APACHE JUNCTION FLOOD RETENTION STRUCTURE

DOWNSTREAM INUNDATION STUDY

Prepared For

USDA SOIL CONSERVATION SERVICE
PHOENIX, ARIZONA

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I. PURPOSE OF STUDY

Discharges from the Emergency Spillway of the Apache Junction FRS are possible in the event of precipitation exceeding a 100-year return period (the Principal Spillway design storm). A hypothetical dam failure at a critical time and location is also considered in order to present a worst-case scenario. The extent of downstream inundation is determined for each of the above cases, for use in emergency planning.

II. SCOPE OF STUDY

The study consists of routing discharge hydrographs, computing flow depths and velocities, and delineating inundated areas for the spillway flows and breach flows at two critical locations. Six events are analyzed:

Spillway Discharges:

- o Emergency Spillway Hydrograph (ESH)
- o Freeboard Hydrograph (FBH)

Dam Breach Just West of Idaho Road:

- o At the peak reservoir level resulting from the Principal Spillway Hydrograph (PSH)
- o At the peak reservoir level resulting from the Freeboard Hydrograph (FBH), i.e., the top of the dam.

Dam Breach at the Southeast Corner of the Apache Junction FRS:

- o At the peak reservoir level resulting from the PSH
- o At the peak reservoir level resulting from the FBH, i.e., the top of the dam.

The Idaho Road breach location is critical for the inundation of developed areas close to the dam. The breach at the southeast corner of the FRS would release water into a relatively confined channel, resulting in minimum attenuation of the flood peak and maximum depth and velocity of flow. The downstream extent of the mapping is limited by the availability of detailed topography. The study also shows the sensitivity of the water level computations to the uncertainty of the Manning roughness coefficient (n).

III. METHODS OF ANALYSIS

A. Runoff Hydrographs

The Apache Junction FRS controls a watershed of 5.79 square miles. Inflow hydrographs for the principal spillway design (PSH) resulting from the 100-year precipitation, and for the emergency spillway design (ESH and FBH), resulting from the ESH precipitation and the PMP, respectively, were developed in the preliminary design study (Reference 1) using the SCS DAMS2 computer program (Reference 2). The hydrographs are shown in tabular form in the report. The DAMS2 program also provided routed spillway discharge hydrographs for the ESH and FBH events, which constitute the upstream boundary conditions for the first two cases listed in II above.

B. Roughness Coefficients

Roughness coefficients, defined as "n" in Manning's equation, are necessary both for routing flood hydrographs and for determining depths and velocities of flows. No field observations of flows are available in the study area from which "n" values could be determined. Field estimates of "n" were made at numerous locations upstream of the flood control works and presented in the Signal Butte FRS Design Hydrology (Reference 3). The estimated values were plotted as a function of slope; those in the areas having slopes of 0.01 to 0.03 ranged between 0.04 and 0.08. These "n" values may be assumed applicable to channels in the downstream study area, which has slopes of .01 to .02. Channels are generally unvegetated and consist of fine gravel. Overbank areas are of sand and coarser gravel and have the "desert brush" type of vegetation; "n" values should be considerably higher. Thus "n" was taken as 0.04 for channels and 0.08 for overbanks as a minimum, and 0.08 and 0.16 respectively as a maximum, and computations were made with both sets of values.

C. Routing of ESH and FBH Discharges

The gradually varied spillway discharge hydrographs corresponding to the ESH and FBH were routed downstream using the HEC-1 computer program, "Flood Hydrograph Package" (Reference 4). The Modified Puls routing option using normal depth storage and outflow was used. Four routing reaches were used, each represented by a typical cross-section defined by eight ground elevation points. Although the normal depth assumption is reasonably accurate for determining storage, it does not produce accurate depths, so the water surface profiles calculated by HEC-1 were not used.

D. Dam Breach Outflows

The maximum potential for downstream flooding would occur if a breach in the dam were to form at the maximum reservoir level corresponding to the FBH, which is the level of the crest of the dam. The probability of this event is extremely small, because the probability of the FBH storm itself is extremely small. An additional case considered is failure at the maximum level resulting from the PSH (i.e., the 100-year storm). In this case a breach would have to be formed by piping, because the water level is nearly 10 feet below the crest of the dam. Since the reservoir contains water only for a period of a few days, a piping failure is also extremely unlikely.

Guidelines on the size and rate of formation of a breach, which determine the peak outflow, are limited. For this study, assumptions should be conservative; therefore it is assumed that the dam is eroded all the way down to the base, and that the breach formation is rapid (5 minutes). Fread (Reference 5) states that the average width of a breach in an earthfill dam ranges from 1 to 3 times the height of the dam. A conservative width of 3 times the height is chosen for this study. For this condition, and breach side slopes of 1:1, Fread's DAMBREAK computer program calculates a maximum outflow of 16,455 cfs for failure at the peak of the FBH storage, and 5300 cfs for the PSH case.

SCS Circular No. 1 (Reference 6) provides two relations for a minimum acceptable value of the peak discharge. The first is as follows:

$$Q_{max} = 1100\ B_r^{1.35}$$

where $B_r = V_s\ H_w/A$ and

V_s = reservoir storage in acre-ft, 2015 for the FBH;

H_w = depth of water at the dam in ft, 20.37 for the FBH;

A = cross-sectional area of the embankment, 1219 ft^2

resulting in Q_{max} = 127,000 cfs for the FBH case. This would require a breach of about 450 feet in width. Not only is such a breach very unlikely, but during the time required for its formation, the reservoir storage would be depleted and the stage drawn down such that the peak discharge would be much less. Therefore this criterion is not considered applicable to such a low dam. An alternate criterion is given as:

$$Q_{max} = 65\ H_w^{1.85}$$

This relation gives Q_{max} of 17,160 cfs for the FBH case and 5280 cfs for the PSH case. These agree very closely with the results computed by the DAMBREAK program, so the latter are used in this study.

E. Dam Breach Flood Routing

The DAMBREAK computer program (Reference 5) was the preferred method for calculation of the downstream propagation of the breach hydrograph. The program solves the equations of unsteady flow to account for the acceleration components of the flood wave and the influence of unsteady backwater conditions. It can treat either subcritical or supercritical flow, but not a transition through critical depth. Non-convergence problems were experienced with the model for both the spillway outflow and dam breach hydrographs. The slope of the downstream terrain is mostly in the region treated by the model as supercritical (over 50 feet per mile), but because of the lack of significant channels, the actual flow is predominantly subcritical, as revealed by steady-flow profile computations. Numerous attempts to achieve convergence by simplifying the cross-section geometry, varying the Manning coefficients, increasing the assumed initial flow, and imposing a uniform slope were unsuccessful.

