S 22° E 590'
6+50

Bud

SWAED-TA

26 December 1978

Honorable Jerry Apodaca
Governor of New Mexico
Santa Fe, NM 87501

Dear Governor Apodaca:

In pursuance of the National Dam Safety Program, representatives from Tierra Engineering Consultants and personnel from the State Engineer Office inspected Brood Pond No. 3 Dam located approximately one mile west of Tierra Amarilla in San Juan County, New Mexico on 21 August 1978. Results of this inspection are contained in the Phase I Inspection Report (Inclosure 2).

My staff has reviewed the inspection report, which was prepared by Tierra Engineering Consultants. Results of this review in the form of comments and recommendations are attached as Inclosure 1.

While I intend to make no public release concerning the report, I note for your consideration that 30 days from the date of this letter, the report will be subject to release upon demand under the Freedom of Information Act. Thus, in the interim, you may wish to initiate a public statement concerning the report.

I would appreciate your keeping me informed of actions taken on our recommendations. Please contact me if you have any questions.

Sincerely yours,

2 Incl
As stated

LARRY A BLAIR
Lieutenant Colonel, CE
Acting District Engineer

Copies furnished: w/incl 1, wo/incl 2
Mr. S.E. Reynolds, State Engr.
Bataan Mem. Bldg., State Capitol
Santa Fe, NM 87503

✓Harold F. Olson, Director w/incl
New Mexico State Natural Res. Dept.
Game & Fish Division
Villagra Bldg.
Santa Fe, NM 87503

Comments and Recommendations
on Brood Pond Dam No. 3

1. Brood Pond Dam No. 3 is an earth-fill dam approximately 36 feet high and 500 feet long on the main embankment and an additional 1030 feet on the north leg of the embankment. The spillway, located on the south abutment, is concrete and includes an upstream approach channel, a downstream discharge channel, a weir with a low flow notch and a ditch intake (La Puente Ditch) incorporated into the south retaining wall. The ditch is operated by a slide gate. The outlet works is a two foot-by-two foot cast in place concrete conduit with a control at the upstream toe. The control consists of a slide gate manually operated from a wooden platform by means of a screw-type gate stem and hand wheel. Brood Pond Dam No. 3 is owned and operated by the New Mexico State Natural Resources Dept., Game and Fish Division and the purpose of the dam is to store and regulate water for irrigation and fish breeding.

2. Brood Pond Dam No. 3 is classified as a small-size structure and is considered to have significant hazard potential. According to screening criteria contained in "Guidelines for Safety Inspection of Dams," the dam and spillway should accommodate a 100-year flood to ½ probable maximum flood (PMF). Spillway capacity is approximately the 100-year flood or 12% of PMF.

3. The inspection report recommends that the vegetation be removed from the spillway channel, abutments and dam embankment. The spillway concrete should be patched and a better irrigation diversion scheme provided. An inspection and maintenance program should be initiated and an emergency operation plan developed for this dam. The outlet works mechanism should be greased, covered from the weather and operated periodically to assure that it is in good condition. Additional recommendations, with which we concur, are contained in the report on page 1 and section 7.

Incl 1

Appendix F

HEC-HMS Modeling

Model Inputs and Hydrologic Analysis

URS CALCULATION COVER SHEET *Quality*

Project Name:	*Laguna Del Campo Dam*	**Project Number:**	*22242013*
Project Location:	*Rio Arriba, NM*	**Client Name:**	*US Fish and Wildlife Service*
PM Name:	*Birgit Dixon*	**PIC Name:**	

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages
(including cover sheet): *5*

Discipline: *Dam EAP*

Title of Calculation: *Average Soil Infiltration Rate Estimation for Laguna Dam*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Brad Rastall*

DESCRIPTION & PURPOSE

To estimate the average soil infiltration rate for Laguna Del Campo Dam watershed

BASIS / REFERENCE / ASSUMPTIONS

•Cudworth, A.G., Flood Hydrology Manual. United States Department of Commerce Bureau of Reclamation. Denver, Colorado. 1989

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	[ ]	[ ]	[ ]	[ ]
1		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

[signature] 01.24.2011
Originator Signature **Date**

Bradley Rastall 1/24/11
Checker Signature **Date**

[signature] 1/24/11
Project Manager Signature **Date**

Distribution:

Project Central File – Quality file folder
Other Specify: _____

Purpose:
Estimate an average infiltration rate for the Laguna Del Campo Dam watershed.

References:

Cudworth, A.G., *Flood Hydrology Manual.* United States Department of Commerce Bureau of Reclamation. Denver, Colorado. 1989

Soil Database of Rio Arriba County, New Mexico, Geospatial Data Gateway, United States Department of Agriculture (USDA). http://datagateway.nrcs.usda.gov/

FILES:

- Calculation Sheets of Soil Infiltration
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Soil Infiltration\SoilInfiltration.xlsx

- Calculation Cover Sheet and Memo
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Soil Infiltration\
  CCS_CalcPacket_Soil_Infiltration_Laguna.doc
  CalcPacket_Soil_Infiltration_Laguna.doc

SUMMARY:

According to the Flood Hydrology Manual (1989), no initial infiltration loss shall be used for PMF estimation. The minimum / ultimate soil infiltration rate was considered as a constant rainfall loss rate for PMF modeling. Hydrological Soil Groups were used to identify the soil infiltration rate. County soil database taken from USDA Geospatial Data Gateway website provides detail soil group distribution. Soil Database of Rio Arriba County was used to delineate the areas of soil groups within the study watershed. The watershed soil map is shown in Figure 1. The recommended soil infiltration rate was referred to the Flood Hydrology Manual. Table 1 expresses the computed weighted soil infiltration rates and an average infiltration rate for the Laguna Del Campo watershed.

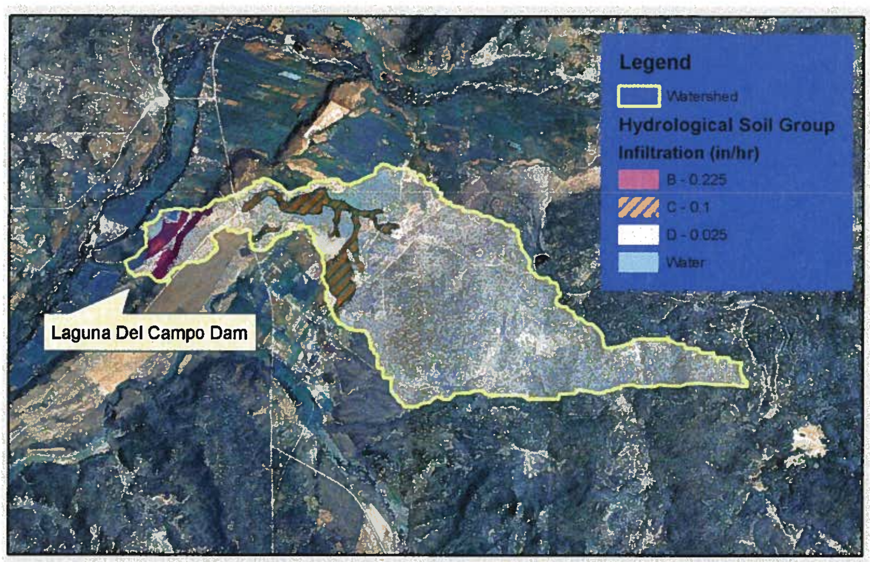


Figure 1. Hydrological Soil Group Distribution within the Laguna Del Campo Dam Watershed

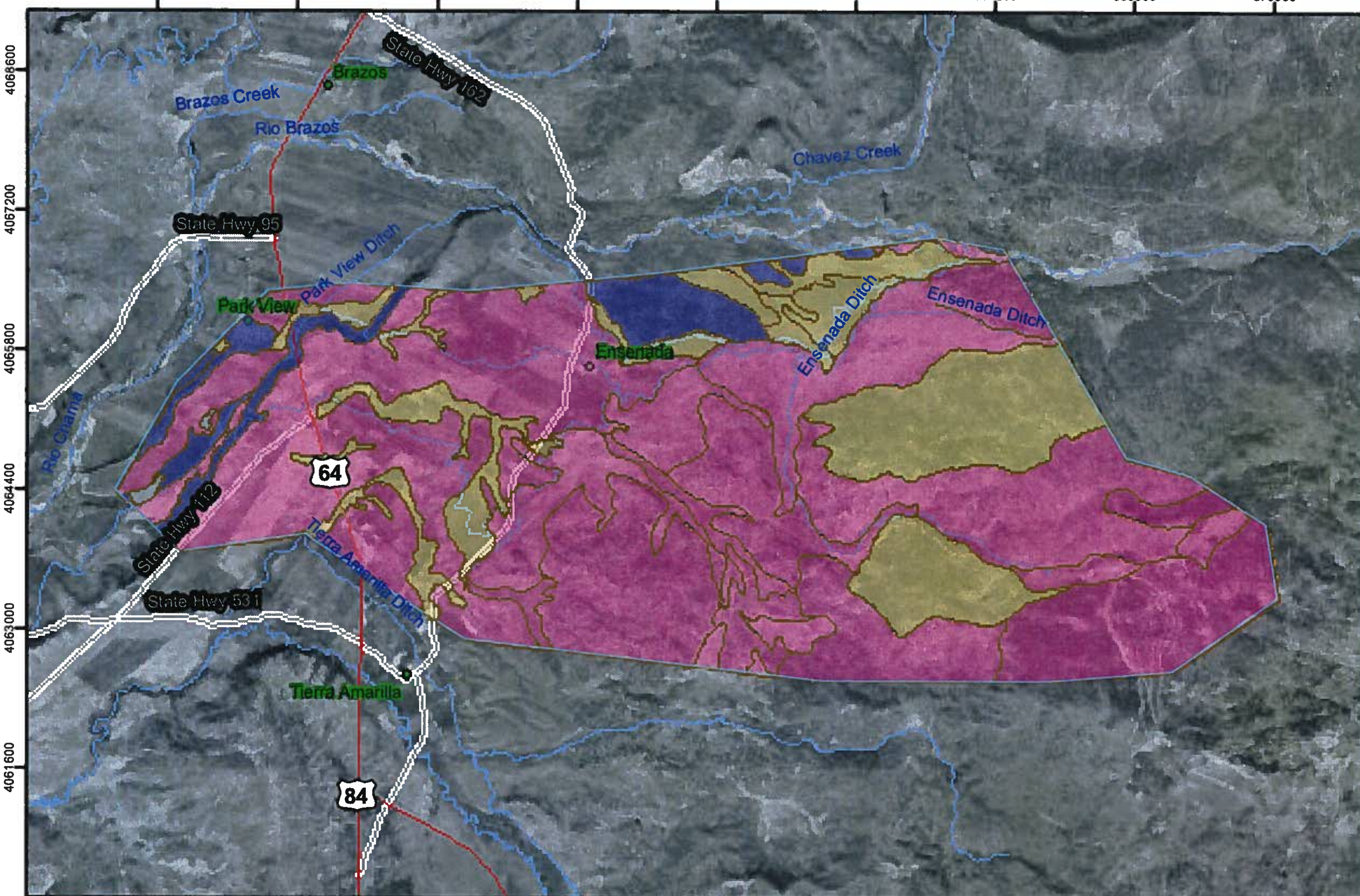
Table 1. Calculation summary of the average minimum soil infiltration rate for the Laguna Del Campo Dam Watershed.

Hydrological Soil Group	Area (ac)	Area Percentage	Min. Infiltration Range (in/hr)*	Suggested Min. Infiltration Rate (in/hr)	Weighted Infiltration Rate (in/hr)
B	86.7	2%	0.15~0.30	0.225	0.005
C	217.6	6%	0.05~0.15	0.1	0.006
D	3,363.8	91%	0.00~0.05	0.025	0.023
Water	14.5	0%	0	0	0.000
Total Area=	3,683			Ave. Infiltration Rate (in/hr):	**0.034**

* Recommended min. soil infiltration rate in Flood Hydrology Manual (1989)

A soil survey report taken from USDA is attached to this study.

Hydrologic Soil Group—Rio Arriba Area, New Mexico, Parts of Rio Arriba and Sandoval Counties
(Laguna Del Campo Watershed Soil Group)



Map Scale: 1:63,800 if printed on A size (8.5" x 11") sheet.

N

Meters: 0 500 1,000 2,000 3,000

Feet: 0 3,500 7,000 14,000 21,000

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Units

Soil Ratings
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Political Features
- Cities

Water Features
- Oceans
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads

MAP INFORMATION

Map Scale: 1:63,800 if printed on A size (8.5" × 11") sheet.

The soil surveys that comprise your AOI were mapped at 1:24,000.

Please rely on the bar scale on each map sheet for accurate map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: UTM Zone 13N NAD83

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Rio Arriba Area, New Mexico, Parts of Rio Arriba and Sandoval Counties
Survey Area Data: Version 10, Dec 19, 2008

Date(s) aerial images were photographed: 10/4/1997; 9/11/1998; 10/5/1997

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Rio Arriba Area, New Mexico, Parts of Rio Arriba and Sandoval Counties				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
54	Capillo silt loam, 0 to 8 percent slopes	D	58.8	0.7%
61	Colomex gravelly silt loam, 0 to 3 percent slopes	B	70.9	0.8%
64	Dula loam, 0 to 2 percent slopes	B	299.9	3.3%
65	Doslomas loam, 0 to 3 percent slopes	C	163.2	1.8%
66	Encicado silty clay loam, 0 to 3 percent slopes	C	445.9	4.9%
117	Chamita loam, 0 to 2 percent slopes	C	195.0	2.2%
119	Roques-Nusmag clay loams, 1 to 8 percent slopes	D	857.4	9.5%
125	Hogg-Mara loams, 2 to 12 percent slopes	D	672.0	7.5%
127	Rombo-Wiggler complex, 5 to 25 percent slopes	D	1,002.2	11.1%
133	Carrick silt loam, 1 to 4 percent slopes	D	2,043.6	22.7%
200	Katlon silt loam, 25 to 45 percent slopes	D	727.3	8.1%
201	Lobat-Abreu gravelly loams, 15 to 60 percent slopes	C	1,004.6	11.1%
203	Nabor-Elbuck complex, 5 to 35 percent slopes	D	1,308.9	14.5%
240	Riverwash	D	26.9	0.3%
242	Tinaja-Rock outcrop complex, 45 to 75 percent slopes	B	127.3	1.4%
W	Water		14.8	0.2%
Totals for Area of Interest			**9,018.8**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Aggregation is the process by which a set of component attribute values is reduced to a single value that represents the map unit as a whole.

A map unit is typically composed of one or more "components". A component is either some type of soil or some nonsoil entity, e.g., rock outcrop. For the attribute being aggregated, the first step of the aggregation process is to derive one attribute value for each of a map unit's components. From this set of component attributes, the next step of the aggregation process derives a single value that represents the map unit as a whole. Once a single value for each map unit is derived, a thematic map for soil map units can be rendered. Aggregation must be done because, on any soil map, map units are delineated but components are not.

For each of a map unit's components, a corresponding percent composition is recorded. A percent composition of 60 indicates that the corresponding component typically makes up approximately 60% of the map unit. Percent composition is a critical factor in some, but not all, aggregation methods.

The aggregation method "Dominant Condition" first groups like attribute values for the components in a map unit. For each group, percent composition is set to the sum of the percent composition of all components participating in that group. These groups now represent "conditions" rather than components. The attribute value associated with the group with the highest cumulative percent composition is returned. If more than one group shares the highest cumulative percent composition, the corresponding "tie-break" rule determines which value should be returned. The "tie-break" rule indicates whether the lower or higher group value should be returned in the case of a percent composition tie.

