cc: C Walling / Dam   XC: 923001- Permits
923001- Corp. Rec.

ARIZONA DEPARTMENT OF WATER RESOURCES

Dam Safety Section
500 North Third Street, Phoenix, Arizona 85004-3903
Telephone (602) 417-2445
Fax (602) 417-2423

Chuck: please let me know what you would like to do with the original gold seal license.
Thanks
Chuck Reece
x 5120

RECEIVED
AUG 27
POWER

August 25, 1997

Mr. Charles S. Reece IV, P.E.
Arizona Electric Power Cooperative, Inc.
Post Office Box 670
Benson, Arizona 85602-0670

Subject: Apache Station Ash/Scrubber Waste Disposal Facility Dam (02.03)
License of Approval

Dear Mr. Reece:

All statutory requirements in connection with the construction of the Apache Station Ash/Scrubber Waste Disposal Facility Dam have now been satisfied. Accordingly, enclosed is a License of Approval to operate the dam. The license outlines the terms and conditions, discussed with you by telephone, under which continued operation of the dam is permitted. This license supersedes any previous operating consent issued by the Department.

During the course of normal operation of your dam, Department engineers will inspect it periodically to confirm that it is being operated and maintained properly. We will contact you in advance of each regularly scheduled inspection to coordinate a mutually satisfactory inspection date. The next regular inspection is currently scheduled for November 1998. In the interim, please contact us immediately should you observe any unusual or alarming circumstances that may adversely affect the safety of the dam.

You may contact Mike Greenslade of our Flood Warning and Dam Safety Section at (602) 417-2400, Extension 7188, if you have any questions. Thank you for your cooperation.

Sincerely,

Dan Roger Lawrence, P.E.
Manager
Dam Safety Section

Enclosure

CC: Donald W. Kimball, AEPCO w/o encl.

State of Arizona
DEPARTMENT OF WATER RESOURCES

LICENSE OF APPROVAL

Pursuant to Title 45 - Waters, Chapter 6, Article 1, of the Arizona Revised Statutes, the DIRECTOR, Department of Water Resources issues this License of Approval to:

ARIZONA ELECTRIC POWER COOPERATIVE, INC.

Authorizing the use of: APACHE STA. ASH/SCRUBBER WDF *Dam and Reservoir, File Number:* 02.03

Located in Section 4 *, Twp.* 16S *, Rge.* 24E *, G. & S.R. B. & M.,* COCHISE *County, State of Arizona, to impound water in accordance with and subject to the following conditions:*

1. The maximum operating water levels shall be: Ash Disposal Pond Cells at elevation 4213.0 ft.; Scrubber Sludge Pond Cells at elevation 4223.0 ft.; Evaporation Pond at elevation 4212.5 ft.
2. An instrumentation monitoring report shall be submitted to ADWR annually in February in accordance with the approved "Embankment Dikes Monitoring Plan", dated December 12, 1996.

This License of Approval supersedes every previous consent for use issued by the State of Arizona relative to said dam and reservoir.

Witness my hand and seal of the Arizona Department of Water Resources

this 25th *day of* AUGUST , 1997

Darrell Jordan

Darrell Jordan
Assistant Director
Surface Water Management Division

Arizona Electric Power Cooperative, Inc.

P.O. Box 670 • Benson, Arizona 85602-0670 • Phone 520-586-3631

May 27, 1997

Mr. William C. Jenkins, P.E.
Arizona Department of Water Resources
500 N. 3rd St.
Phoenix, AZ 85004

**RE: COMBUSTION WASTE DISPOSAL FACILITY W.O. 923001/933148
PROJECT NO. 91-033-1-010
CONTRACT 923001-2 - POND AND APPURTENANCES CONSTRUCTION
INDIVIDUAL POND WAVE HEIGHT, SETUP, & RUNUP CALCULATIONS**

Dear Mr. Jenkins:

In accordance with your Mr. Gerald Cox's request, enclosed are the subject matter. These are in support of AEPCO's request to allow a maximum reservoir height of three feet below the dike crest (4213' for ash disposal & 4223' for scrubber sludge disposal ponds) for all disposal ponds, with exception of the evaporation pond. Please review these calculations at your earliest convenience and let me know at (520)586-5120 should you have any questions or need additional information concerning the issuance of an Approval to Operate for the Facility.