T.R. No. 66 (Reference 7) provides dimensionless solutions to dambreak equations based on the Att-Kin hydrologic method and the method of characteristics. This approach has the disadvantage of only treating a single reach, by averaging the storage characteristics of all cross-sections within the reach. However, since variations in the downstream topography are not drastic, T.R. 66 is a suitable alternative to the DAMBREAK program.

The application of T.R. 66 involves first the determination of whether a curvilinear or triangular hydrograph is appropriate. Since the flow just downstream of the breach is subcritical, the curvilinear hydrograph applies. Second, the storage characteristics of the downstream reaches are resolved into the parameters "m" and "k" by the method of averages, for a range of flows spanning the anticipated variation of the peak flow. Finally, peak flows and times of peak at selected downstream sections are determined from the provided nomograph as a function of m and k. T.R. 66 presents the above technique as a step-by-step procedure.

F. Computation of Water Surface Profiles and Flow Velocity

None of the routing methods discussed above can compute accurate water depths because of the necessity of simplifying cross-section data. Since the terrain is quite flat in the direction perpendicular to the flow, accurate profiles are important; a one-foot rise in water level can result in the inundation of many additional acres. Therefore, water surface profiles were calculated for a range of steady flows using the HEC-2 computer program (Reference 8).

For each of the three downstream flow paths, cross-sections were selected at intervals of 10 to 20 feet of elevation. Each section was extended until sufficient area was contained between the elevation of the ridges and the channel bottoms to convey the anticipated flows. Generally there were no more than 6 feet of difference between the high and low ground elevations for most sections. The HEC-2 program was run to compute profiles for a range of steady flows and for the low (.04 and .08) and high (.08 and .16) Manning "n's". Relatively low areas or obvious channels were designated as "channels" with the lower n values, for the purpose of calculating maximum velocities of flow. Typically, 25 to 50% of the width of a section was designated as "channel" and the rest as "overbank." Although the upper limits of n values (i.e., 0.08 for channels and 0.16 for overbanks) seem high compared to normal usage, it should be noted that the flow will be extremely shallow, especially in overbank areas, so that relatively small roughness elements will have a significant effect on the flow. Therefore the effective roughness is probably higher than the terrain would indicate.

IV. RESULTS OF STUDY

A. Routings of Outflow Hydrographs

Peak discharges and the travel times of the peaks were determined as far downstream as the limits of the available detailed topography. The HEC-1 program was used for the spillway discharges, and the T.R. 66 procedure for the Idaho Road and Southeast Corner breaches; the results are shown in Figures 1, 2, and 3, respectively. The uncertainty in the Manning coefficients results in significant differences in travel times. For the purpose of emergency planning, the shorter travel times corresponding to the low "n" values should be used.

B. Water Surface Profiles and Channel Velocities

From the peak discharge curves in Figures 1, 2, and 3, flows were interpolated for each of the cross-sections used in the water surface profile computations. For these flows, corresponding water surface elevations and mean channel velocities were interpolated from the data produced by the HEC-2 runs. The results are shown in Tables 1, 2, and 3, and continuous water surface profiles are plotted in Figures 4, 5, and 6. Although the peak discharges are lower when high "n" values are used, the water levels are higher at most locations because of the reduced conveyance of the section (by more than a foot in some areas). The higher water levels are used to delineate potential inundation. Flow velocities range up to about 8 ft per second with the low "n" values and 6 ft per second with the high "n's" for the FBH events. These velocities do not represent possible localized higher velocities resulting from eddies or small channels.

C. Inundation Maps

Three inundation maps have been prepared for the spillway and the two breach locations, based on the profiles shown in Figures 4, 5, and 6 (for the high "n" values). Some engineering judgment was necessary in areas where the computed water surface elevation exceeded all ground elevations in the selected cross-sections; also in areas where channels disappeared, originated, or traveled diagonally to the flow path between cross-section locations. The inundated areas shown on the maps are dependent on the accuracy of the topography. Therefore they will not reflect any future changes resulting from development, construction, or possible changes in drainage patterns resulting from local heavy rainstorms.

D. Other Breach Locations

It is evident from the inundation maps for the two FBH cases that almost any location within the mapped area downstream from the FRS could be flooded by a breach at the peak of the FBH. For lower reservoir levels such as the PSH case shown, there would be numerous dry areas. It is not considered practical to delineate all of these. These breach events could only occur as a result of major rainstorms, which would cause considerable flooding in the same area if the FRS were not in place.

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2. Soil Conservation Service, "DAMS2 (Interim Version) Structure Site Analysis Computer Program," Technical Release No. 48, Department of Agriculture, October 1982.

3. Soil Conservation Service, "Design Hydrology, Signal Butte F.R.S., Buckhorn - Mesa W.P.P.," unpublished notes, 1981.

4. Hydrologic Engineering Center, "HEC-1 Flood Hydrograph Package," U.S. Army Corps of Engineers, Davis, California, September 1981.

5. Fread, D. L., "DAMBRK: The NWS Dam-Break Flood Forecasting Model," Office of Hydrology, National Weather Service, Silver Spring, MD, July 18, 1984.

6. Soil Conservation Service, National Engineering Manual, 210-V, Circular No. 1, "Subject: Dam Breach Discharge Criteria," USDA, Washington, DC, July 14, 1980.

7. Soil Conservation Service, Technical Release No. 66, "Simplified Dam-Breach Routing Procedure," USDA, Washington, DC, September 1985.

8. Hydrologic Engineering Center, "HEC-2 Water Surface Profiles," U.S. Army Corps of Engineers, Davis, California, January 1981.