The result returned by this aggregation method represents the dominant condition throughout the map unit only when no tie has occurred.

Component Percent Cutoff: *None Specified*

Components whose percent composition is below the cutoff value will not be considered. If no cutoff value is specified, all components in the database will be considered. The data for some contrasting soils of minor extent may not be in the database, and therefore are not considered.

Tie-break Rule: Lower

The tie-break rule indicates which value should be selected from a set of multiple candidate values, or which value should be selected in the event of a percent composition tie.

URS **CALCULATION COVER SHEET** Quality

Project Name:	*Laguna Del Campo Dam*	**Project Number:**	*22242013*
Project Location:	*Rio Arriba, NM*	**Client Name:**	*US Fish and Wildlife Service*
PM Name:	*Gregg Batchelder-Adams*	**PIC Name:**	*John France*

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages *(including cover sheet)*: *7*

Discipline: *Dam EAP*

Title of Calculation: *Basin Unit Hydrograph Estimation for Laguna Dam*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Brad Rastall*

DESCRIPTION & PURPOSE

To estimate unit hydrograph for Laguna Del Campo Dam watershed

BASIS / REFERENCE / ASSUMPTIONS

•Cudworth, A.G., Flood Hydrology Manual. United States Department of Commerce Bureau of Reclamation. Denver, Colorado. 1989

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	MHS.J	01.20.2011	[ ]	[ ]
1		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

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APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

[signature] 01.20.2011
Originator Signature Date

Bradley C. Rastall 1/20/2011
Checker Signature Date

[signature] 1/21/11
Project Manager Signature Date

Distribution:
Project Central File – Quality file folder
Other Specify: ____

Purpose:
Generate a unit hydrograph for the Laguna Del Campo watershed for the local PMP storm event and the general PMP storm event.

References:

Cudworth, A.G., *Flood Hydrology Manual.* United States Department of Commerce Bureau of Reclamation. Denver, Colorado. 1989

FILES:

- Calculation Sheets of Unit Hydrograph
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Unit Hydrograph\Basin_UH_Laguna.XLS

- Calculation Cover Sheet and Memo
  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\Unit Hydrograph\
  CCS_ UnitHydrograph_Laguna.doc
  CalcPacket_UnitHydrograph_Laguna.doc

SUMMARY:

The watershed was determined to be located in the "Colorado Plateau" region. The attached calculation sheet shows the estimation of Basin Unit Hydrograph complying with USBR's *Flood Hydrology Manual* (Cudworth, 1989). Required inputs for the spreadsheet included drainage area, basin slope, length of watercourse, distance to centroid, and the average weighted Manning's n value. The catchment of Laguna Del Campo Dam and watershed characteristics are expressed in Figure 1.

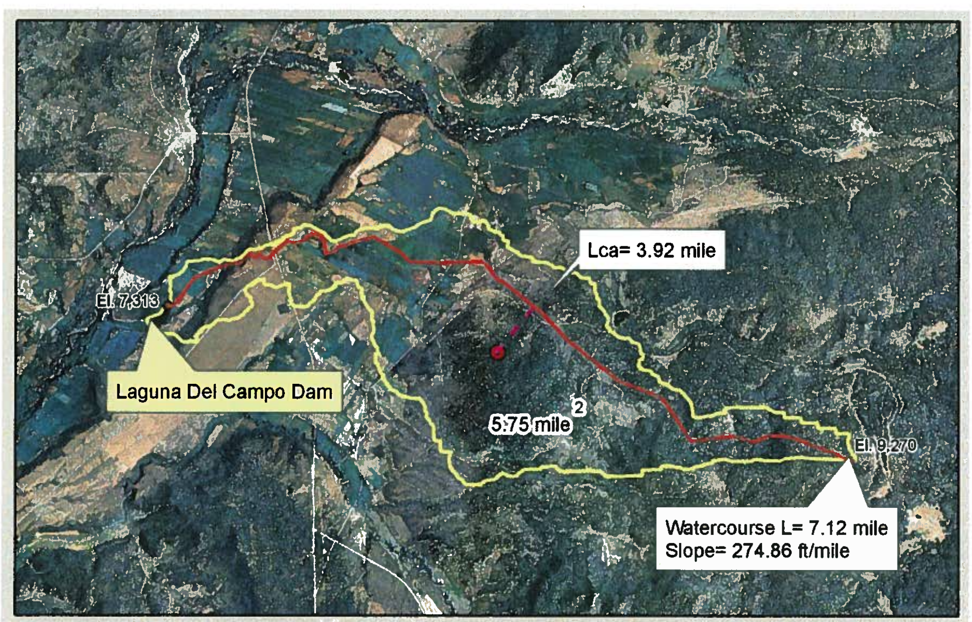


Figure 1. Watershed characteristics of Laguna Del Campo Dam

USBR's *Flood Hydrology Manual* (Cudworth, 1989) states that K_n values are typically between 0.042 and 0.070. Review of the studied watersheds in Table 4-3 of USBR's *Flood Hydrology Manual* , a K_n value of 0.055 was used in this analysis.

A summary of hydrologic parameters used to estimate the unit hydrograph is presented below.

Sub-basin	Basin Area (mi^2)	Main Watercourse Flow Length, **L** (mi)	Distance to Centroid, **L$_{ca}$** (mi)	Main Watercourse Slope, **S** (ft/mi)	Lag Coefficient, **Kn**
Laguna Del Campo	5.75	7.12	3.92	274.86	0.055

The lag time was estimated using the lag time equation presented in Cudworth (1989).

$$L_g = 26K_n\left(\frac{LL_{ca}}{S^{0.5}}\right)^{0.33}$$

Subbasin	**Lag Time (hrs)**
Laguna Del Campo	1.7

COLORADO PLATEAU UNIT HYDROGRAPH 29-Apr-11

Laguna Del Campo Dam

DAMID: NA MS

Drainage Area =	5.75 sq. miles	Lg+D/2 =	1.78 Hours
Basin Slope =	274.86 ft./mile	Basin Factor =	1.68
L =	7.12 mi., Length of Watercourse	V' =	154.62 cfs/Day
Lca =	3.92 mi., Distance to Centroid	Qs =	86.8 * q, cfs
Kn =	0.055 -, Ave. Weighted Manning's n		

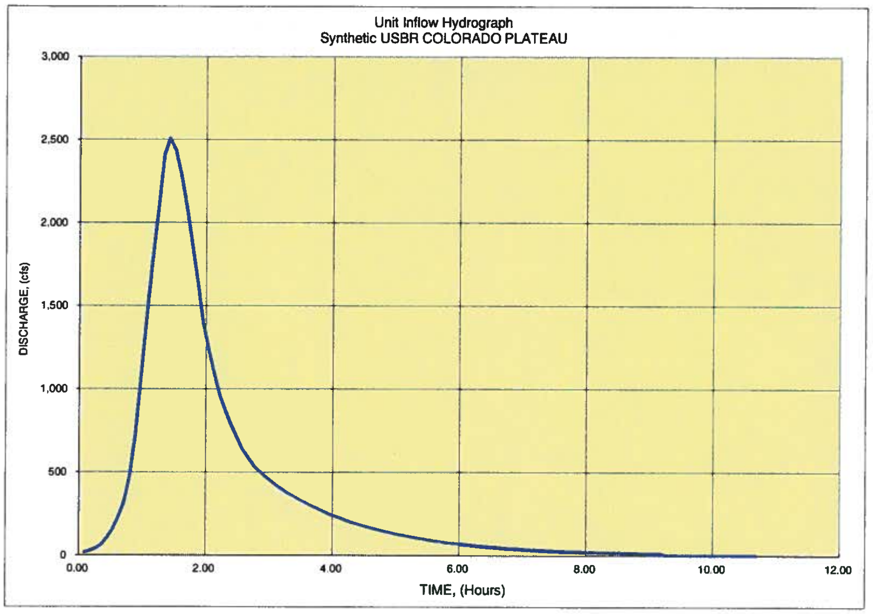
PARAMETERS:

Calculated: Lag Time, Lg = 1.70 Hours Unit Duration, D = 18.53 minutes

Calculated Timestep = 5.34 minutes

Data to be used in Analysis Unit Duration, D = 10 minutes, round down to nearest of 5, 10, 15, 30, 60, 120, 180, or 360

Selected Timestep = 5 minutes, integer value evenly divisible into 60



UI Record - Unit Graph 5 **minute interval**

UI	15	26	39	58	92	137	196	272	385	565
UI	836	1171	1488	1809	2091	2398	2499	2448	2323	2154
UI	1951	1738	1515	1321	1178	1050	935	851	775	707
UI	639	592	545	512	483	456	430	407	386	366
UI	349	331	315	298	283	269	255	240	230	217
UI	206	196	187	177	168	160	152	144	136	129
UI	124	118	112	106	101	95	90	86	81	77
UI	73	70	67	63	59	57	55	51	49	46
UI	44	42	40	38	36	34	32	31	29	28
UI	26	24	23	23	21	20	19	19	18	17
UI	16	15	14	14	13	13	11	10	10	10
UI	1									

USBR calculated unitgraph peak = 2509 Interpolated Peak = 2499

Time t, % of Lg+D/2	Hours	Min.	q	Qs cfs	Time t, % of Lg+D/2	Hours	Min.	q	Qs cfs
5.0	0.09	5.3	0.19	16	305.0	5.43	326.0	1.15	100
10.0	0.18	10.7	0.32	28	310.0	5.52	331.4	1.08	94
15.0	0.27	16.0	0.48	42	315.0	5.61	336.7	1.02	89
20.0	0.36	21.4	0.74	64	320.0	5.70	342.1	0.97	84
25.0	0.45	26.7	1.21	105	325.0	5.79	347.4	0.91	79
30.0	0.53	32.1	1.81	157	330.0	5.88	352.7	0.86	75
35.0	0.62	37.4	2.63	228	335.0	5.97	358.1	0.82	71
40.0	0.71	42.8	3.68	319	340.0	6.06	363.4	0.78	68
45.0	0.80	48.1	5.47	475	345.0	6.15	368.8	0.74	64

HEC-HMS format

5	15
10	26
15	39
20	58
25	92
30	137
35	196
40	272
45	385
50	565
55	836
60	1171
65	1488
70	1809
75	2091
80	2398
85	2499
90	2448
95	2323
100	2154
105	1951
110	1738
115	1515
120	1321
125	1178
130	1050

50.0	0.89	53.4	8.41	730	350.0	6.24	374.1	0.69	60
55.0	0.98	58.8	12.61	1,094	355.0	6.32	379.5	0.66	57
60.0	1.07	64.1	16.50	1,432	360.0	6.41	384.8	0.63	55
65.0	1.16	69.5	20.50	1,779	365.0	6.50	390.2	0.59	51
70.0	1.25	74.8	23.97	2,080	370.0	6.59	395.5	0.56	49
75.0	1.34	80.2	27.75	2,408	375.0	6.68	400.8	0.53	46
80.0	1.43	85.5	28.91	2,509	380.0	6.77	406.2	0.50	43
85.0	1.51	90.9	28.07	2,436	385.0	6.86	411.5	0.47	41
90.0	1.60	96.2	26.38	2,290	390.0	6.95	416.9	0.45	39
95.0	1.69	101.5	24.18	2,099	395.0	7.04	422.2	0.42	36
100.0	1.78	106.9	21.55	1,870	400.0	7.13	427.6	0.40	35
105.0	1.87	112.2	18.92	1,642	405.0	7.22	432.9	0.38	33
110.0	1.96	117.6	16.08	1,396	410.0	7.30	438.3	0.36	31
115.0	2.05	122.9	14.19	1,232	415.0	7.39	443.6	0.34	30
120.0	2.14	128.3	12.61	1,094	420.0	7.48	448.9	0.33	29
125.0	2.23	133.6	11.04	958	425.0	7.57	454.3	0.30	26
130.0	2.32	139.0	9.99	867	430.0	7.66	459.6	0.28	24
135.0	2.41	144.3	9.04	785	435.0	7.75	465.0	0.27	23
140.0	2.49	149.6	8.20	712	440.0	7.84	470.3	0.26	23
145.0	2.58	155.0	7.36	639	445.0	7.93	475.7	0.24	21
150.0	2.67	160.3	6.78	588	450.0	8.02	481.0	0.23	20
155.0	2.76	165.7	6.20	538	455.0	8.11	486.4	0.22	19
160.0	2.85	171.0	5.83	506	460.0	8.19	491.7	0.21	18
165.0	2.94	176.4	5.47	475	465.0	8.28	497.0	0.20	17
170.0	3.03	181.7	5.15	447	470.0	8.37	502.4	0.19	16
175.0	3.12	187.1	4.84	420	475.0	8.46	507.7	0.18	16
180.0	3.21	192.4	4.57	397	480.0	8.55	513.1	0.17	15
185.0	3.30	197.7	4.31	374	485.0	8.64	518.4	0.16	14
190.0	3.38	203.1	4.10	356	490.0	8.73	523.8	0.15	13
195.0	3.47	208.4	3.87	336	495.0	8.82	529.1	0.15	13
200.0	3.56	213.8	3.68	319	500.0	8.91	534.5	0.13	11
205.0	3.65	219.1	3.47	301	505.0	9.00	539.8	0.12	10
210.0	3.74	224.5	3.28	285	510.0	9.09	545.1	0.12	10
215.0	3.83	229.8	3.10	269	515.0	9.17	550.5	0.11	10
220.0	3.92	235.2	2.93	254	520.0	9.26	555.8		
225.0	4.01	240.5	2.75	239	525.0	9.35	561.2		
230.0	4.10	245.8	2.63	228	530.0	9.44	566.5		
235.0	4.19	251.2	2.47	214	535.0	9.53	571.9		
240.0	4.28	256.5	2.33	202	540.0	9.62	577.2		
245.0	4.36	261.9	2.22	193	545.0	9.71	582.6		
250.0	4.45	267.2	2.10	182	550.0	9.80	587.9		
255.0	4.54	272.6	1.99	173	555.0	9.89	593.2		
260.0	4.63	277.9	1.88	163	560.0	9.98	598.6		
265.0	4.72	283.3	1.78	154	565.0	10.07	603.9		
270.0	4.81	288.6	1.68	146	570.0	10.15	609.3		
275.0	4.90	294.0	1.59	138	575.0	10.24	614.6		
280.0	4.99	299.3	1.50	130	580.0	10.33	620.0		
285.0	5.08	304.6	1.43	124	585.0	10.42	625.3		
290.0	5.17	310.0	1.36	118	590.0	10.51	630.7		
295.0	5.26	315.3	1.28	111	595.0	10.60	636.0		
300.0	5.34	320.7	1.21	105	600.0	10.69	641.3		

NOTES : 1. Methodology used Dimensionless Unit Hydrograph.
2. For values of q use Table 4-13 from Flood Hydrology Manual

UH volume = 306.83 AF
Total runoff = 306.67 AF
Ratio = 1.00

135	935
140	851
145	775
150	707
155	639
160	592
165	545
170	512
175	483
180	456
185	430
190	407
195	386
200	366
205	349
210	331
215	315
220	298
225	283
230	269
235	255
240	240
245	230
250	217
255	206
260	196
265	187
270	177
275	168
280	160
285	152
290	144
295	136
300	129
305	124
310	118
315	112
320	106
325	101
330	95
335	90
340	86
345	81
350	77
355	73
360	70
365	67
370	63
375	59
380	57
385	55
390	51
395	49
400	46
405	44
410	42
415	40