Sincerely,

CSReece IV

Charles S. Reece IV, P.E.
Mechanical Engineer

enc

xc C. Davis, w/o enc
L. Huff, w/o enc
rt G. Grim, w/enc
C. Walling, w/enc
File 923001-ADWR, w/enc

S:\POWER\GENENGR\CHUCK\923001\ADWR\WAVEHGT.WP5

Significant wave height

$$Z_w = 0.034\ V_w^{1.06}\ F^{0.47}$$ ②

where Z_w = average height of highest 1/3 wave

V_w = wind velocity (miles/hr) = 55 mph ①

F = fetch (miles)

Z_w Table		Fetch (ft)	Fetch (miles)	Z_w (ft)
Ash Pond	1	1,560	0.295	1.34
	2	1,570	0.297	1.34
	3	1,700	0.322	1.40
	4	1,870	0.354	1.46
Scrubber Pond	1	1,880	0.356	1.46
	2	1,880	0.356	1.46

Add wind tide to wave height calculations
for maximum wave height and runup
See page 2 for wind tide calcs.

originals to
file, copies
to ADWR

Wind tide or set-up calculation

$$S = \frac{U^2 F}{1{,}400\, D} \quad ③$$

where S = wind tide (feet)
U = avg. wind velocity (mph)
F = fetch (miles)
D = avg depth along fetch line (feet)

(see previous pages for wind vel. & Fetch)

S Table		Avg Depth Along Fetch (ft)	Wind Tide (feet)
Ash Pond	1	27	0.02
	2	27	0.02
	3	27	0.03
	4	27	0.03
Scrubber Pond	1	11	0.07
	2	13	0.06

Add wind tide height to wave height calculation

Calculation of wave period & wave length

$$T_w = 0.46\, V_w^{0.44}\, F^{0.28}$$

where T_w = wave period

$$\lambda = 5.12\, T_w^2$$

where λ = wave length (feet)

Calc table

	T_w	λ	Z_w/λ
Ash Pond 1	1.91	18.7	0.07
2	1.91	18.7	0.07
3	1.95	19.5	0.07
4	2.01	20.6	0.07
Scrubber Pond 1	2.01	20.6	0.07
2	2.01	20.6	0.07

Embankment slope = $1/3$ = 0.333

From Figure 7.13 (2)

Relative Run up $\frac{Z_r}{Z_w} = 1.48$

thus Z_r = wave runup (vert) (ft)

$Z_r = 1.48\,(Z_w + S)$ (see next page for S)

Z_r table	Z_w (ft)	S (ft)	Z_r (ft)	IDF (ft)	Pond Height Below Crest (IDF + Z_r) (ft)
Ash Pond 1	1.34	0.02	2.01	0.8	2.81
2	1.34	0.02	2.01	0.8	2.81
3	1.40	0.03	2.12	0.8	2.92
4	1.46	0.03	2.21	0.8	3.01
Scrubber Pond 1	1.46	0.07	2.26	0.8	3.06
2	1.46	0.06	2.25	0.8	3.05

Note: IDF is maximum inflow design flood which is ½ of predicted maximum precipitation or 0.8 ft

From the previous calculations, it may be concluded that a maximum pond level of three feet below the crest will not produce waves which overtop the dike. Maximum pond heights would be 4213' (4216-3) and 4223' (4226-3) for ash and scrubber sludge disposal ponds, respectively.
(see B&M Design Notes & Analysis for Comb. Waste Disposal Facility for Evaporation pond wave runup calculations)

References:

1. B&M Design Notes & Analysis for ... Ponds
2. Water Resources Engineering
3. Hand book of Applied Hydraulics

Burns & McDonnell
ENGINEERS - ARCHITECTS - CONSULTANTS
Kansas City, Missouri

Client AEPCO

Project No. 91-033-1-010 Date 3/20/92

7.2 WAVE RUNUP

WAVE RUNUP CALCS

Made By MKF

Checked By KSC/TLL

Preliminary ______ Final ______

ASSUMPTIONS: - 55 MPH DESIGN WIND SPEED
- FETCH LENGTH ≈ 2800 L.F. = 0.53 MILES
- PREVAILING WIND DIRECTION = SOUTHEAST TO NORTHWEST
- DESIGN SOURCE "WATER-RESOURCES ENGINEERI