TABLE I

SPILLWAY DISCHARGE PROFILES

DISTANCE FROM DAM (MI)	LOW N DISCHARGE (cfs)	LOW N STAGE (ft. MSL)	LOW N VEL** (ft/sec)	HIGH N DISCHARGE (cfs)	HIGH N STAGE (ft. SML)	HIGH N VEL** (ft/sec)
	EMERGENCY SPILLWAY HYDROGRAPH					
0.028	1898*	1785.27	2.8	1898*	1786.42	1.8
0.114	1895	1779.80	4.7	1890	1780.10	3.0
0.161	1894	1775.08	5.2	1880	1775.99	2.5
0.388	1888	1757.12	5.4	1850	1757.33	4.3
0.691	1880*	1737.08	4.6	1833*	1737.95	2.5
0.991	1855	1717.52	5.1	1780	1717.64	4.4
1.303	1830	1697.27	5.3	1740	1698.30	2.0
1.682	1809*	1676.61	4.1	1676*	1676.58	4.1
2.098	1765	1657.08	2.4	1620	1657.67	1.1
2.430	1722*	1641.20	4.6	1519*	1641.31	3.6
2.717	1690	1627.50	5.5	1440	1628.64	2.1
2.960	1670	1618.06	4.1	1340	1617.81	5.2
3.174	1651*	1608.39	2.0	1233*	1609.30	1.3
	FREEBOARD HYDROGRAPH					
0.028	10600*	1788.28	6.3	10600*	1789.61	3.9
0.114	10595	1781.11	5.9	10590	1782.03	3.6
0.161	10590	1776.92	7.6	10580	1777.86	4.9
0.388	10580	1759.07	7.5	10510	1759.80	5.7
0.691	10520*	1738.81	8.9	10341*	1740.28	4.6
0.991	10410	1719.72	8.8	10080	1720.26	6.6
1.303	10260	1699.40	6.2	9750	1700.22	3.9
1.682	10088*	1677.62	5.8	9321*	1678.22	3.5
2.098	9955	1658.74	3.2	9060	1659.64	1.9
2.430	9871*	1642.77	7.8	8943*	1643.62	4.8
2.717	9760	1629.84	4.6	8790	1630.61	2.7
2.960	9650	1618.95	6.4	8600	1619.55	3.4
3.174	9513*	1610.68	3.7	8356*	1611.14	2.6

*Computed flow - others interpolated
**Mean velocity in deeper channel areas

TABLE 2

DAM BREACH DISCHARGE PROFILES FOR
BREACH AT IDAHO ROAD

DISTANCE FROM DAM (MI)	LOW N			HIGH N		
	DISCHARGE (cfs)	STAGE (ft. MSL)	VEL** (ft/sec)	DISCHARGE (cfs)	STAGE (ft. MSL)	VEL** (ft/sec)
	BREACH WITH PSH					
0	5302*	-	-	5302*	-	-
0.1	5060	1779.33	6.2	4880	1780.18	3.2
0.18	4800	1773.37	4.3	4610	1773.63	3.2
0.29	4560*	1765.26	3.8	4295*	1766.02	1.8
0.52	4270	1750.12	3.8	3880	1750.28	2.9
0.83	4080*	1729.34	5.9	3500*	1730.22	2.6
1.03	3900	1716.62	3.0	3300	1716.66	2.4
1.21	3710*	1706.42	5.7	3075*	1707.13	2.4
1.55	3380	1686.34	4.6	2780	1686.16	4.5
1.77	3180*	1673.47	2.8	2545*	1674.00	1.3
2.09	2900	1655.41	5.5	2320	1655.26	5.2
2.44	2757*	1638.12	2.7	2175*	1638.46	1.7
	BREACH WITH FBH					
0	16455*	-	-	16455*	-	-
0.1	15890	1780.64	8.2	15890	1781.61	5.3
0.18	15570	1774.29	6.8	15570	1775.37	4.4
0.29	15140*	1766.39	5.0	15140*	1767.50	2.8
0.52	14400	1751.09	5.7	14200	1751.76	3.8
0.83	13300*	1730.54	7.5	12830*	1731.42	3.8
1.03	12890	1717.24	4.8	12120	1717.88	3.6
1.21	12500*	1707.66	6.9	11520*	1708.86	3.5
1.55	11580	1686.94	7.0	10540	1687.25	5.7
1.77	11025*	1674.40	3.7	10040*	1675.21	1.9
2.09	10470	1656.56	5.7	9250	1656.50	5.4
2.44	9873*	1639.70	4.0	8560*	1640.30	2.9

*Computed flow - others interpolated
**Mean velocity in deeper channel areas

TABLE 3

DAM BREACH DISCHARGE PROFILES FOR
BREACH AT SOUTHEAST CORNER OF F.R.S.

DISTANCE FROM DAM (MI)	LOW N DISCHARGE (cfs)	LOW N STAGE (ft. MSL)	LOW N VEL** (ft/sec)	HIGH N DISCHARGE (cfs)	HIGH N STAGE (ft. MSL)	HIGH N VEL** (ft/sec)
	BREACH WITH PSH					
0	5302*	-	-	5302*	-	-
0.123	4990	1782.48	4.1	4880	1783.18	2.7
0.303	4666*	1772.89	4.6	4400*	1773.43	3.1
0.559	4230	1758.85	6.3	3870	1759.96	3.4
0.900	3870*	1737.71	4.0	3390*	1738.00	2.9
1.231	3500*	1718.05	4.0	3075*	1718.74	2.3
1.468	3330	1704.70	6.3	2790	1704.81	4.6
1.667	3181*	1694.45	4.1	2598*	1694.98	2.1
1.809	3060	1684.40	4.8	2460	1684.44	3.9
2.036	2863*	1673.55	1.7	2280*	1673.85	1.1
	BREACH WITH FBH					
0	16455*	-	-	16455*	-	-
0.123	15750	1784.05	6.2	15680	1785.17	4.2
0.303	14974*	1773.63	7.9	14892*	1774.61	4.6
0.559	14190	1759.34	4.2	13820	1760.37	2.4
0.900	13493*	1737.87	7.4	12670*	1738.99	3.7
1.231	12835*	1718.29	7.5	11683*	1720.51	3.4
1.468	12280	1706.89	7.5	11100	1706.87	7.0
1.638	12177*	1700.04	4.3	10696*	1700.75	3.5