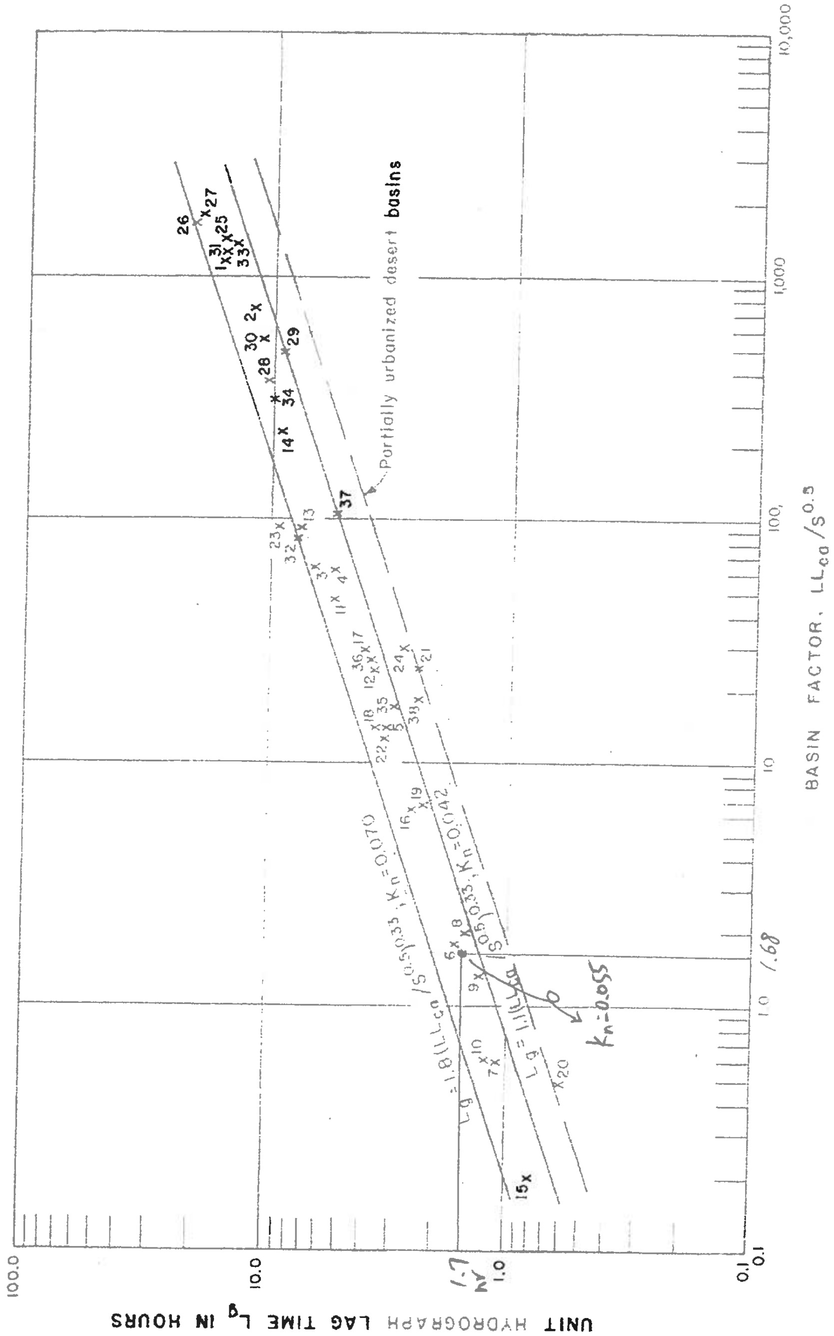


Figure 4-8.—Unit hydrograph lag relationships for the Southwest Desert, Great Basin, and Colorado Plateau. 103-D-1852.

Table 4-3.—Unit hydrograph lag data for the Southwest Desert, Great Basin, and Colorado Plateau.

Index No.	Station and location	Drainage area, mi²	Basin factor, $LL_{ca}/S^{0.5}$	Lag time, L_g, hours	K_n	C_t
1	Salt River at Roosevelt, AZ	4341.0	1261.0	16.0	0.058	1.51
2	Verde R. above E. Verde and below Jerome, AZ	3190.0	760.0	12.0	.052	1.35
3	Tonto Cr. above Gun Cr., AZ	678.0	66.3	6.5	.063	1.64
4	Agua Fria R. nr. Mayor, AZ	590.0	63.2	5.4	.053	1.38
5	San Gabriel R. at San Gabriel Dam, CA	162.0	14.4	3.3	.053	1.38
6	West Fk. San Gabriel R. at Cogswell Dam, CA	40.4	1.8	1.6	.051	1.33
7	Santa Anita Cr. at Santa Anita Dam, CA	10.8	0.6	1.1	.050	1.30
8	Sand Dimas Cr. at San Dimas Dam, CO	16.2	2.0	1.5	.046	1.20
9	Eaton Wash at Eaton Wash Dam, CA	9.5	1.3	1.3	.046	1.20
10	San Antonio Cr. nr. Claremont, CA	16.9	0.6	1.2	.055	1.43
11	Santa Clara R. nr. Saugus, CA	355.0	48.2	5.6	.060	1.56
12	Temecula Cr. at Pauba Canyon, CA	168.0	24.1	3.7	.050	1.30
13	Santa Margarita R. nr. Fallbrook, CA	645.0	99.2	7.3	.062	1.61
14	Santa Margarita R. at Ysidora, CA	740.0	228.0	9.5	.061	1.59
15	Live Oak Cr. at Live Oak Dam, CA	2.3	0.2	0.8	.052	1.35
16	Tujunga Cr. at Big Tujunga Dam, CA	81.4	6.5	2.5	.052	1.35
17	Murrieta Cr. at Temecula, CA	220.0	28.9	4.0	.051	1.33
18	Los Angeles R. at Sepulveda Dam, CA	152.0	14.3	3.5	.056	1.46
19	Pacoima Wash at Pacoima Dam, CA	27.8	6.8	2.4	.049	1.27
20	East Fullerton Cr. at Fullerton Dam, CA	3.1	0.5	0.6	.029	0.75
21	San Jose Cr. at Workman Mill Rd. CA	81.3	24.8	2.4	.032	0.83
22	San Vincente Cr. at Foster, CA	75.0	12.8	3.2	.053	1.38
23	San Diego R. nr. Santee, CA	380.0	95.4	9.2	.078	2.03
24	Deep Cr. nr. Hesperia, CA	137.0	28.1	2.8	.036	0.94
25	Bill Williams R. at Planet, AZ	4730.0	1476.0	16.2	.056	1.46
26	Gila R. at Conner No. 4 Damsite, AZ	2840.0	1722.0	21.5	.071	1.85
27	San Francisco R. at Jct. with Blue R., AZ	2000.0	1688.0	20.6	.068	1.77
28	Blue R., nr. Clifton, AZ	790.0	352.0	10.3	.057	1.48
29	Moencopi Wash nr. Tuba City, AZ	2490.0	473.0	9.2	.046	1.20
30	Clear Cr. nr. Winslow, AZ	607.0	570.0	11.2	.053	1.38
31	Puerco R. nr. Admana, AZ	2760.0	1225.0	15.9	.058	1.51
32	Plateau Cr. nr. Cameo, CO	604.0	89.9	7.9	.069	1.79
33	White R. nr. Watson, UT	4020.0	1473.0	15.7	.054	1.40
34	Paria R. at Lees Ferry, AZ	1570.0	296.0	10.2	.060	1.56
35	New River at Rock Springs, AZ	67.3	16.5	3.1	.047	1.22
36	New River at New River, AZ	85.7	26.3	3.7	.048	1.25
37	New R. at Bell Road nr. Phoenix, AZ	187.0	108.0	5.3	.043	1.12
38	Skunk Cr. nr. Phoenix, AZ	64.6	18.7	2.4	.035	0.91

URS **CALCULATION COVER SHEET** *Quality*

Project Name:	*Laguna Del Campo Dam*	**Project Number:**	*22242013*
Project Location:	*Rio Arriba, NM*	**Client Name:**	*US Fish & Wildlife Service*
PM Name:	*Gregg Batchelder-Adams*	**PIC Name:**	*John France*

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic — File Name:

(Select as appropriate) ☒ Hard-copy — Unique Identification:

Number of pages (including cover sheet): *7*

Discipline: *Dam Breach Analysis*

Title of Calculation: *Calculations of Dam Breach Parameters for Laguna Del Campo Dam*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Brad Rastall*

DESCRIPTION & PURPOSE

To estimate dam breach parameters for Laguna Del Campo Dam

BASIS / REFERENCE / ASSUMPTIONS

Froehlich Breach Predictor Equations (1995)

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	[HKS.]	2011-2-8	[BWR]	2/8/11
1	Rating Curve overflow at dike	☐	☐	☐	[MS]	02.01.2011	[BWR]	2/2/11
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

[signature] — 02.08.2011
Originator Signature — Date

Bradley Rastall — 2/8/11
Checker Signature — Date

[signature] — 2/9/11
Project Manager Signature — Date

Distribution:
Project Central File – Quality file folder
Other Specify: ____

Purpose

This analysis is to compute the dam breach parameters using Froehlich Breach Predictor Equations (1995) for the Laguna Del Campo Dam. Four scenarios are computed in this analysis. They are

1. Sunny Day Breach,
2. Overtop Breach during 72 hour PMP
3. Overtop Breach during 6 hour PMP, and
4. Overtop Breach during 50% 6 hour PMP.

Given

- Calculation Packages
  1. Elevation-Area-Storage Relationship of Laguna Dam (URS, 2011).
  2. Dam Flood Modeling Using HEC-HMS for Laguna Dam (URS, 2011)

Reference

- Rules and Regulations Governing Dam Design, Construction and Dam Safety, Office of the State Engineer, New Mexico (2010).

Files

- MS Excel Calculation Sheets

  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\DamBreachParameters\ DamBreach_Parameters_Laguna.xls

- Calculation Cover Sheet and Summary

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  CCS_DamBreachParameters.doc

  DamBreachParameters.doc

Summary

This dam breach parameter calculation is supporting the dam flood hydrological modeling for the Laguna Dam. According to the study scenarios and requirements of the hydrological models using HEC-HMS, four scenarios were calculated in this analysis. The scenarios are shown in Table 1. Froehlich Breach Predictor Equations (1995) were used to estimate the dam breach parameters. Froehlich Breach Predictor Equations (1995) in SI units are

$$B = 0.1803KV_w^{0.32}h_b^{0.19}$$

$$t = 0.00254V_w^{0.53}h_b^{-0.9}$$

where B= average breach width (m); K= overtopping multiplier, 1.4 for overtopping, 1.0 for piping; V= volume of water-tailing mixture stored above breach invert at time of failure (m^3); h_b= height of breach (m); and t=failure time (hour). The trigger water level for sunny day failure was assumed to be the spillway crest. The trigger water level for each overtop failure scenario was taken from the HEC-HMS hydrological model which indicate the reservoir level causes a probable greatest breach flood during a design PMP event. The calculation sheets are attached to this analysis.

Table 1. Dam failure scenarios and estimated dam breach parameters

Dam Failure Scenario	Failure Type	Trigger W.S.	Water Volume (ft^3)	Time of Failure (hr)	Breach Bottom Width (ft)
Sunny Day	Piping	98.74	4,336,703	0.2	7.5
6 hour PMP	Overtop	105.8	9,350,523	0.2	36.9
50% 6 hour PMP	Overtop	105.1	8,686,822	0.2	35.7

Table 2. Water volumes at the trigger levels for dam breach analysis.

				Existing Pond Volume			
Elevation (ft)	**Area (ft^2)**	**Depth (ft)**	**Acres**	**Incremental Vol. (ft^3)**	**Accumulated Vol. (ft^3)**	**Accumulated Vol. (Ac-Ft)**	
73.0	0	0.0	0.0	-		0.00	
75.0	5,650	2.0	0.1	5,650	5,650	0.13	
80.0	43,525	7.0	1.0	122,937	128,587	2.95	
85.0	122,325	12.0	2.8	414,626	543,213	12.47	
90.0	223,676	17.0	5.1	865,004	1,408,216	32.33	
95.0	339,424	22.0	7.8	1,407,750	2,815,967	64.65	
98.75	471,635	25.8	10.8	1,520,736	4,336,703	99.56	Spillway Crest
99.0	480,449	26.0	11.0	119,011	4,455,713	102.29	
104.0	829,866	31.0	19.1	3,275,788	7,731,501	177.49	Dam Crest
104.5	864,808	31.5	19.9	423,668	8,155,170	187.22	
105.0	899,749	32.0	20.7	441,139	8,596,309	197.34	
105.1	910,504	32.1	20.9	90,513	8,686,822	199.42	
105.5	953,522	32.5	21.9	372,805	9,059,627	207.98	
105.8	985,786	32.8	22.6	290,896	9,350,523	214.66	
106.0	1,007,295	33.0	23.1	199,308	9,549,831	219.23	
110.0	1,437,480	37.0	33.0	4,889,551	14,439,382	331.48	

DAM BREACH PARAMETER CALCULATION

Froehlich Breach Predictor Equations (1995)

Project Name: Laguna Del Campo Dam EAP, NM
Description: Sunny Day Breach - Piping Failure from Outlet Works
Spillway Crest El. 98.75
Job No.: 22242013
Data: 02.07.2011

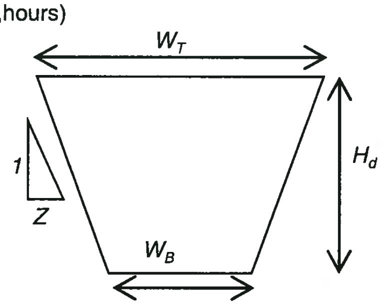
Constant, K=	1	ft (1.4 for overtopping failure and 1.0 for piping failure)
Top of Dam Crest=	104	ft
Stored Water Volume, V_w=	4336703	ft^3
Breach Invert=	73	ft
Breath Slope, Z=	1	H:1V
Crest Length, W=	528	ft
DamFront Face Slope, Zf=	2	H:1V
Dam Back Face Slope, Zb=	3	H:1V

Failure Depth, H_d=	31	ft
Average Breath Width, b=	38.5	ft
Time of Failure, t_f=	0.2	hr
Breach Top Width, W_T=	69.5	ft
Breach Bottom Width, W_B=	7.5	ft
Dam Breach Volume=	648,831	ft^3

Froehlich Breach Predictor Equations (1995) in SI units (meters, m3/s,hours)

$$B = 0.1803\, K V_w^{0.32} h_d^{0.19}$$

$$t_f = 0.00254 V_w^{0.53} h_d^{-0.9}$$



Reference:

1. Froehlich, D. C. 1995(a). "Peak Outflow from Breached Embankment Dam," Water Resources Engineering , Proceedings of the 1995 ASCE Conference on Water Resources Engineering, San Antonio, Texas, August 14-18, 1995, 887-891.

2. Froehlich, D. C. 1995(b). "Embankment Dam Breach Parameters Revisited," Journal of Water Resources Planning and Management, 121(1), 90-97.