SIGNIFICANT WAVE HEIGHT

$Z_w = 0.034 V_w^{1.06} F^{0.47}$ V_w = WIND VELOCITY
F = FETCH (IN MILES)
$= 0.034(55)^{1.06}(0.53)^{0.47}$ Z_w = AVG HEIGHT OF HIGHEST ⅓
$= 1.76$ FT ✓ = SIGNIFICANT WAVE HEIGHT

MINIMUM TIME DURATION OF WIND VELOCITY = 10 MIN

RELATIVE RUN-UP

Wave Period → $T_w = 0.46 V_w^{0.44} F^{0.28}$
$= 0.46(55)^{0.44}(0.53)^{0.28}$
$= 2.25$ ✓

WAVE LENGTH → $\lambda = 5.12 T_w^2$
$= 5.12(2.25)^2$
$= 25.92$ ✓

VALUE OF $\frac{Z_w}{\lambda} = \frac{1.76}{25.92} = 0.068 \approx 0.07$ ✓

EMBANKMENT SLOPE $= \frac{1}{3} = 0.333$ ✓

FROM FIGURE 7-13:
RELATIVE RUN-UP $\frac{Z_r}{Z_w} \approx 1.48$

VERTICAL HEIGHT OF RUNUP $= Z_r = 1.48(1.76)$
$= \underline{2.6'}$ ✓

LENGTH OF RUNUP ALONG SLOPE $= 2.6(3)$
$= \underline{7.8'}$ ✓

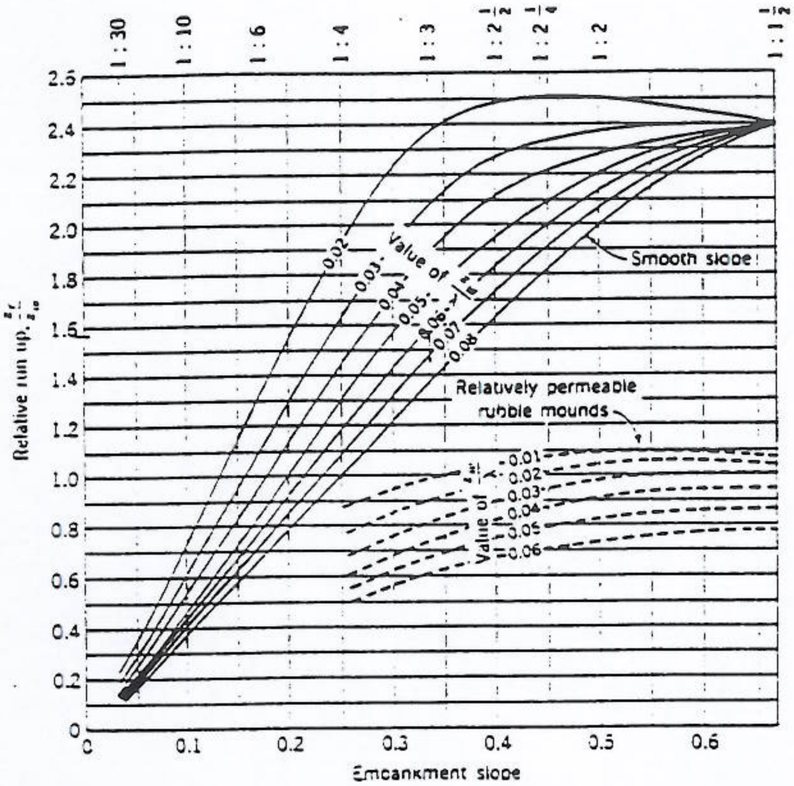

FIG. 7-13. Wave run-up ratios versus wave steepness and embankment slopes. *(From Saville, McClendon, and Cochran.)*