* Computed flow - others interpolated
** Mean velocity in deeper channel areas

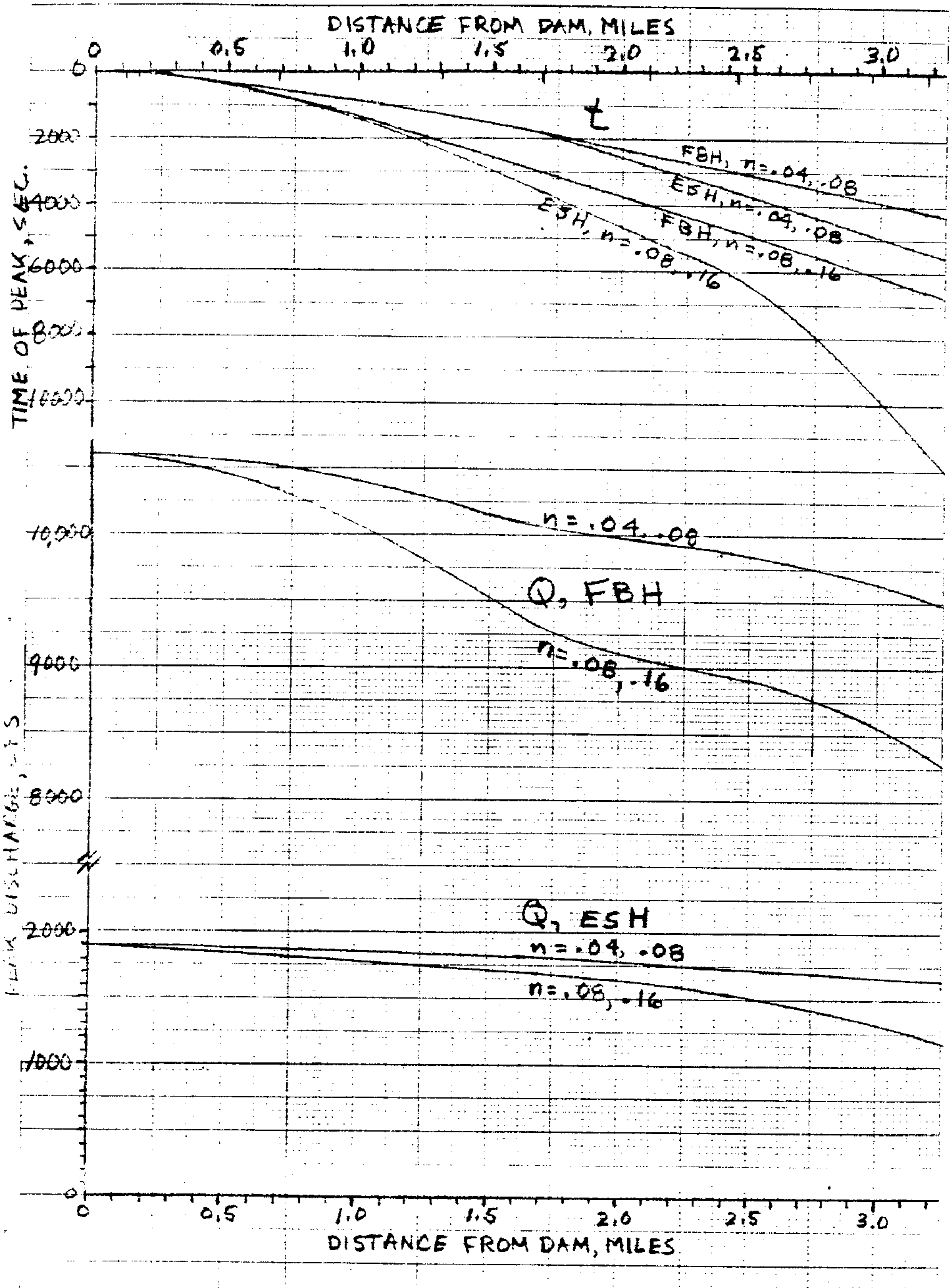

FIGURE 1: SPILLWAY OUTFLOW ROUTING

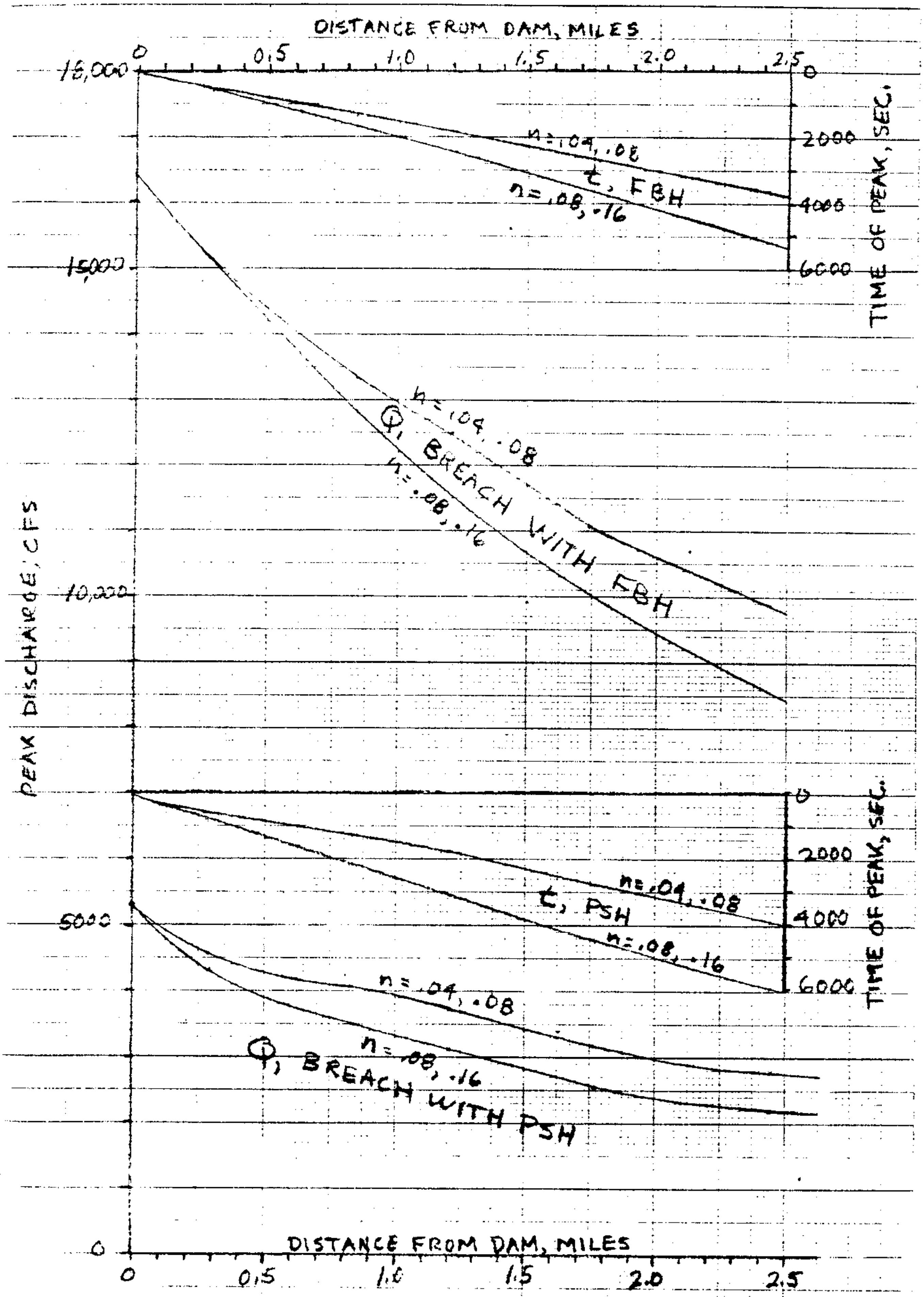