Prepared by Hui-Ming (Max) Shih, Ph.D., PE, CFM

DAM BREACH PARAMETER CALCULATION

Froehlich Breach Predictor Equations (1995)

Project Name: Laguna Del Campo Dam EAP, NM
Description: 50% 6hr PMP Breach - Overtopping Failure from Dam Crest
Trigger W.S.=105.1
Job No.: 22242013
Data: 02.29.2011

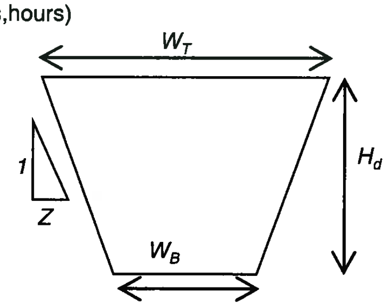
Constant, K=	1.4	ft (1.4 for overtopping failure and 1.0 for piping failure)
Dam Crest/Max W.S.=	105.1	ft
Stored Water Volume, V_w=	8686822	ft^3
Breach Invert=	73	ft
Breath Slope, Z=	1	H:1V
Crest Length, W=	500	ft
DamFront Face Slope, Zf=	2	H:1V
Dam Back Face Slope, Zb=	3	H:1V

Failure Depth, H_d=	32.1	ft
Average Breath Width, b=	67.8	ft
Time of Failure, t_f=	0.2	hr
Breach Top Width, W_T=	99.9	ft
Breach Bottom Width, W_B=	35.7	ft
Dam Breach Volume=	1,180,642	ft^3

Froehlich Breach Predictor Equations (1995) in SI units (meters, m3/s,hours)

$$B = 0.1803\ K V_w^{0.32} h_d^{0.19}$$

$$t_f = 0.00254 V_w^{0.53} h_d^{-0.9}$$



Reference:

1. Froehlich, D. C. 1995(a). "Peak Outflow from Breached Embankment Dam," Water Resources Engineering , Proceedings of the 1995 ASCE Conference on Water Resources Engineering, San Antonio, Texas, August 14-18, 1995, 887-891.

2. Froehlich, D. C. 1995(b). "Embankment Dam Breach Parameters Revisited," Journal of Water Resources Planning and Management, 121(1), 90-97.

Prepared by Hui-Ming (Max) Shih, Ph.D., PE, CFM

DAM BREACH PARAMETER CALCULATION

Froehlich Breach Predictor Equations (1995)

Project Name: Laguna Del Campo Dam EAP, NM
Description: 6hr PMP Breach - Overtopping Failure from Dam Crest
Trigger W.S.=105.8
Job No.: 22242013
Data: 04.29.2011

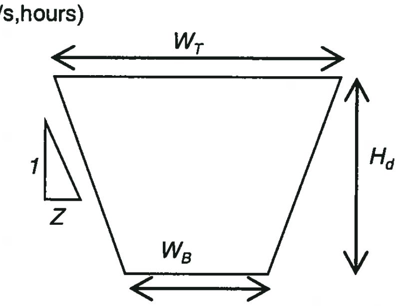
Constant, K=	1.4	ft (1.4 for overtopping failure and 1.0 for piping failure)
Dam Crest/Max W.S.=	105.8	ft
Stored Water Volume, V_w=	9350523	ft^3
Breach Invert=	73	ft
Breath Slope, Z=	1	H:1V
Crest Length, W=	500	ft
DamFront Face Slope, Zf=	2	H:1V
Dam Back Face Slope, Zb=	3	H:1V

Failure Depth, H_d=	32.8	ft
Average Breath Width, b=	69.7	ft
Time of Failure, t_f=	0.2	hr
Breach Top Width, W_T=	102.5	ft
Breach Bottom Width, W_B=	36.9	ft
Dam Breach Volume=	1,242,868	ft^3

Froehlich Breach Predictor Equations (1995) in SI units (meters, m3/s,hours)

$$B = 0.1803\, K V_w^{0.32} h_d^{0.19}$$

$$t_f = 0.00254 V_w^{0.53} h_d^{-0.9}$$



Reference:

1. Froehlich, D. C. 1995(a). "Peak Outflow from Breached Embankment Dam," Water Resources Engineering , Proceedings of the 1995 ASCE Conference on Water Resources Engineering, San Antonio, Texas, August 14-18, 1995, 887-891.

2. Froehlich, D. C. 1995(b). "Embankment Dam Breach Parameters Revisited," Journal of Water Resources Planning and Management, 121(1), 90-97.

Prepared by Hui-Ming (Max) Shih, Ph.D., PE, CFM

DAM BREACH PARAMETER CALCULATION

Froehlich Breach Predictor Equations (1995)

Project Name: Laguna Del Campo Dam EAP, NM
Description: 72hr PMP Breach - Overtopping Failure from Dam Crest
Trigger W.S.=105.5
Job No.: 22242013
Data: 04.29.2011

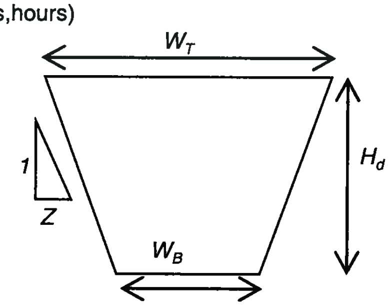
Constant, K=	1.4	ft (1.4 for overtopping failure and 1.0 for piping failure)
Dam Crest/Max W.S.=	105.5	ft
Stored Water Volume, V_w=	9059627	ft^3
Breach Invert=	73	ft
Breath Slope, Z=	1	H:1V
Crest Length, W=	500	ft
DamFront Face Slope, Zf=	2	H:1V
Dam Back Face Slope, Zb=	3	H:1V

Failure Depth, H_d=	32.5	ft
Average Breath Width, b=	68.9	ft
Time of Failure, t_f=	0.2	hr
Breach Top Width, W_T=	101.4	ft
Breach Bottom Width, W_B=	36.4	ft
Dam Breach Volume=	1,215,863	ft^3

Froehlich Breach Predictor Equations (1995) in SI units (meters, m3/s,hours)

$$B = 0.1803\, K V_w^{0.32} h_d^{0.19}$$

$$t_f = 0.00254 V_w^{0.53} h_d^{-0.9}$$



Reference:

1. Froehlich, D. C. 1995(a). "Peak Outflow from Breached Embankment Dam," Water Resources Engineering , Proceedings of the 1995 ASCE Conference on Water Resources Engineering, San Antonio, Texas, August 14-18, 1995, 887-891.

2. Froehlich, D. C. 1995(b). "Embankment Dam Breach Parameters Revisited," Journal of Water Resources Planning and Management, 121(1), 90-97.

Prepared by Hui-Ming (Max) Shih, Ph.D., PE, CFM

URS **CALCULATION COVER SHEET** *Quality*

Project Name:	*Laguna Del Campo Dam*	**Project Number:**	*22242013*
Project Location:	*Rio Arriba, NM*	**Client Name:**	*US Fish and Wildlife Service*
PM Name:	*Gregg Batchelder-Adams*	**PIC Name:**	*John France*

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages *(including cover sheet)*: ~~5~~ 6

Discipline:	*Dam Breach Analysis*
Title of Calculation:	*Dam Flood Hydrograph estimation for Laguna Del Campo Dam*
Calculation Originator:	*Max Shih*
Calculation Contributors:	*[If applicable, names of other contributors]*
Calculation Checker:	*Brad Rastall*

DESCRIPTION & PURPOSE

To estimate dam flood hydrographs using HEC-HMS for Laguna Del Campo Dam

BASIS / REFERENCE / ASSUMPTIONS

HEC-HMS,
Rules and Regulations Governing Dam Design, Construction and Dam Safety, NM Office of the State Engineer

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	MS	02.04.2011	BRGS	02.09.2011
1	Rating Curve overflow at dike	☐	☐	☒	MS1	02.2.2011	BLM	5/2011
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the initial issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

Originator Signature — Date: 02.09.2011

Checker Signature — Date: 2/9/11

Project Manager Signature — Date: 2/10/11

Distribution:

Project Central File – Quality file folder

Other Specify: ____

Date: May 12, 2010

Purpose

The scopes of this analysis are

1.) to use HEC-HMS program to generate the dam flood hydrographs for the selected dam breach scenarios, and
2.) to identify the critical flood events which are required by state regulations.

Given

- Calculation Packages
    1. Elevation-Area-Storage Relationship of Laguna Del Campo Dam (URS, 2011).
    2. Dam Breach Parameters for Laguna Del Campo Dam (URS, 2011)
    3. Dam Stage-Discharge Rating Curve (URS, 2011)
    4. Rainfall Distributions of Local and General PMP events (URS, 2011)
    5. Unit Hydrograph Estimation (URS, 2011)
    6. Site Catchment Soil Infiltration Calculation (URS, 2011)

Files

- HEC-HMS modeling

  N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\DamFloods\HEC-HMS\LagunaDamFloods\ LagunaDamFloods.hms

- Calculation Cover Sheet and Summary

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  DamFlood_Modeling.doc

  CCS_DamFlood_Modeling.doc

Reference

- US Army Corp HEC-HMS v.3.4
- Rules and Regulations Governing Dam Design, Construction and Dam Safety, Office of the State Engineer, New Mexico (2010).
- Dam Safety Inspection Report for Laguna Del Campo, OSE File Nos. D-313 & D-155, Rio Arriba County (June, 2009).

Summary

According to the state rules and regulations of dam safety and recent dam inspection report, the Laguna Del Campo Dam (Laguna Dam) is classified as a small dam with high hazard. In this analysis, three dam breach flood hydrographs were defined and estimated for Laguna Del Campo Dam. They are sunny day dam breach, dam breach floods due to 50% Probable Maximum Precipitation (PMP) and 100% PMP. The evaluation procedures and design flood requirements are complied with state rules and regulations.

HEC-HMS v3.4 program was utilized to estimate dam floods. The initial reservoir surface is assumed as the emergency spillway crest, El. 98.75ft. The outlet works was considered closed or blocked since it is not reachable when extreme rainfall event or dam breach happens. All potential floods due to local PMP (duration=6hr) and general PMP (duration=72hr) were analyzed in this study. For local PMP, state regulations recommend that the rainfall distributions suggested by HMR No.5 and EM 1110-2-1411need to be analyze to define the critical local PMP event. For general PMP, the rainfall distributions based on the center peak distribution and 2/3 peak distribution were utilized to identify the critical general PMP event. The dam floods due to PMP events were estimated and summarized in Table 1. Excluding the consideration of dam failure, the local 6hr PMP expresses the greatest peak outflow from the Laguna Del Campo Reservoir than other PMP scenarios.

In the dam breach analysis, a sunny day failure was considered to occur when the embankment is broken from the outlet structure. The trigger level is at the emergency spillway crest. Overtop failure was applied to the PMP events with dam breach since the simulated PMF outflows would overtop the dam crest and the west dike crest. The trigger levels were defined by repeating calculations of the reservoir level and storage, anticipated dam breach parameters, and dam breach modeling, as well as computing the greatest outflow from the Laguna Dam. The computed peak outflows are shown in Table 1. The results indicate the local PMP dominates the peak outflows from the Laguna Dam. The computed outflow hydrographs of a sunny day breach, a dam flood due to the 50% local PMP, and the dam breach caused by the 100% local PMP support the dam flood inundation analysis. The screen shots of these three hydrographs were shown below.

Overtopping flow from the west dike was calculated using the common weir flow equation and overflow depth above the dike crest over time taken from HEC-HMS modeling results. The computed outflow hydrographs of 100% local PMP and 50% local PMP were separated into overflow hydrographs from the west dike and the breach flow to the downstream of the Laguna Del Campo Dam. Figures 1 and 2 show the computed hydrographs.

Table 1. Summary of computed dam floods for Laguna Del Campo Dam

Flood Scenario			Failure Type	Trigger W.S. (ft)	Max. W.S.(ft)	Overtopping Depth (ft)	Peak Inflow (cfs)	Peak Outflow (cfs)	Critical Event
	Storm	Rainfall Distribution							
No Dam Breach	6hr PMP	EM 1110-2-1411*	N/A	N/A	106.5	2.5	19846	19793	X
		HMR No.5	N/A	N/A	106.5	2.5	19799	19733	
	72hr PMP	Center Peak	N/A	N/A	105.6	1.6	10768	10757	
		2/3 Peak	N/A	N/A	105.6	1.6	10817	10818	
	50% 6hr PMP	EM 1110-2-1411	N/A	N/A	105.5	1.5	9864	9836	
Dam Breach	Sunny Day	N/A	Piping	98.74	98.7	-	0	8019	X
	6hr PMP*	EM 1110-2-1411	Overtop	105.8	105.9	1.9	19846	26903	X
	72hr PMP	2/3 Peak	Overtop	105.5	105.5	1.5	10817	20830	
	50% 6hr PM	EM 1110-2-1411	Overtop	105.1	105.2	1.2	9864	23807	

* Laguna Del Campo is classified as a small high hazard dam. The Inflow Design Flood (IDF) required by state regulations is 100% PMF.

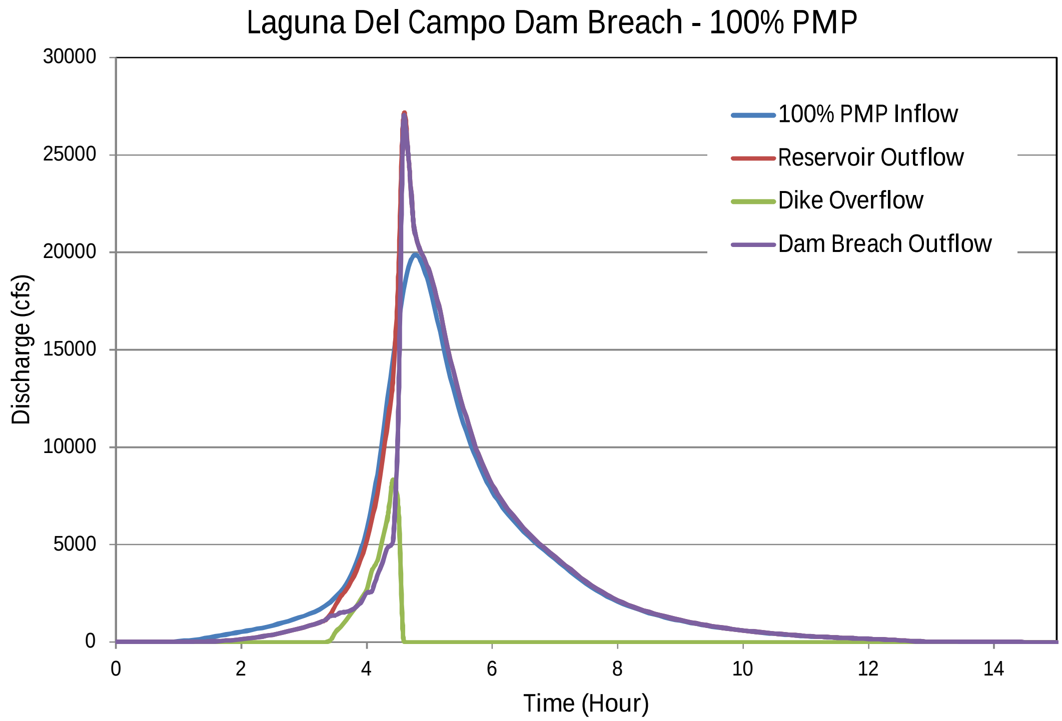

Figure 1. Computed 100% PMP Hydrographs for Laguna Del Campo Reservoir.