rubble mounds. Height of run-up z_r is shown as a ratio z_r/z_w and is dependent on the ratio of wave height to wavelength (wave steepness). Wavelength λ may be computed from

$$\lambda = 5.12 t_w^2 \tag{7-4}$$

where the wave period t_w is given by

$$t_w = 0.46 V_w^{0.44} F^{0.28} \tag{7-5}$$

TABLE 7-2. Per Cent of Waves Exceeding Various Wave Heights Greater than z_w

(After Saville, McClendon, and Cochran)

z'/z_w	1.67	1.40	1.27	1.12	1.07	1.02
Per cent of waves > z'	0.4	2	4	8	10	12

OTHER McGRAW-HILL HANDBOOKS OF INTEREST

HANDBOOK OF APPLIED HYDRAULICS

CALVIN VICTOR DAVIS *Editor-in-Chief*

Chief Technical Advisor, Harza Engineering Company, Chicago

KENNETH E. SORENSEN *Co-Editor*

Vice President, Harza Engineering Company, Chicago

THIRD EDITION

McGRAW-HILL BOOK COMPANY

New York St. Louis San Francisco London Sydney
Toronto Mexico Panama

tabular methods, etc.). Thus a variety of sequences of monthly and annual inflows may be considered, the sum total of which gives an evaluation of the probabilities involved.

There is an inherent psychological advantage to the stochastic approach, since the results must be expressed in terms of probabilities. This discourages a false sense of security that the design drought period will not be exceeded in severity—a feeling which can easily be obtained when the basis of design is the most critical period of record.

The stochastic approach is under rapid development. Although the basic philosophy, theories, and some procedural matters have been worked out, much remains to be done in the development of specific procedures before the approach can be considered to be fully operational. The classic work in this field was performed by Hurst,[1] followed by highly significant work by Langbein[2] and Fiering.[3]

To utilize the stochastic approach, it is necessary to develop several statistical parameters. These include the values of mean flow, measures of the variability of individual occurrences from the mean, and correlations between flows during previous periods and the flow during following periods. Although techniques for these analyses are undergoing rapid change, reference by the reader to the writings of the pioneer investigators referred to will provide basic background on the details of procedures that may be used.

After the statistical parameters are developed, they may be used to select random values of streamflow. This is the equivalent of "pulling historically observed streamflows from a hat," with proper constraints being applied to assure that unrealistic results will not be obtained. These constraints include such factors as the limitation on minimum flow that will be randomly selected (obviously it cannot be less than zero and usually it will be more) and the effect of the runoff from a previous period on the otherwise randomly selected runoff of a following period.

In using the stochastic approach it is necessary to decide on the number of years of record that should be generated and analyzed. While the number of years required to produce a result which has a certain probability of accuracy is undoubtedly subject to mathematical analysis, procedures have not yet been developed for this determination. Some investigators have arbitrarily utilized a 100-year generated record, but in some instances comparison of two separately generated 100-year periods has resulted in substantially different storage requirements.

Whatever the period of record that may be generated, it can be done most expeditiously and economically by use of a digital computer. Programs are available for this purpose, and these usually can be combined expeditiously with programs for analysis of the performance of the reservoir in meeting a variety of storage and delivery requirements.

The use of stochastic procedures, through development of a stochastic model, should not be interpreted as ignoring of historical data. Historical data are used to develop means, parameters of variability of flow, and relationships between successive flow periods. The only element of historical occurrences which is given small importance is the sequence of flows. This, however, is highly desirable since the historical sequence represents only one of many possibilities of sequence. The stochastic approach attempts, therefore, to put the historical sequence into proper perspective as to its probability of recurrence.

[1] Hurst, H. E., Long Term Storage Capacity of Reservoirs, *Trans. ASCE*, 1951, pp. 770–808.
[2] Langbein, W. B., Queuing Theory and Water Storage, *Proc. ASCE*, Paper 1811, October, 1958.
Langbein, W. B., and N. C. Matalas, Information Content of the Mean, *J. Geophysical Res.*, August, 1962.
[3] Fiering, Myron B., Queuing Theory and Simulation in Reservoir Design, *Trans. ASCE*, 1962, vol. 1, pp. 1114–1144.