FIGURE 2: IDAHO ROAD BREACH ROUTING

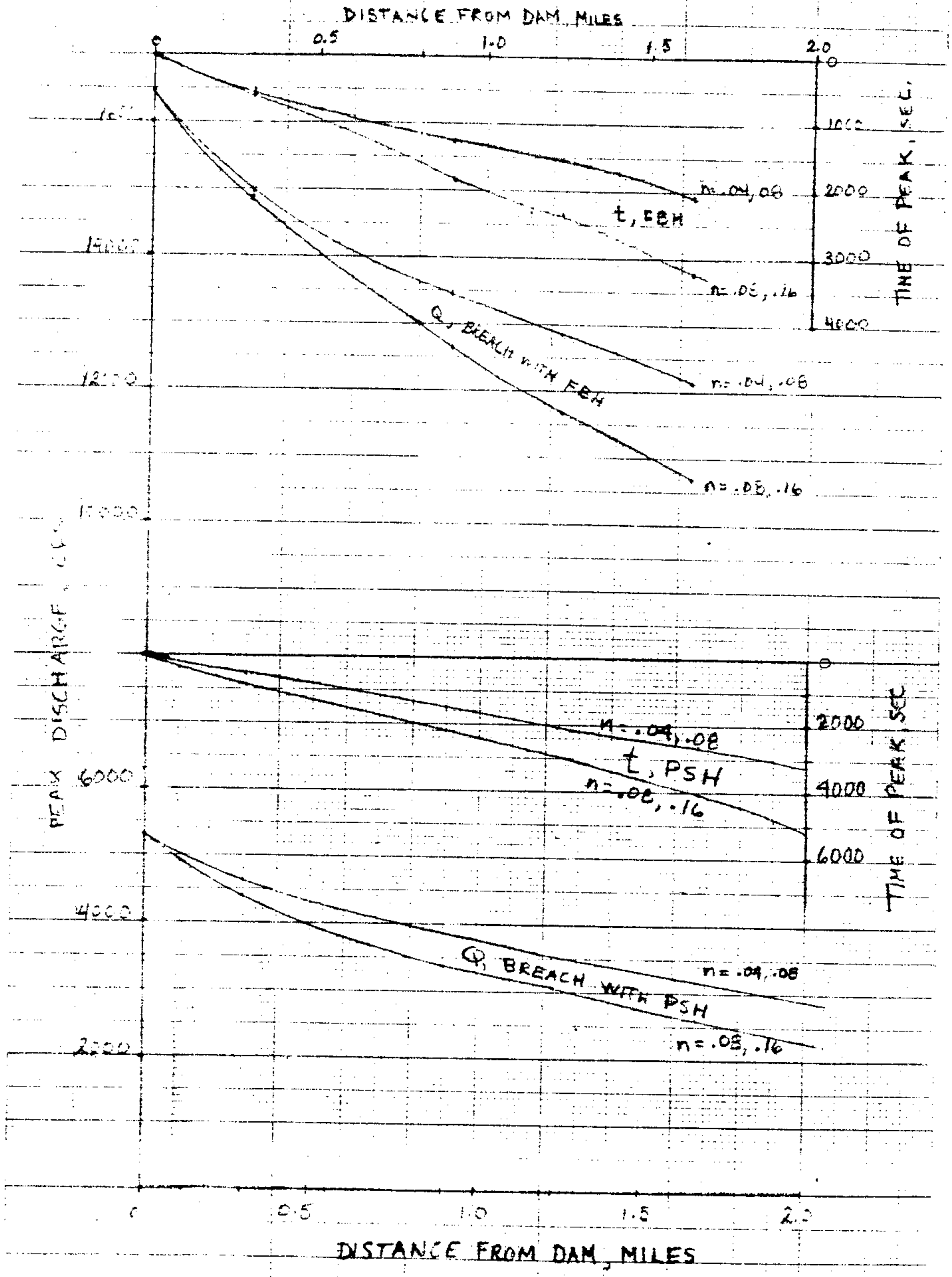

FIGURE 3: SOUTH EAST CORNER BREACH ROUTING

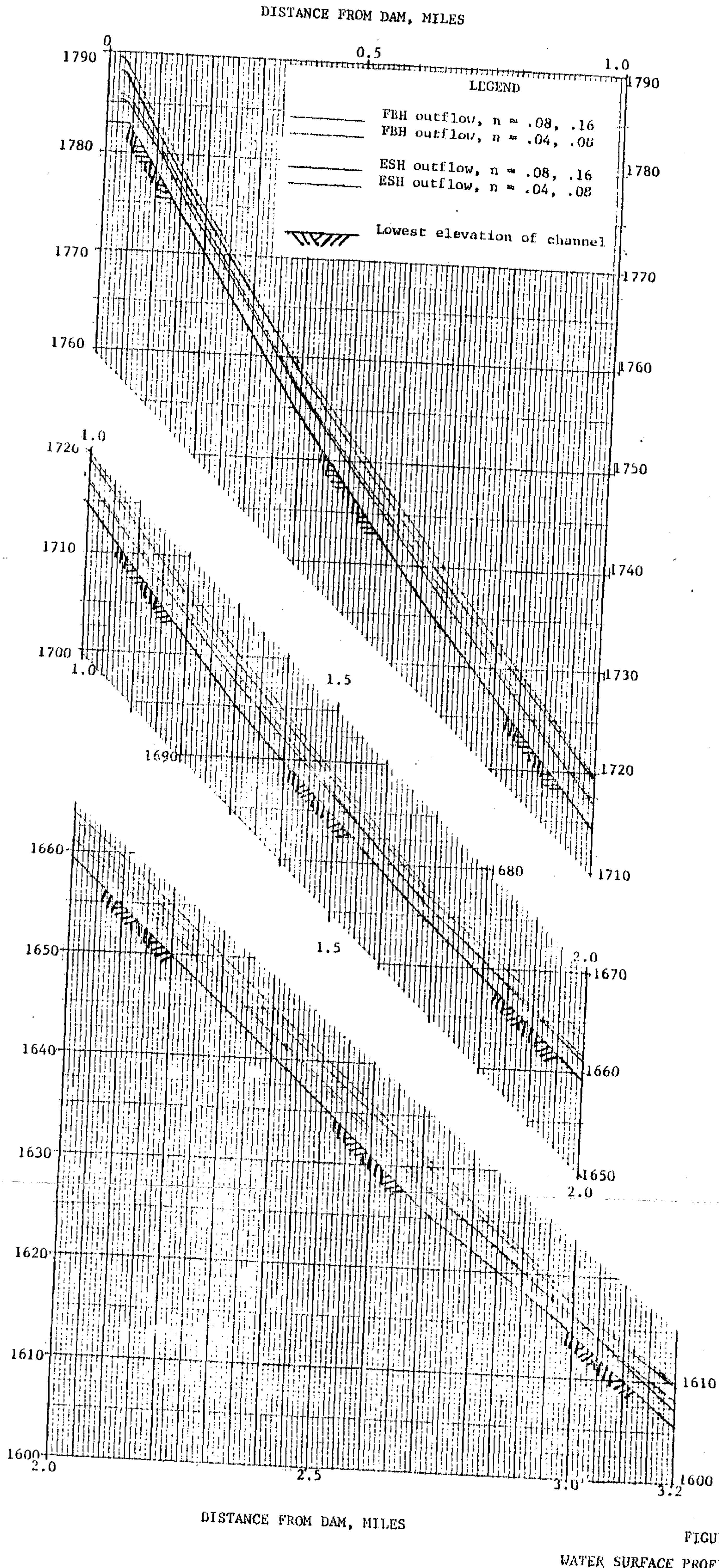

FIGURE 4