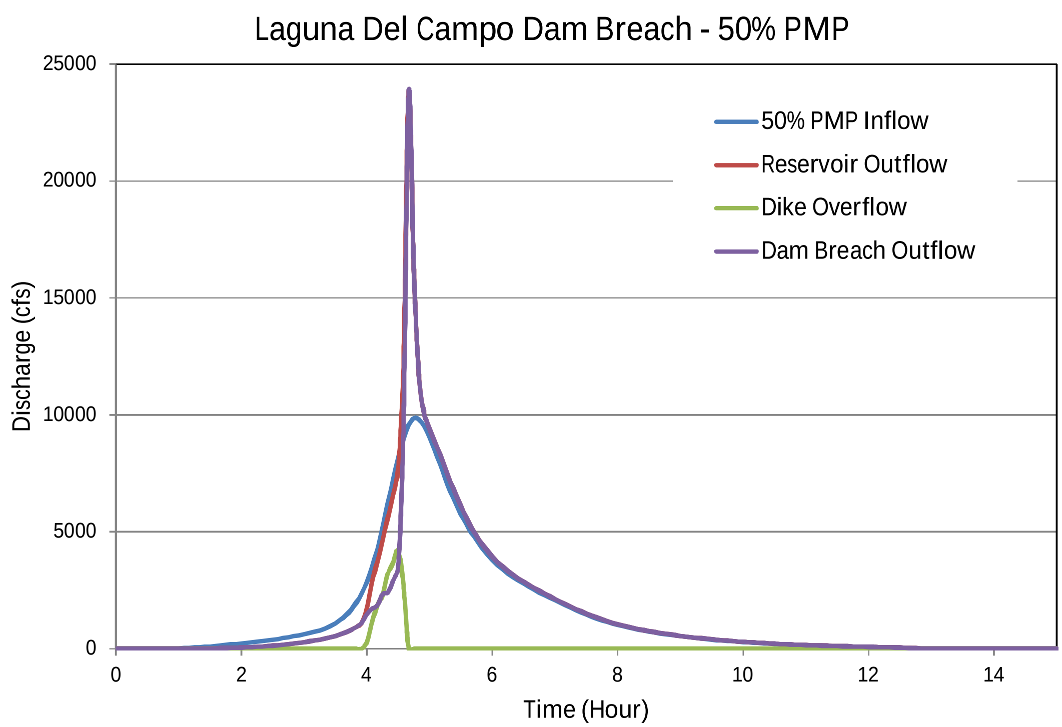

Figure 2. Computed 50% PMP Hydrographs for Laguna Del Campo Reservoir.

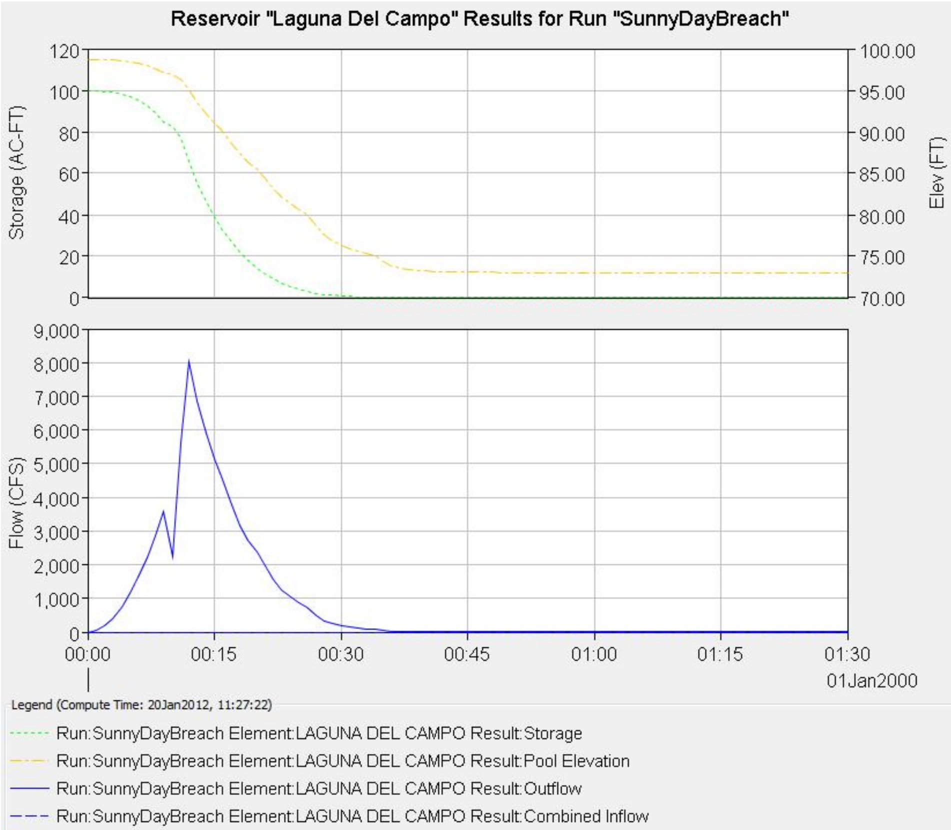

Figure 1. Simulated Sunny Day Brach Hydrograph for Laguna Dam.

Appendix G

Prediction of Embankment Dam Breach Parameters (Wahl, 1998) – Select Pages

H.

Prediction of Embankment Dam Breach Parameters

A Literature Review and Needs Assessment

DSO-98-004

Dam Safety Research Report

by
Tony L. Wahl
Water Resources Research Laboratory

U.S. Department of the Interior
Bureau of Reclamation
Dam Safety Office

July 1998

generally only qualititative or visual in nature. The digital image database is especially interesting.

Predicting Breach Parameters from Case Study Data

Table 2 summarizes the relations proposed by previous investigators for predicting breach parameters (e.g., geometry, time of formation) from case study data. The earliest contributions were made by Johnson and Illes (1976), who published a classification of failure shapes for earth, gravity, and arch dams. For earth dams, the breach shape was described as varying from triangular to trapezoidal as the breach progressed. The great majority of earth dam breaches are described as trapezoidal in the literature.

Table 2. — Breach parameter relations based on dam-failure case studies.
For explanations of symbols see the *Notation* section at the end of this report.

Reference	**Number of Case Studies**	**Relations Proposed (S.I. units, meters, m³/s, hours)**
Johnson and Illes (1976)		$0.5h_d \leq B \leq 3h_d$ for earthfill dams
Singh and Snorrason (1982, 1984)	20	$2h_d \leq B \leq 5h_d$ 0.15 m $\leq d_{ovtop} \leq 0.61$ m 0.25 hr $\leq t_f \leq 1.0$ hr
MacDonald and Langridge-Monopolis (1984)	42	Earthfill dams: $V_{er} = 0.0261(V_{out}*h_w)^{0.769}$ [best-fit] $t_f = 0.0179(V_{er})^{0.364}$ [upper envelope] Non-earthfill dams: $V_{er} = 0.00348(V_{out}*h_w)^{0.852}$ [best fit]
FERC (1987)		B is normally 2-4 times h_d B can range from 1-5 times h_d Z = 0.25 to 1.0 [engineered, compacted dams] Z = 1 to 2 [non-engineered, slag or refuse dams] t_f = 0.1-1 hours [engineered, compacted earth dam] t_f = 0.1-0.5 hours [non-engineered, poorly compacted]
Froehlich (1987)	43	$\overline{B}^{*} \quad 0.47K_o\ S^{*\ \ 0.25}$ K_o = 1.4 overtopping; 1.0 otherwise $Z \quad 0.75K_c(h_w^{*})^{1.57}(\overline{W}^{*})^{0.73}$ K_c = 0.6 with corewall; 1.0 without a corewall $t_f^{*} = 79(S^{*})^{0.47}$
Reclamation (1988)		$B = (3)h_w$ $t_f = (0.011)B$
Singh and Scarlatos (1988)	52	Breach geometry and time of failure tendencies B_{top}/B_{bottom} averages 1.29
Von Thun and Gillette (1990)	57	B, Z, t_f guidance (see discussion)
Dewey and Gillette (1993)	57	Breach initiation model; B, Z, t_f guidance
Froehlich (1995b)	63	$\overline{B} \quad 0.1803K_oV_w^{\ 0.32}h_b^{0.19}$ $t_f = 0.00254\,V_w^{0.53}h_b^{(-0.90)}$ K_o= 1.4 for overtopping; 1.0 otherwise

Singh and Snorrason (1982) provided the first quantitative guidance on breach width. They plotted breach width versus dam height for 20 dam failures and found that breach width was generally between 2 and 5 times the dam height. The failure time, from inception to completion of breach, was generally 15 minutes to 1 hour. They also found that for overtopping failures, the maximum overtopping depth prior to failure ranged from 0.15 to 0.61 meters (0.5 to 2.0 ft).

MacDonald and Langridge-Monopolis (1984) proposed a breach formation factor, defined as the product of the volume of breach outflow (including initial storage and concurrent inflow) and the depth of water above the breach invert at the time of failure. They related the volume of embankment material removed to this factor for both earthfill and non-earthfill dams (e.g., rockfill, or earthfill with erosion-resistant core). Further, they concluded from analysis of the 42 case studies cited in their paper that the breach side slopes could be assumed to be 1h:2v in most cases; the breach shape was triangular or trapezoidal, depending on whether the breach reached the base of the dam. An envelope curve for the breach formation time as a function of the volume of eroded material was also presented for earthfill dams; for non-earthfill dams the time to failure was unpredictable, perhaps because, in some cases, failure may have been caused by structural instabilities rather than progressive erosion. The authors described iterative procedures for estimating breach parameters, simulating breach outflows using DAMBRK or other models, and revising breach parameter estimates as necessary.

Froehlich (1987) developed nondimensional prediction equations for estimating average breach width, average side-slope factor, and breach formation time. The predictions were based on characteristics of the dam, including reservoir volume, height of water above the breach bottom, height of breach, width of the embankment at the dam crest and breach bottom, and coefficients that account for overtopping vs. non-overtopping failures and the presence or absence of a corewall. Froehlich also concluded that, all other factors being equal, breaches caused by overtopping are wider and erode laterally at a faster rate than breaches caused by other means.

Froehlich revisited his 1987 analysis in a 1995 paper, using data from a total of 63 case studies. Eighteen of these failures had not been previously documented in the literature reviewed for this report. Froehlich developed new prediction equations for average breach width and time of failure. In contrast to his 1987 relations, the new equations are not dimensionless. Both 1995 relations had better coefficients of determination than did the 1987 relations, although the difference for the time of failure relation was very slight. Froehlich did not suggest a prediction equation for the average breach side slopes in his 1995 paper, but simply suggested assuming breach side slope factors of $Z = 1.4$ for overtopping failures or $Z = 0.9$ for other failure modes. He noted that the average side slope factor for the 63 case studies was nearly 1.0. The data set showed that there are some significant outliers in this regard.

Reclamation (1988) provided guidance for selecting ultimate breach width and time of failure to be used in hazard classification studies using the SMPDBK model. The suggested values are not intended to yield accurate predictions of peak breach outflows, but rather are intended to produce conservative, upper bound values that will introduce a factor of safety into the hazard classification procedure. For earthen dams, the recommended breach width is 3 times the breach depth, measured from the initial reservoir water level to the breach bottom elevation (usually assumed to be the streambed elevation at the toe of the dam). The recommended time for the breach to develop (hours) is 0.011 times the breach width (meters).

Singh and Scarlatos (1988) documented breach geometry characteristics and time of failure tendencies from a survey of 52 case studies. They found that the ratio of top and bottom breach widths, B_{top}/B_{bottom}, ranged from 1.06 to 1.74, with an average value of 1.29 and standard deviation of 0.180. The ratio of the top breach width to dam height was widely scattered. The breach side slopes were inclined 10-50° from vertical in most cases. Also, most failure times were less than 3 hours, and 50 percent of the failure times were less than 1.5 hours.

Von Thun and Gillette (1990) and Dewey and Gillette (1993) used the data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure. They proposed that breach side slopes be assumed to be 1:1 except for dams with cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (h:v) may be more appropriate.

Von Thun and Gillette proposed the following relationship for average breach width:

$$\bar{B} \quad 2.5h_w \quad C_b \qquad (1)$$

with h_w being the depth of water at the dam at the time of failure, and C_b a function of reservoir storage as follows:

Reservoir Size, m^3	C_b, meters	Reservoir Size, acre-feet	C_b, feet
< $1.23*10^6$	6.1	< 1,000	20
$1.23*10^6$ - $6.17*10^6$	18.3	1,000-5,000	60
$6.17*10^6$ - $1.23*10^7$	42.7	5,000-10,000	140
> $1.23*10^7$	54.9	>10,000	180

They noted that this relationship more accurately fits the full range of historical case study data than do the eroded embankment volume relations based on the breach formation factor proposed by MacDonald and Langridge-Monopolis. The volume of eroded embankment is useful, however, as a check on the reasonableness of breach geometries predicted by other means. Von Thun and Gillette presented a plot of eroded embankment volume versus water outflow volume and the depth of water above the breach invert, with contours indicating upper bounds of reasonable breach geometry estimates. They also noted that the small database of large-dam failures tends to indicate 150 meters (500 ft) as a possible upper bound for breach width.

Von Thun and Gillette proposed two methods for estimating breach formation time. Plots of breach formation time versus depth of water above the breach invert suggested upper and lower bound prediction equations for erosion resistant and easily eroded materials of:

$$t_f = 0.020h_w + 0.25 \qquad \textit{[erosion resistant]} \qquad (2)$$

$$t_f = 0.015h_w \qquad \textit{[easily erodible]} \qquad (3)$$

where t_f is in hours and h_w is in meters.

Von Thun and Gillette also developed equations for breach formation time based on observations of average lateral erosion rates (the ratio of final breach width to breach formation time) versus depth of water above the breach invert. They found a stronger correlation between the lateral erosion rate and depth than for the total breach formation time versus depth. Tests of fuse plug embankments intended to erode easily suggest upper bounds on the lateral erosion rate. Using lateral erosion rate data, Von Thun and Gillette put forth two additional equations:

$$t_f \quad \frac{\overline{B}}{4h_w} \qquad \textit{[erosion resistant]} \qquad (4)$$

$$t_f = \frac{\overline{B}}{4h_w + 61.0} \qquad \textit{[highly erodible]} \qquad (5)$$

with h_w and $\overline{B}$ both given in meters. Each of these equations requires an assumption or prediction of the average breach width.

These equations reflect both case study data and results of controlled laboratory tests of fuse plug embankments (Pugh, 1985) using both highly erodible and slightly cohesive materials.

Predicting Peak Outflows from Case Study Data

In lieu of determining breach parameters and then routing inflow and reservoir storage through the breach, many investigators have used the case study data to develop empirical equations relating peak breach outflow to dam height, reservoir storage volume, or combinations of the two. These relations are summarized in Table 1 and discussed in more detail below. Figures 13 through 15 also graphically show these relations compared to the case study data.

Kirkpatrick (1977) presented data from 13 embankment dam failures and 6 additional hypothetical failures, and proposed a best-fit relation for peak discharge as a function of the depth of water behind the dam at failure. This analysis included data from the failure of St. Francis Dam, California, which was a concrete gravity structure. St. Francis Dam was originally thought to have failed due to piping through the right abutment, but a recent study suggests that it may have failed due to a combination of overturning of a concrete gravity section and landslide failure of the left abutment, and thus may not be appropriate for inclusion in the analysis (Rogers and McMahon, 1993).

The Soil Conservation Service (1981) used the 13 case studies cited by Kirkpatrick to develop a power law equation relating the peak dam failure outflow to the depth of water at the dam at the time of failure. This appears to be an enveloping curve, although three data points are slightly above the curve. Reclamation (1982) extended this work and proposed a similar envelope equation for peak breach outflow using case study data from 21 dams.