15. Application of Stochastic Approach. The use of the stochastic approach can be demonstrated by studies for a river in the Philippines. In that basin there were no runoff or precipitation data available for a large portion of the 1941–1949 period, with appreciable periods of record both before and after that time. Accordingly, the normal procedure for bridging of the gap in data by correlating runoff with precipitation was not possible.

The storage reservoir proposed for this basin is of the holdover type, with several years of drawdown between reservoir fillings. Because of this, it was essential that the sequence of runoffs for consecutive years be representative. The assumption that the record prior to 1941 would be continuous with the record starting in 1949 would be open to serious question. Accordingly, it was concluded that the best procedure would be to consider water supply on a stochastic basis.

In the stochastic analysis new sequences of historical flows were generated, utilizing the historical flow data as the basis. Normal distribution of the historical values about the mean was assumed in this analysis, except for certain controls that were placed on the sequence of monthly events. This control was developed by plotting successive monthly flows, one against the other, to develop serial correlations of successive monthly flows.

The generation of monthly streamflows, as well as operation studies necessary to estimate the dependable outflow, was by use of the digital computer. The drawdown was determined for four assumed controlled outflows by the digital computer, for several 100-year series of generated streamflow, and the resulting drawdowns were averaged. The results, expressed in terms of 0.1 percent, 1.0 percent, and 5.0 percent probabilities of occurrence, are shown in Fig. 8. These curves furnished a basis for selection of the probabilities of a shortage in water supply that could be tolerated in design of the reservoir.

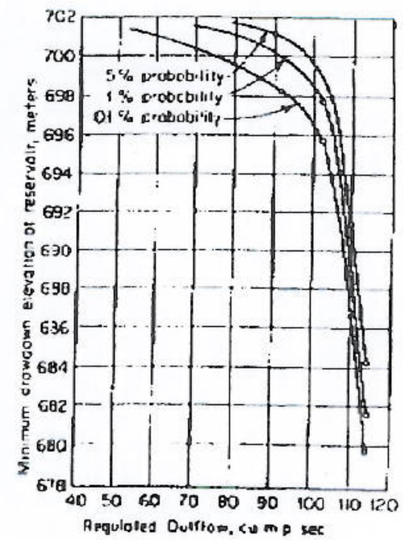

Fig. 8. Reservoir drawdown in critical period.

RESERVOIR WAVE ACTION

16. Freeboard Allowances. The term "freeboard" is frequently used in different ways. As defined previously, freeboard must include consideration of the following:

1. Height of wind tide (referred to also as setup)
2. Height of waves in deep water generated by winds
3. Effect of wave run-up on sloping embankments on height of waves
4. Any additional margin of safety considered necessary

Final design decisions on freeboard allowances usually involve consideration of the type of dam, the situation governing the spillway design flood, and the effect of waves. This section is concerned only with wave action. An excellent article by Saville, McClendon, and Cochran[1] and a manual by the U.S. Corps of Engineers[2] form the basis of procedures given in this section for computing waves.

17. Basic Assumptions. A number of formulas have been developed for computation of heights of wind tide, waves, and run-up. Most of these involve use of wind velocity and fetch as basic parameters. While the different formulas will yield different results, the variation between formulas is frequently not so great as the variation possible in results that are due to assumptions as to wind velocity and fetch. Thus, the development of reasonable assumptions is of prime importance.

Wind. The magnitude of wind tide, wave height, and run-up will vary with the magnitude of wind velocity and the duration of that velocity. Thus, it is desirable to develop wind data on a duration basis, if possible. The proper combination of velocity and duration is not always subject to precise determination, although procedures are available in the computation of wave heights for determining the minimum required time to reach maximum wave heights. Frequently, maximum wave conditions will not result unless the duration is in the order of 1 hr. Therefore, in wind-tide calculations the maximum observed 60-min average wind is frequently taken as the first trial for design. This assumption then can be checked against the values of wind tides derived as described later.