WATER SURFACE PROFILES FOR FBH AND ESH SPILLWAY OUTFLOWS

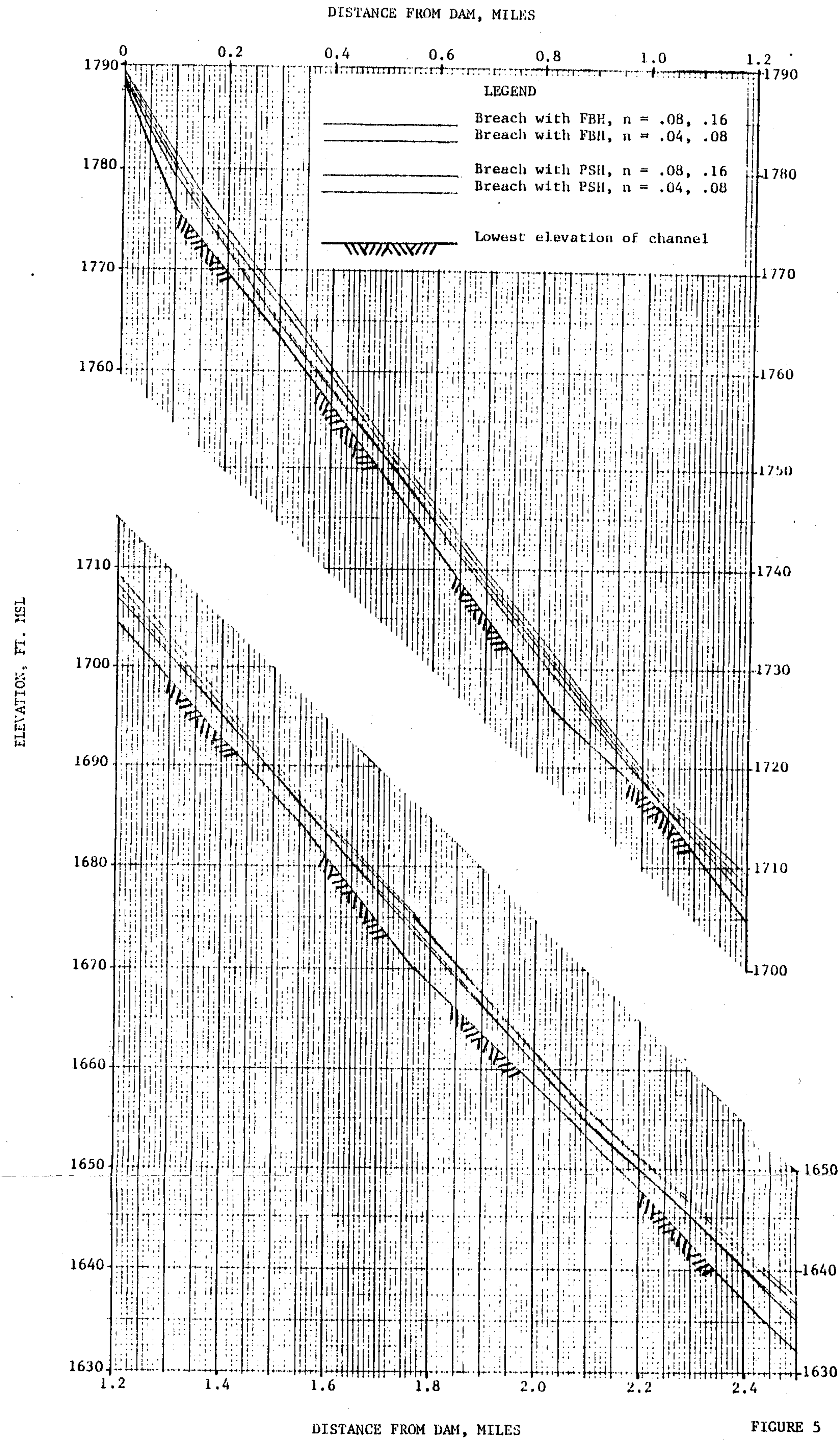

FIGURE 5

WATER SURFACE PROFILES FOR
DAM BREACH NEAR IDAHO ROAD

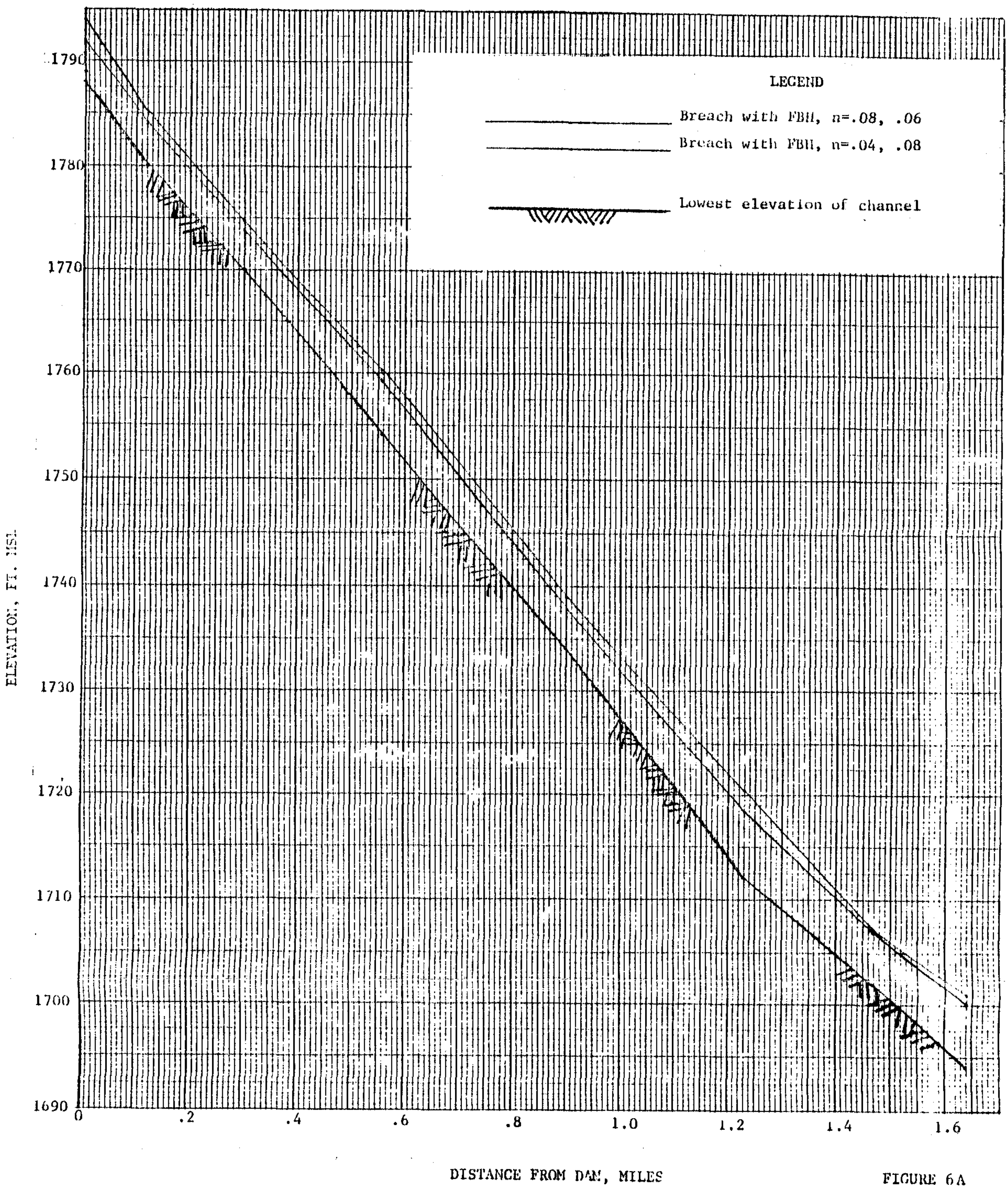

FIGURE 6A