NOTATION

	Particle packing factor, ratio of roughness height to roughness spacing
B	Breach width (general)
$\overline{B}$	Average breach width $(B_{top} + B_{bottom})$ / 2
$\overline{B}^*$	Dimensionless average breach width $(\overline{B} / h_b)$
B_{top}	Breach width at top of breach
B_{bottom}	Breach width at bottom of breach
B_{avg}	Average breach width
C_b	Constant in Von Thun and Gillette breach width relation
c	Coefficient in equation for , dependent on aeration and particle packing factors
	Angle of repose
d	Drop in reservoir level through a breach (Walder and O'Connor, 1997)
d_m	Mean roughness height
d_{ovtop}	Depth of overtopping flow at failure
d_s	Equivalent stone diameter
D_c	Height of dam crest relative to dam base (Walder and O'Connor, 1997)
·	Erosion rate, mass/time
E_L	Lateral erosion rate, distance/time
f	Darcy's friction factor
g	Acceleration of gravity
$_s$	Unit weight of solid material
$_w$	Unit weight of water
	Dimensionless parameter relating breach erosion rate and reservoir size (Walder and O'Connor, 1997)
h_b	Height of breach
h_d	Height of dam
h_w	Hydraulic depth of water at dam at failure, above breach bottom
h_w^*	Dimensionless height of water above breach bottom, (h_w/h_b)
J_a	Joint alteration number
J_n	Joint set number
J_r	Joint roughness number
J_s	Relative ground structure number
K_c	Core wall correction factor (0.6 if dam contains a core wall; 1.0 otherwise)
k	Mean vertical erosion rate of breach (Walder and O'Connor, 1997)
k_d	Erosion detachment rate coefficient
K_h	Headcut erodibility index
K_o	Overtopping correction factor (1.4 if failure mode is overtopping; 1.0 otherwise)
	Surface flow resistance factor (analogous to Darcy's f)
M_s	Earth mass strength number
Q_p	Peak breach outflow
Q_p^*	Dimensionless peak breach outflow, $Q_p/g^{1/2}d^{5/2}$, (Walder and O'Connor, 1997)
RQD	Rock quality designation
	Aeration factor, specific weight of air-water mixture divided by specific

	weight of pure water
S	Storage
S^*	Dimensionless storage, (S/h_b^3)
	Shields parameter
$_c$	Critical shear stress
$_e$	Erosionally effective stress
t_f	Breach formation time, hours
t_f^*	Dimensionless breach formation time, $t_f / \sqrt{gh_b}$ (t_f, g, and h_b must be in units that produce a dimensionless t_f^*.)
	Downstream embankment slope angle
v	Flow velocity down embankment slope
v_c	Critical velocity to dislodge riprap particles
V_{er}	Volume of embankment material eroded
V_0, V_{out}	Volume of water discharged through breach (initial storage + inflow during failure)
V_w	Volume of water above breach invert elevation at time of breach
$\overline{W}^*$	Dimensionless average embankment width $(W_{crest}+W_{bottom})/(2h_b)$
y_m	Mean water depth normal to embankment slope
Z	Breach opening side slope factor (Z horizontal:1 vertical)
$Z_{e/u}$	Upstream embankment face slope factor
$Z_{e/d}$	Downstream embankment face slope factor

Appendix H

FLO-2D - Dam Flood Inundation Modeling

URS **CALCULATION COVER SHEET** *Quality*

Project Name:	*Laguna Del Campo Dam*	**Project Number:**	*22242013*
Project Location:	*Rio Arriba, NM*	**Client Name:**	*US Fish and Wildlife Service*
PM Name:	*Gregg Batchelder-Adams*	**PIC Name:**	*John France*

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages *(including cover sheet)*: *6*

Discipline: *Dam Breach Analysis*

Title of Calculation: *Dam flood inundation modeling using FLO-2D for Laguna Del Campo Dam*

Calculation Originator: *Max Shih*

Calculation Contributors: *[If applicable, names of other contributors]*

Calculation Checker: *Chad Martin*

DESCRIPTION & PURPOSE

To estimate dam flood inundation for Laguna Del Campo Dam

BASIS / REFERENCE / ASSUMPTIONS

FLO-2D v2007.06

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☐	[illegible]	02.01.2011	C [?] M	3 [?] 11
1	*Dike overflow addition*	☐	☐	☐	[illegible]	05.02.2011	C [?] M	5 [4] 11
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☒ The calculations associated with this Cover Sheet have been checked.

Originator Signature — Date: 05.02 2011

Checker Signature — Date: 5/4/2011

Project Manager Signature — Date

Distribution:

Project Central File – Quality file folder

Other Specify: _____

Purpose:
FLO-2D models were created to route dam floods from Laguna Del Campo Dam to the El Vodo Reservoir.

Assumptions:

1. DEM data with 10-meter resolution is adequate to represent the natural topography for the purpose of this study.

References

1. Calculation packet titled "Dam Flood Hydrograph Estimation for Laguna Del Campo Dam" by Max Shih on 2/9/2011
2. United States Geological Survey (USGS). The National Map Seamless Server. <http://seamless.usgs.gov/index.php> 2011.

Files/Folders:

FLO-2D Models:
N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\FLO-2D\FLO-2D Models\
50%PMF_w_Breach\
SunnyBreach\
100%PMF_w_Breach\

Summary Table:
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Calculation Cover Sheet and Summary:
N:\Projects\22242013_USFWS4_NM_Dams_EAP\Sub_00\10.0_Calculations_Analysis_Data\Laguna Del Campo Dam\FLO-2D\CalcPacket_FLO-2D -Laguna.doc

General Approach
FLO-2D (version 2007.06) was used in this analysis. Approximately 13 miles of river was included in the FLO-2D model. A map of the model is presented in the figure below.

Elevation Data
The FLO-2D topographic surface was generated from 10-meter, USGS digital elevation model (DEM) data obtained from USGS (2011). It should be noted that USGS DEM's often do not adequately resolve roadway embankments, stream crossings and areas of recent development, therefore, the flood routing results may not provide adequate detail near the aforementioned areas.

Grid Size
A grid element size of 50 feet was selected and was assumed to adequately resolve the topography, yet limit the total number of cells so as to implement the modeling. It was determined that 50-foot grid cells resulted in the highest model resolution with a reasonable computation time. The modeling area is shown in Figure 1.

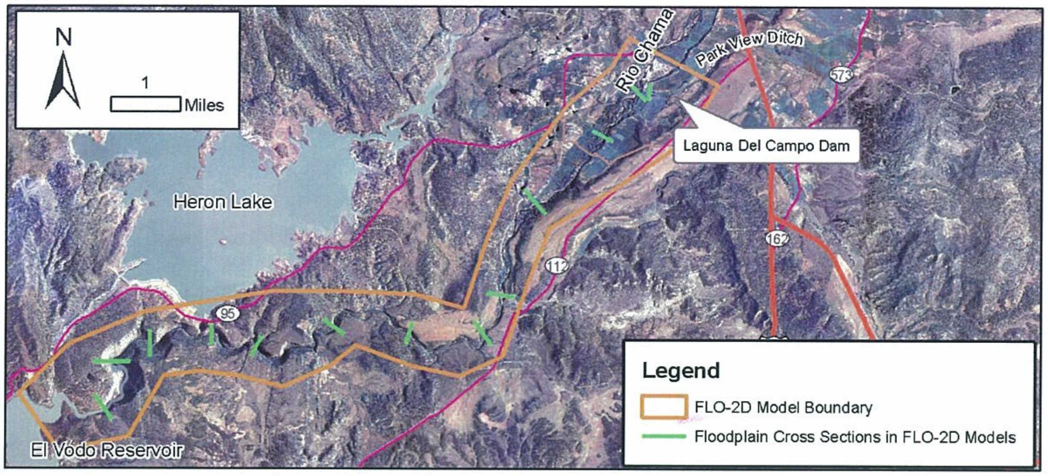


Figure 1. Dam flood inundation modeling area in FLO-2D Model

Manning's n:
The routed floodwave travels over floodplains that contain scattered brush, agriculture farms, and trees throughout the model. Based upon the aerial photographs, and engineering judgment, a Manning's n-value of 0.035 and 0.05 was assigned to the channel and floodplain area respectively.

Inflows:
The inflow hydrographs were obtained from the HEC-HMS model. They are sunny day breach, 100% PMF breach, and 50% PMP breach. For 100% PMF and 50% PMP breach events, there are two outflows, overtopping flow from the west dike and breach flow from the dam, from the reservoir. The overtopping flow from the dike was considered as a wide spread flow from the 1030 ft long dike crest. In the FLO-2D model input, nineteen cells along the west dike were assigned as inflow elements to release overtopping flow from the reservoir. The breach bottom width of each scenario is in a range between 7.5 ft to 37 ft which is less than one cell size, 50 ft. Therefore, one inflow cell was used to represent the dam breach inflow element in the FLO-2D models for all flood inundation scenarios.

Additional model computational input parameters:
- Maximum floodplain Froude number = 2.00
- Shallow flow Manning's n value = 0.20
- Surface detention = 0.10
- Percent change in flow depth = 0.20
- Dynamic wave stability coefficient = 1.00
- No infiltration loss during routing

Flood Plain Cross Sections:
Floodplain cross-sections were created at locations within the model of critical interest including structures, channel influences, and contraction or expansion, so that output data could be easily examined as the floodwave progresses downstream. The computed results at each floodplain cross sections are shown below.

Flood Arrival Time Computation:
The dam breach timing is initiated (t = 0) at breach initiation. According to the HEC-HMS reservoir routing outputs, the PMF dam breach timing is initiated (t = 0) at 4.17 hour from the beginning of the 50% PMP and 100% PMP events. These values were used as a cutoff to compute the floodwave arrival and peak stage times obtained from the FLO-2D outputs.

D/S of Laguna Del Campo Dam			
0.33 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	7,279	23,945	28,070
Max. Water Surface Elevation (ft)	7254.6	7259.6	7259.9
Max. Flood Stage (ft)	6.9	10.8	11.4
Time to Max. Stage (hr:min)	0:13	0:30	0:25
Time to Floodwave Arrival (hr:min)	0:09	0:01	0:01
Peak Velocity (ft/sec)	36.0	48.4	50.2

Confluence with Rio Chama			
0.48 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	5,755	22,831	30,515*
Max. Water Surface Elevation (ft)	7224.2	7228.3	7230.3
Max. Flood Stage (ft)	4.5	7.0	7.7
Time to Max. Stage (hr:min)	0:16	0:33	0:28
Time to Floodwave Arrival (hr:min)	0:14	0:13	0:07
Peak Velocity (ft/sec)	5.7	9.7	10.5

* Flood peak raise due to the additional lateral inflow from the dike overflow.

U/S of State HWY 572			
1.47 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	2,644	19,461	27,958
Max. Water Surface Elevation (ft)	7184.4	7187.6	7188.0
Max. Flood Stage (ft)	3.9	7.3	7.8
Time to Max. Stage (hr:min)	0:36	0:42	0:38
Time to Floodwave Arrival (hr:min)	0:34	0:25	0:19
Peak Velocity (ft/sec)	6.4	8.4	9.0

D/S of State HWY 572			
2.95 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	1,887	16,689	25,908
Max. Water Surface Elevation (ft)	7146.9	7152.5	7154.3
Max. Flood Stage (ft)	4.4	10.0	11.7
Time to Max. Stage (hr:min)	1:12	0:55	0:50
Time to Floodwave Arrival (hr:min)	0:59	0:43	0:31
Peak Velocity (ft/sec)	7.6	8.3	9.9

Confluence with Rito de Tierra Amarilla			
4.48 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	244	13,604	24,435
Max. Water Surface Elevation (ft)	7103.6	7110.0	7112.7
Max. Flood Stage (ft)	2.2	8.8	11.5

Time to Max. Stage (hr:min)	3:07	1:15	1:03
Time to Floodwave Arrival (hr:min)	2:54	1:07	0:55
Peak Velocity (ft/sec)	2.7	9.1	9.8

West of Highway 112			
5.15. Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	207	12,829	23,893
Max. Water Surface Elevation (ft)	7085.9	7092.9	7095.4
Max. Flood Stage (ft)	1.4	7.9	10.3
Time to Max. Stage (hr:min)	3:44	1:25	1:10
Time to Floodwave Arrival (hr:min)	3:44	1:19	1:01
Peak Velocity (ft/sec)	4.9	16.7	18.7

U/S of Heron Res. Outlet			
6.65 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	122	11,474	22,978
Max. Water Surface Elevation (ft)	7053.6	7062.2	7065.4
Max. Flood Stage (ft)	1.2	9.5	12.9
Time to Max. Stage (hr:min)	5:48	1:42	1:23
Time to Floodwave Arrival (hr:min)	5:23	1:37	1:13
Peak Velocity (ft/sec)	4.1	18.6	21.5

U/S of Heron Res. Outlet			
8.15 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	87	10,902	22,330
Max. Water Surface Elevation (ft)	7006.9	7016.1	7020.1
Max. Flood Stage (ft)	1.7	10.1	14.3
Time to Max. Stage (hr:min)	7:56	1:57	1:33
Time to Floodwave Arrival (hr:min)	7:56	1:49	1:25
Peak Velocity (ft/sec)	2.8	18.3	22.5

U/S of Heron Res. Outlet			
9.73 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	56	9,955	21,424
Max. Water Surface Elevation (ft)	6958.7	6968.4	6973.2
Max. Flood Stage (ft)	0.9	10.6	15.4
Time to Max. Stage (hr:min)	10:44	2:13	1:46
Time to Floodwave Arrival (hr:min)	10:44	2:07	1:37
Peak Velocity (ft/sec)	1.7	13.0	18.5

U/S of Heron Res. Outlet			
10.53 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	56	8,756	19,675

Max. Water Surface Elevation (ft)	6936.3	6947.6	6952.8
Max. Flood Stage (ft)	2.1	13.3	18.5
Time to Max. Stage (hr:min)	14:24	2:28	1:58
Time to Floodwave Arrival (hr:min)	13:16	2:19	1:49
Peak Velocity (ft/sec)	0.9	11.8	17.1

D/S of Heron Res. Outlet			
11.56 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	26	7,756	19,033
Max. Water Surface Elevation (ft)	6922.1	6931.6	6937.0
Max. Flood Stage (ft)	1.2	10.7	16.1
Time to Max. Stage (hr:min)	20:01	2:46	2:37
Time to Floodwave Arrival (hr:min)	17:45	2:31	2:01
Peak Velocity (ft/sec)	0.8	10.0	15.5

U/S of El Vado Reservior			
12.27 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	22	7,142	18,956
Max. Water Surface Elevation (ft)	6920.6	6928.9	6933.5
Max. Flood Stage (ft)	0.9	9.1	13.6
Time to Max. Stage (hr:min)	23:02	3:14	2:38
Time to Floodwave Arrival (hr:min)	20:49	2:43	2:07
Peak Velocity (ft/sec)	0.7	8.6	11.2

Entrance of El Vado Reservior			
13.17 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	13	6,054	15,778
Max. Water Surface Elevation (ft)	6917.9	6925.3	6928.9
Max. Flood Stage (ft)	1.1	8.5	12.1
Time to Max. Stage (hr:min)	29:36	3:28	2:44
Time to Floodwave Arrival (hr:min)	26:08	3:01	2:19
Peak Velocity (ft/sec)	1.0	7.2	10.4