Care should be taken to utilize only those wind conditions considered possible of occurrence at the same time or immediately following the meteorological conditions causing the pool level under consideration. For example, if the spillway design storm is not of the hurricane type, the winds used to compute freeboard allowances should not be of the hurricane type.

Fetch. Fetch length is the horizontal distance of open water surface over which the wind blows. The use of the greatest straight-line distance over open water in wave computations will result in computed wave heights that are too high, since the amount of adjoining open water having shorter but significant fetches influences the waves. Observations on artificial reservoirs have indicated that use of an "effective fetch" is more reliable. The effective fetch is computed by dividing the 45° angle on either side of the maximum fetch line into about 15 equal segments, multiplying the fetch length for each segment by the cosine of the angle of deviation from the maximum fetch line, and dividing the sum of the products by the sum of the cosines.

Wind velocities over water are generally higher than over land under comparable meteorological conditions, because of lesser roughness. The following values represent averages observed on artificial reservoirs:

Fetch, miles	0.5	1.0	2.0	3.0	4.0	5.0 (or over)
Wind ratio $\frac{\text{over water}}{\text{over land}}$	1.08	1.13	1.21	1.26	1.28	1.30

The maximum potential wind velocity may not always coincide in direction with the direction of maximum fetch. If observations of maximum winds of given directions are available, the use of the effective fetch length can be carried one step further, utilizing the appropriate design wind velocities with the fetches indicated.

18. Wind Tide. Wind tide, or "setup," is the piling up of water at the leeward end of an enclosed body of water, as a result of the horizontal stress on water exerted by the wind. The magnitude of wind tide can be expressed by the following modification of the Zuider Zee formula:

$$S = \frac{U^2F}{1{,}400D} \qquad (3)$$

in which S is the wind tide (in feet) above still water, U is average wind velocity (in statute miles per hour) over the fetch distance F (in miles), and D is the average depth of water along the fetch line (in feet).

19. Wave Height and Other Characteristics. Wind-generated waves in a large body of water are not uniform in height. Successive waves will not be identical—each wave will be preceded and succeeded by a higher or lower wave. Data obtained from recordings of 45 storm periods at Fort Peck and Denison reservoirs have shown a very close comparison between the observed frequency distribution of wave heights on inland reservoirs with observed data on oceans. The following characteristics have been observed for the spectrum of waves observed at a given time and place:

% of total number of waves averaged to compute specific wave height H	Ratio of H to average wave height H_{av}	Ratio of H to significant wave height H_s	% of waves exceeding H
1	2.66	1.67	0.4
5	2.24	1.40	2
10	2.03	1.27	4
33⅓	1.60	1.00	13
50	1.42	0.89	20
100	1.00	0.62	46

The significant wave height H_s is defined as the average height of the highest one-third of all waves in a spectrum. As will be seen from the above tabulation, 13 percent of all waves can be expected to exceed H_s. These values would be reached at the end of a buildup period and give measures of the variations that can be expected in wave-height distributions. H_s may be computed by the set of curves in Fig. 9. Knowing the effective fetch and the wind velocity, the curve can be entered with these values to give the minimum time duration and value of H_s.

Once the value of H_s is computed, the occurrence frequency of a wave of any height can be computed from the preceding tabulation. The design height for waves H can be selected on the basis of consideration of frequency of winds of a given magnitude, duration of winds, and frequency of waves of given size. The finally selected design height must be a judgment value, involving consideration of the type of dam involved, as well.

20. Wave Run-up on Slopes. A wind-generated wave will be influenced when it runs up the slope of an embankment. The effect may be either to increase or to decrease the height of the wave in relation to the still-water surface, depending on wave characteristics and the slope, roughness, and permeability of the embankment. Therefore, the effect of run-up is usually combined with the actual wave height in computing allowances for wave action, into a single item designated as wave run-up height R.

In this sense R is the vertical distance between the maximum elevation obtained by a wave running up an embankment and the water elevation at the toe of the slope. The water elevation at the toe of the slope is the still-water elevation plus wind tide. Because of the relationship between wave height and run-up, it usually is convenient to compute run-up as a function of wave height.