WATER SURFACE PROFILES FOR DAM BREACH AT SOUTHEAST CORNER OF FRS WITH FBH

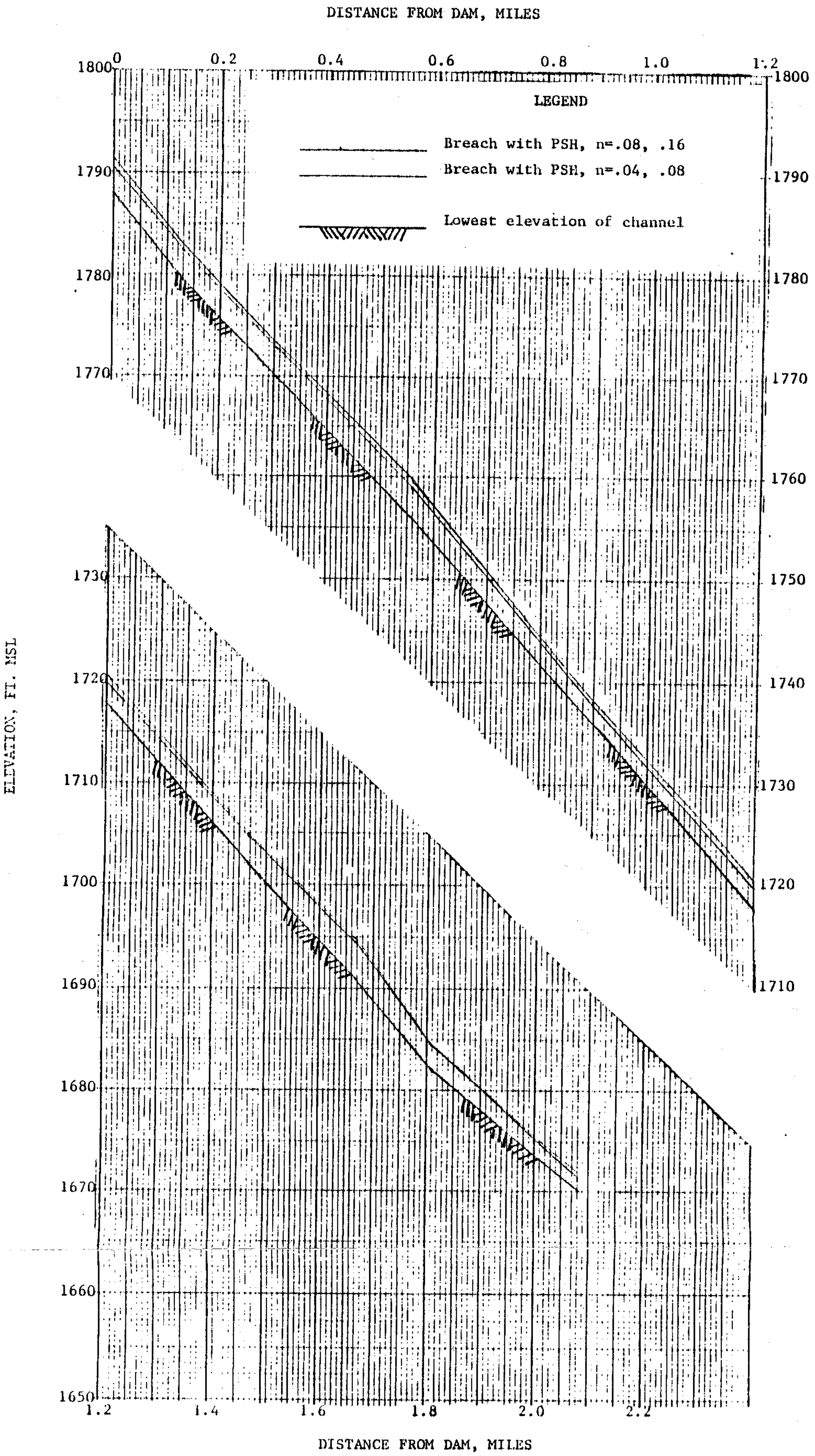

FIGURE 6B
WATER SURFACE PROFILES FOR DAM
BREACH AT SOUTHEAST CORNER OF FRS
WITH PSH

Figure 7