Appendix I

Flood Inundation Maps

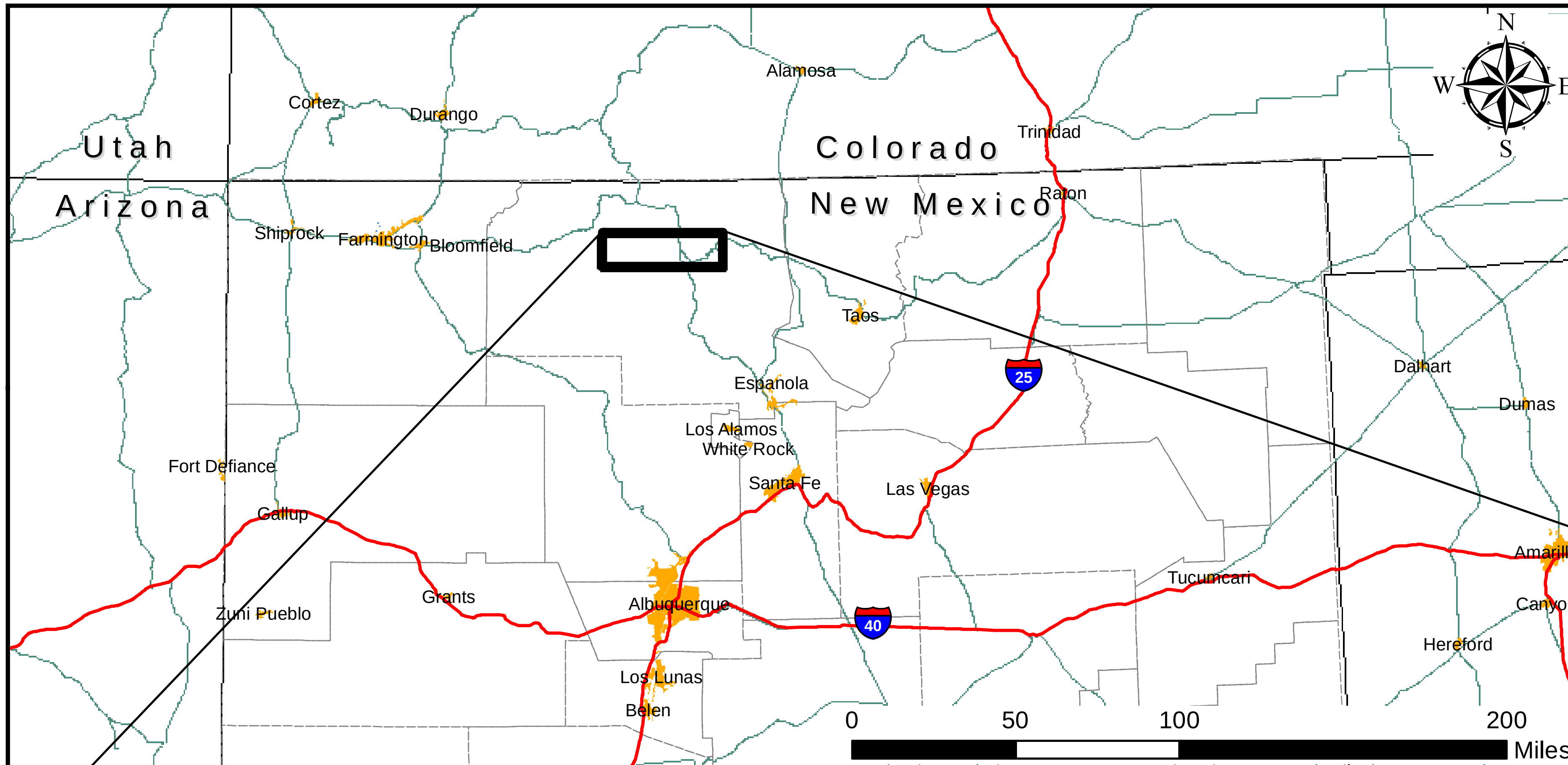
Laguna Del Campo Dam

Dam Breach Analysis
Dam Flood Inundation
Map Index

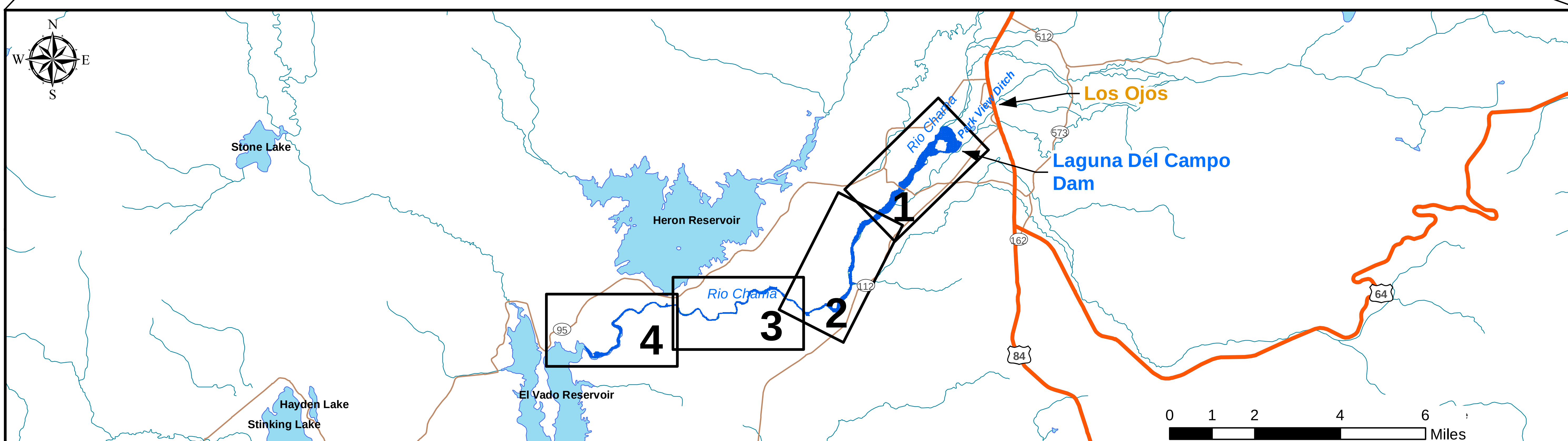
Prepared For:
New Mexico Department of Game and Fish

Prepared By:
URS

JULY 2012
DAM NID NM 00313
OSE File No. D-313



Location Maps



U/S of State HWY 572			
1.47 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	2,644	19,461	27,958
Max. Water Surface Elevation (ft)	7184.4	7187.6	7188.0
Max. Flood Stage (ft)	3.9	7.3	7.8
Time to Max. Stage (hr:min)	0:36	0:42	0:38
Time to Floodwave Arrival (hr:min)	0:34	0:25	0:19
Peak Velocity (ft/sec)	6.4	8.4	9.0

Confluence with Rio Chama			
0.48 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	5,755	22,831	30,515*
Max. Water Surface Elevation (ft)	7224.2	7228.3	7230.3
Max. Flood Stage (ft)	4.5	7.0	7.7
Time to Max. Stage (hr:min)	0:16	0:33	0:28
Time to Floodwave Arrival (hr:min)	0:14	0:13	0:07
Peak Velocity (ft/sec)	5.7	9.7	10.5

*Flood peak increase due to the additional lateral inflow from the dike overflow

D/S of Laguna Del Campo Dam			
0.33 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	7,279	23,945	28,070
Max. Water Surface Elevation (ft)	7254.6	7259.6	7259.9
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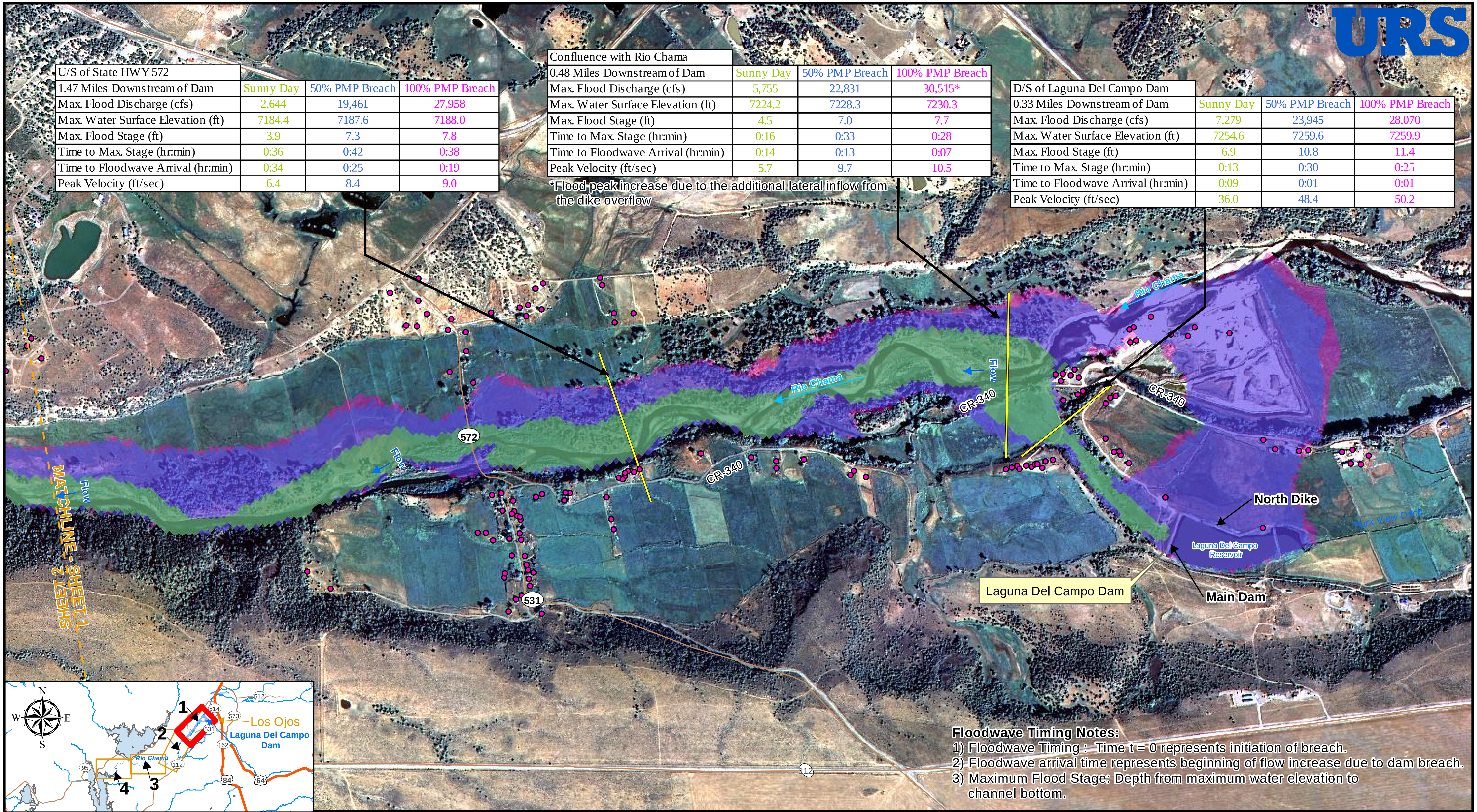
Time to Max. Stage (hr:min)	0:13	0:30	0:25
Time to Floodwave Arrival (hr:min)	0:09	0:01	0:01
Peak Velocity (ft/sec)	36.0	48.4	50.2



Floodwave Timing Notes:

1) Floodwave Timing : Time t = 0 represents initiation of breach.
2) Floodwave arrival time represents beginning of flow increase due to dam breach.
3) Maximum Flood Stage: Depth from maximum water elevation to channel bottom.

LEGEND

- Possible Occupied Building
- Sunny Day Breach Inundation Limits
- 50% PMP Dam Breach Inundation Limits
- 100% PMP Dam Breach Inundation Limits

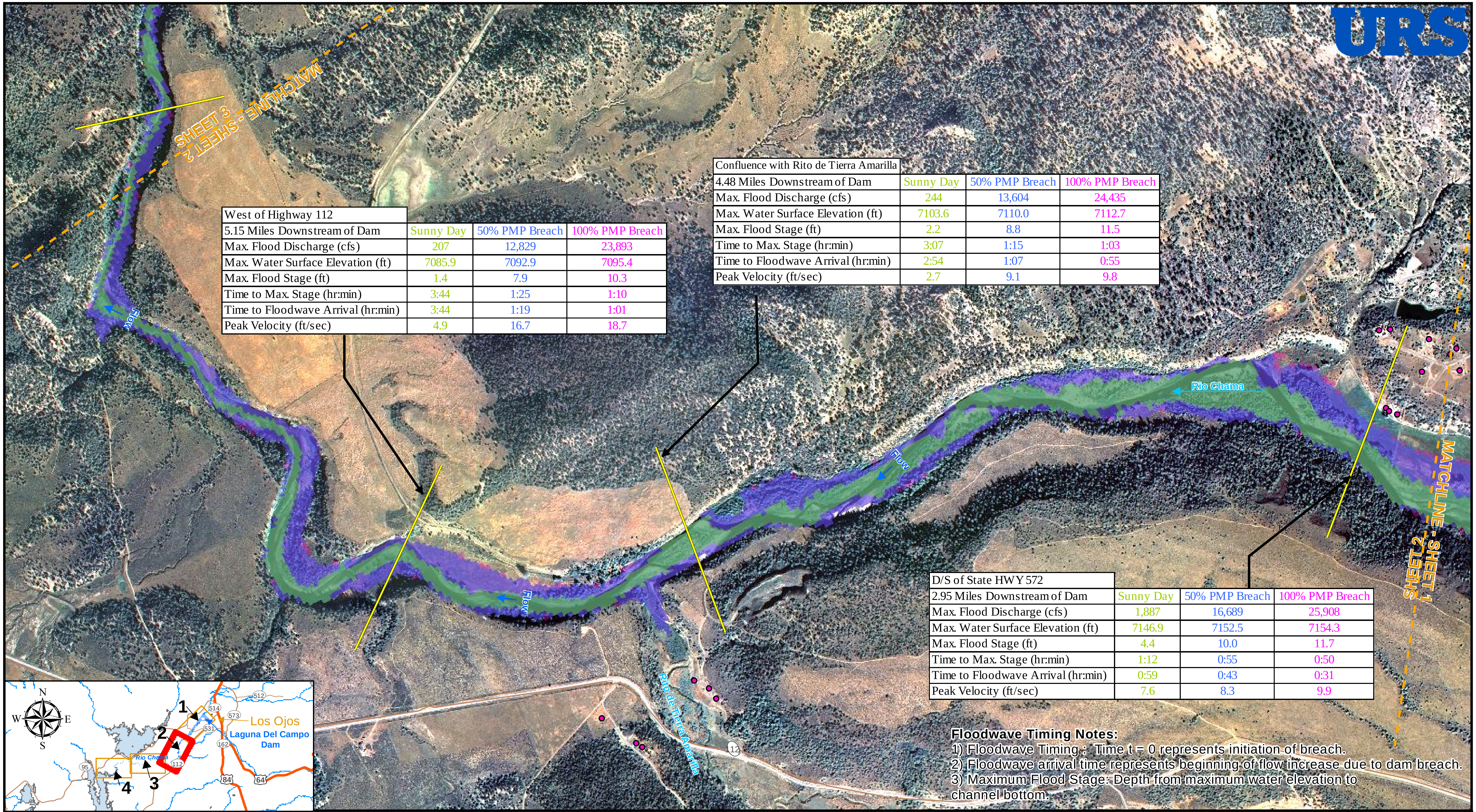
MAP SCALE

0 500 1,000 2,000 Feet

General Note: Inundation zones are based solely on floodwaters from the watershed above Laguna Del Campo Dam. No contributing tributary flow was considered. Methods, procedures, and assumptions used to develop the flooded areas, the limits of the flooding shown and floodwave travel times are approximate and should only be used as a guideline for establishing evacuation zones. Actual areas inundated will depend on actual failure conditions and may differ from the areas shown on this map.