Inundation Map Spillway Outflows
with ESH and FBH

Figure 8
Inundation Map Dam Breach
Near Idaho Road

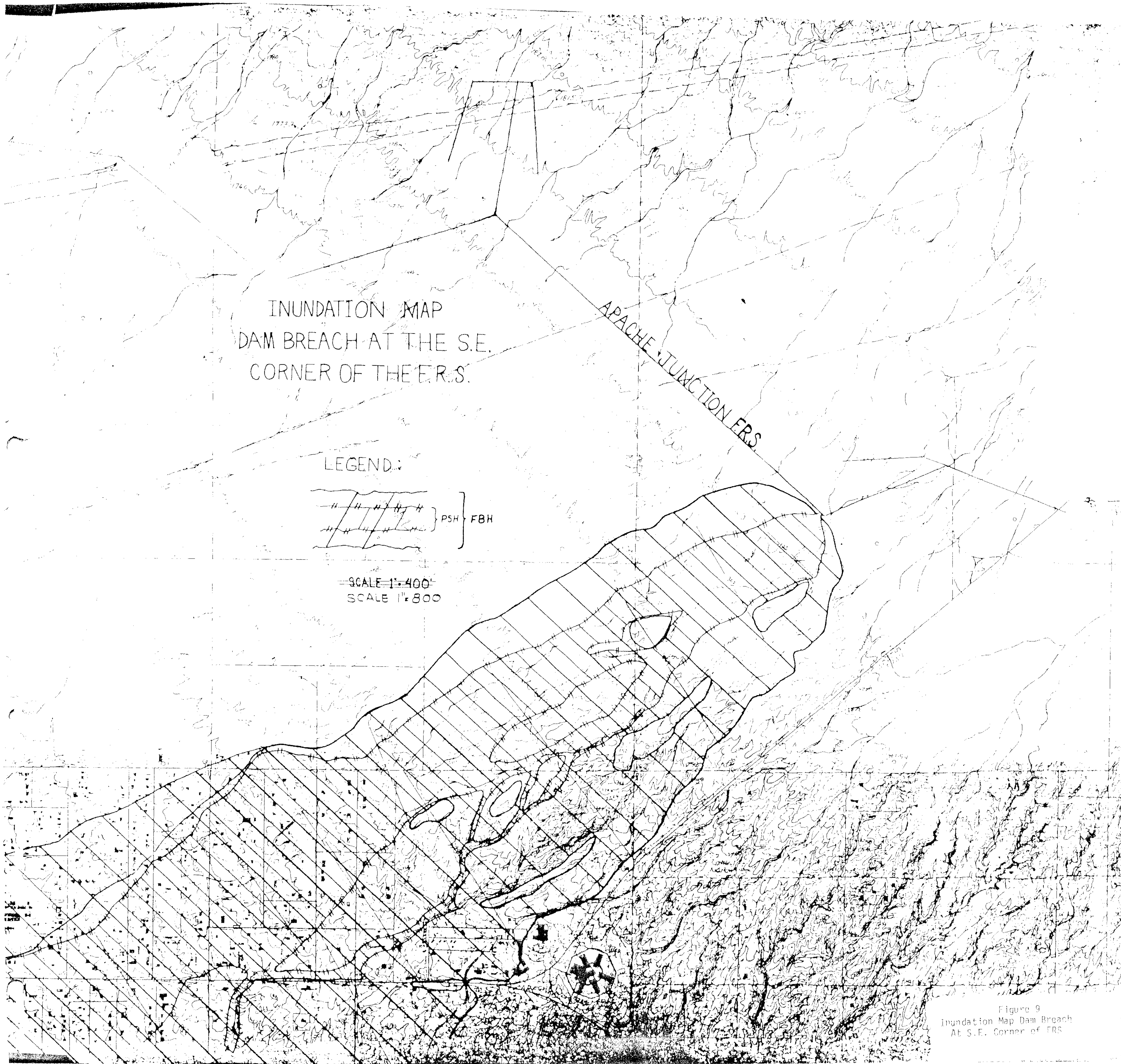

Figure 9
Inundation Map Dam Breach
At S.E. Corner of FRS