Laguna Del Campo Dam
Dam Breach Analysis
Dam Flood Inundation Map
Sheet 1 of 4

July 2012



West of Highway 112			
5.15 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	207	12,829	23,893
Max. Water Surface Elevation (ft)	7085.9	7092.9	7095.4
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Time to Floodwave Arrival (hr:min)	3:44	1:19	1:01
Peak Velocity (ft/sec)	4.9	16.7	18.7

Confluence with Rito de Tierra Amarilla			
4.48 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	244	13,604	24,435
Max. Water Surface Elevation (ft)	7103.6	7110.0	7112.7
Max. Flood Stage (ft)	2.2	8.8	11.5
Time to Max. Stage (hr:min)	3:07	1:15	1:03
Time to Floodwave Arrival (hr:min)	2:54	1:07	0:55
Peak Velocity (ft/sec)	2.7	9.1	9.8

D/S of State HWY 572			
2.95 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	1,887	16,689	25,908
Max. Water Surface Elevation (ft)	7146.9	7152.5	7154.3
Max. Flood Stage (ft)	4.4	10.0	11.7
Time to Max. Stage (hr:min)	1:12	0:55	0:50
Time to Floodwave Arrival (hr:min)	0:59	0:43	0:31
Peak Velocity (ft/sec)	7.6	8.3	9.9

Floodwave Timing Notes:

1) Floodwave Timing: Time t = 0 represents initiation of breach.
2) Floodwave arrival time represents beginning of flow increase due to dam breach.
3) Maximum Flood Stage: Depth from maximum water elevation to channel bottom.

LEGEND

- Possible Occupied Building
- Sunny Day Breach Inundation Limits
- 50% PMP Dam Breach Inundation Limits
- 100% PMP Dam Breach Inundation Limits

N S E W

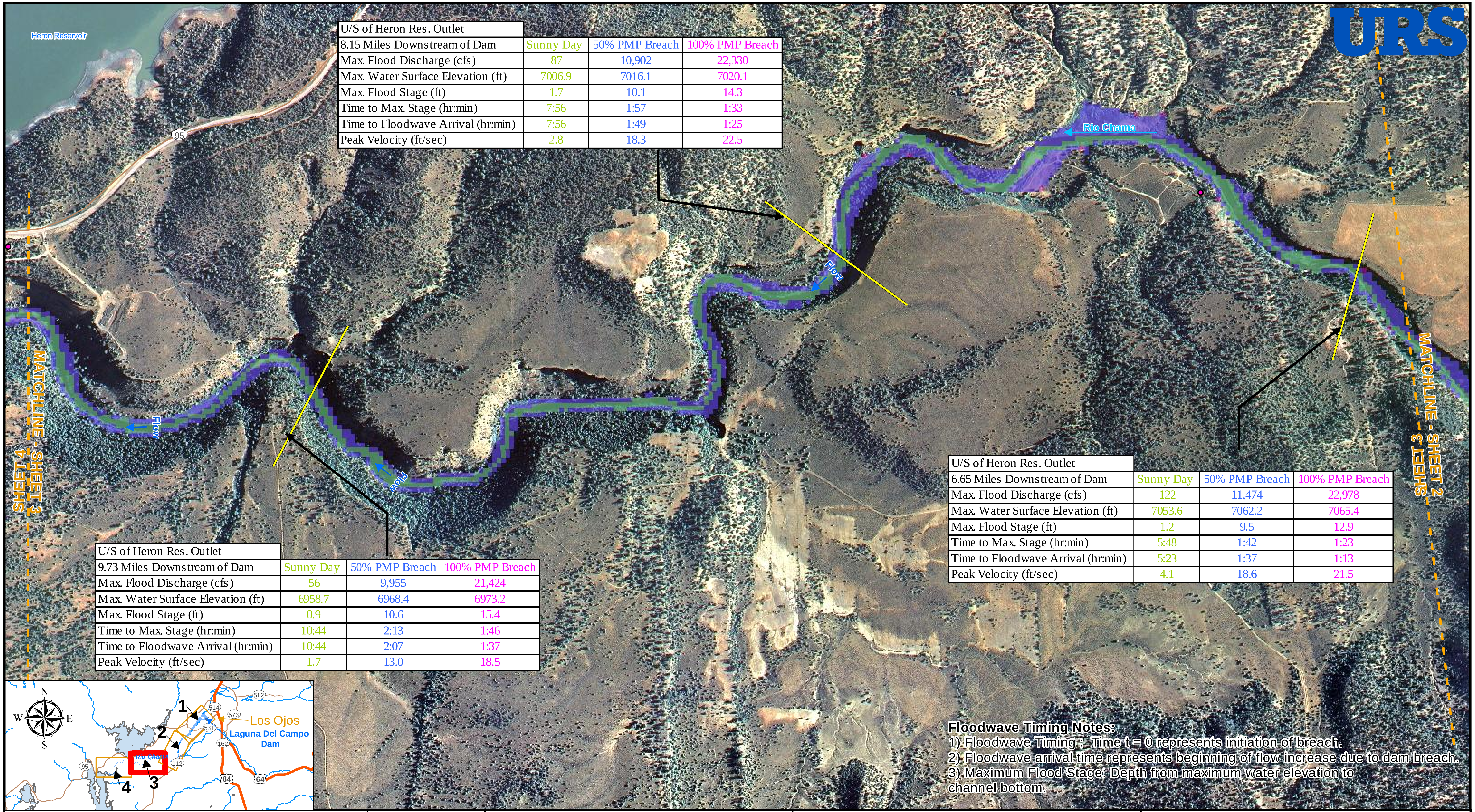
MAP SCALE

0 500 1,000 2,000 Feet

General Note: Inundation zones are based solely on floodwaters from the watershed above Laguna Del Campo Dam. No contributing tributary flow was considered. Methods, procedures, and assumptions used to develop the flooded areas, the limits of the flooding shown and floodwave travel times are approximate and should only be used as a guideline for establishing evacuation zones. Actual areas inundated will depend on actual failure conditions and may differ from the areas shown on this map.

Laguna Del Campo Dam
Dam Breach Analysis
Dam Flood Inundation Map
Sheet 2 of 4

July 2012



U/S of Heron Res. Outlet			
8.15 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	87	10,902	22,330
Max. Water Surface Elevation (ft)	7006.9	7016.1	7020.1
Max. Flood Stage (ft)	1.7	10.1	14.3
Time to Max. Stage (hr:min)	7:56	1:57	1:33
Time to Floodwave Arrival (hr:min)	7:56	1:49	1:25
Peak Velocity (ft/sec)	2.8	18.3	22.5

U/S of Heron Res. Outlet			
6.65 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	122	11,474	22,978
Max. Water Surface Elevation (ft)	7053.6	7062.2	7065.4
Max. Flood Stage (ft)	1.2	9.5	12.9
Time to Max. Stage (hr:min)	5:48	1:42	1:23
Time to Floodwave Arrival (hr:min)	5:23	1:37	1:13
Peak Velocity (ft/sec)	4.1	18.6	21.5

U/S of Heron Res. Outlet			
9.73 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	56	9,955	21,424
Max. Water Surface Elevation (ft)	6958.7	6968.4	6973.2
Max. Flood Stage (ft)	0.9	10.6	15.4
Time to Max. Stage (hr:min)	10:44	2:13	1:46
Time to Floodwave Arrival (hr:min)	10:44	2:07	1:37
Peak Velocity (ft/sec)	1.7	13.0	18.5

Floodwave Timing Notes:

1) Floodwave Timing - Time t = 0 represents initiation of breach.
2) Floodwave arrival time represents beginning of flow increase due to dam breach.
3) Maximum Flood Stage: Depth from maximum water elevation to channel bottom.

LEGEND

- Possible Occupied Building
- Sunny Day Breach Inundation Limits
- 50% PMP Dam Breach Inundation Limits
- 100% PMP Dam Breach Inundation Limits

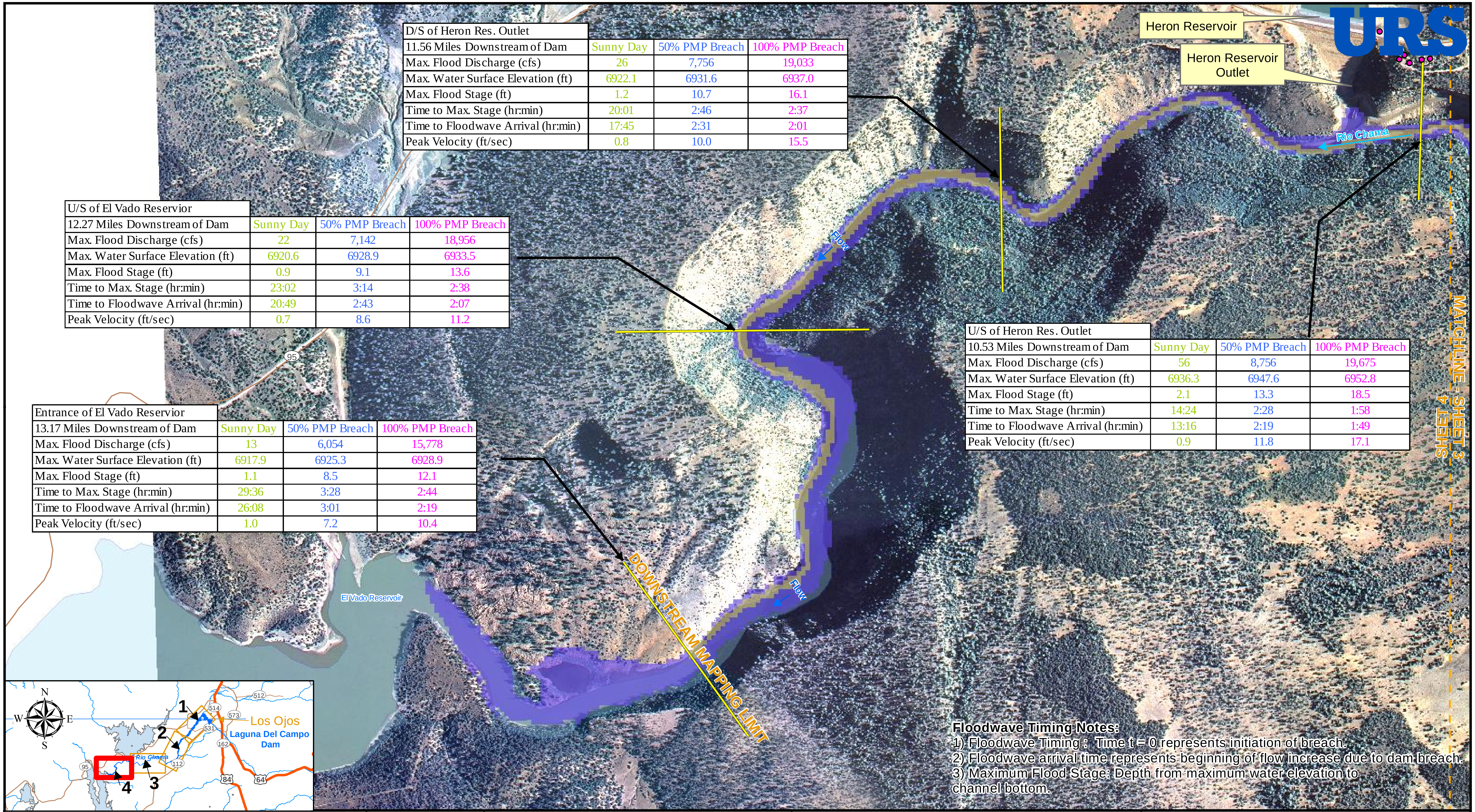
MAP SCALE

0 500 1,000 2,000 Feet

General Note: Inundation zones are based solely on floodwaters from the watershed above Laguna Del Campo Dam. No contributing tributary flow was considered. Methods, procedures, and assumptions used to develop the flooded areas, the limits of the flooding shown and floodwave travel times are approximate and should only be used as a guideline for establishing evacuation zones. Actual areas inundated will depend on actual failure conditions and may differ from the areas shown on this map.

Laguna Del Campo Dam
Dam Breach Analysis
Dam Flood Inundation Map
Sheet 3 of 4

July 2012



D/S of Heron Res. Outlet			
11.56 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	26	7,756	19,033
Max. Water Surface Elevation (ft)	6922.1	6931.6	6937.0
Max. Flood Stage (ft)	1.2	10.7	16.1
Time to Max. Stage (hr:min)	20:01	2:46	2:37
Time to Floodwave Arrival (hr:min)	17:45	2:31	2:01
Peak Velocity (ft/sec)	0.8	10.0	15.5

U/S of El Vado Reservior			
12.27 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	22	7,142	18,956
Max. Water Surface Elevation (ft)	6920.6	6928.9	6933.5
Max. Flood Stage (ft)	0.9	9.1	13.6
Time to Max. Stage (hr:min)	23:02	3:14	2:38
Time to Floodwave Arrival (hr:min)	20:49	2:43	2:07
Peak Velocity (ft/sec)	0.7	8.6	11.2

U/S of Heron Res. Outlet			
10.53 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	56	8,756	19,675
Max. Water Surface Elevation (ft)	6936.3	6947.6	6952.8
Max. Flood Stage (ft)	2.1	13.3	18.5
Time to Max. Stage (hr:min)	14:24	2:28	1:58
Time to Floodwave Arrival (hr:min)	13:16	2:19	1:49
Peak Velocity (ft/sec)	0.9	11.8	17.1

Entrance of El Vado Reservior			
13.17 Miles Downstream of Dam	Sunny Day	50% PMP Breach	100% PMP Breach
Max. Flood Discharge (cfs)	13	6,054	15,778
Max. Water Surface Elevation (ft)	6917.9	6925.3	6928.9
Max. Flood Stage (ft)	1.1	8.5	12.1
Time to Max. Stage (hr:min)	29:36	3:28	2:44
Time to Floodwave Arrival (hr:min)	26:08	3:01	2:19
Peak Velocity (ft/sec)	1.0	7.2	10.4

Floodwave Timing Notes:
1) Floodwave Timing : Time t = 0 represents initiation of breach.
2) Floodwave arrival time represents beginning of flow increase due to dam breach.
3) Maximum Flood Stage: Depth from maximum water elevation to channel bottom.

LEGEND

- Possible Occupied Building
- Sunny Day Breach Inundation Limits
- 50% PMP Dam Breach Inundation Limits
- 100% PMP Dam Breach Inundation Limits

N W E S

MAP SCALE

0 500 1,000 2,000 Feet

General Note: Inundation zones are based solely on floodwaters from the watershed above Laguna Del Campo Dam. No contributing tributary flow was considered. Methods, procedures, and assumptions used to develop the flooded areas, the limits of the flooding shown and floodwave travel times are approximate and should only be used as a guideline for establishing evacuation zones. Actual areas inundated will depend on actual failure conditions and may differ from the areas shown on this map.

Laguna Del Campo Dam
Dam Breach Analysis
Dam Flood Inundation Map
Sheet 4 of 4

July 2012