[1] Saville, Thorndike J., Elmo W. McClendon, and Albert L. Cochran, Freeboard Allowances for Waves in Inland Reservoirs, Trans. ASCE, 1963, pt. IV, pp. 195–226.
[2] Waves in Inland Reservoirs, Tech. Mem. 132, Beach Erosion Board, U.S. Corps of Engineers, November, 1962.

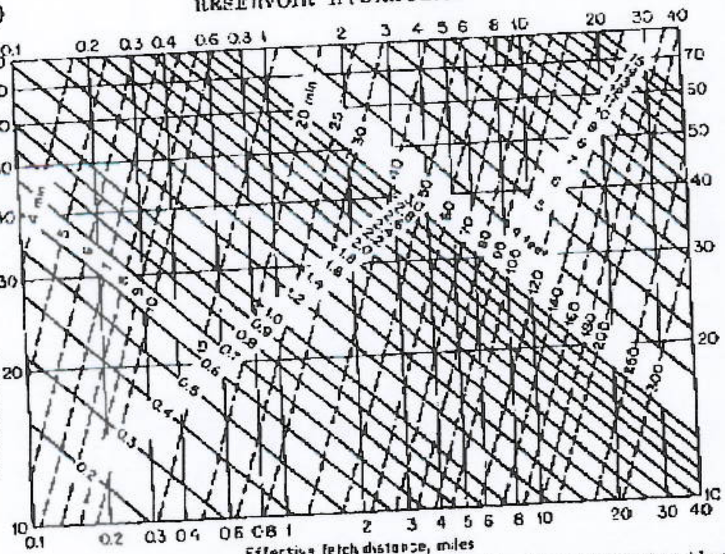

FIG. 9. Generalized correlations of significant wave heights H_s with related factors (deep-water conditions). Solid lines represent significant wave heights, in feet; dashed lines represent minimum wind duration, in minutes, required for generation of wave heights indicated for corresponding wind velocities and fetch distance. (*Beach Erosion Board, U.S. Corps of Engineers, Tech. Mem.* 132.)

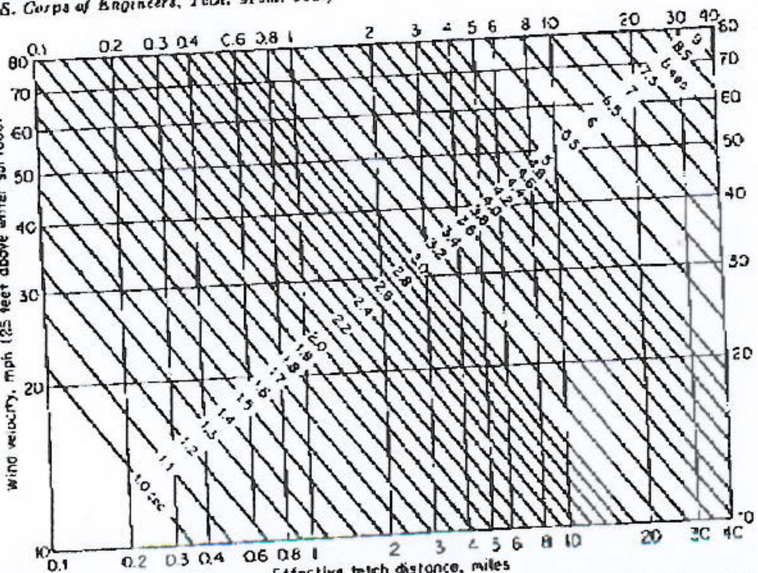

FIG. 10. Generalized relations between wave periods (T) and related factors (deep-water conditions). (*Beach Erosion Board, U.S. Corps of Engineers, Tech. Mem.* 132.)

The wave characteristics are represented by the wave steepness ratio H_o/L_o, where H_o = specific wave height and L_o = wave length, measured from crest to crest, in deep water. H_s may, for practical purposes in deep reservoirs, be taken as equal to H. L_o may be computed from the following formula:

$$L_o = 5.12T^2 \tag{4}$$

where T is the wave period, which may be determined from Fig. 10. This wave period is approximately the same for waves ranging between the significant wave H_s and

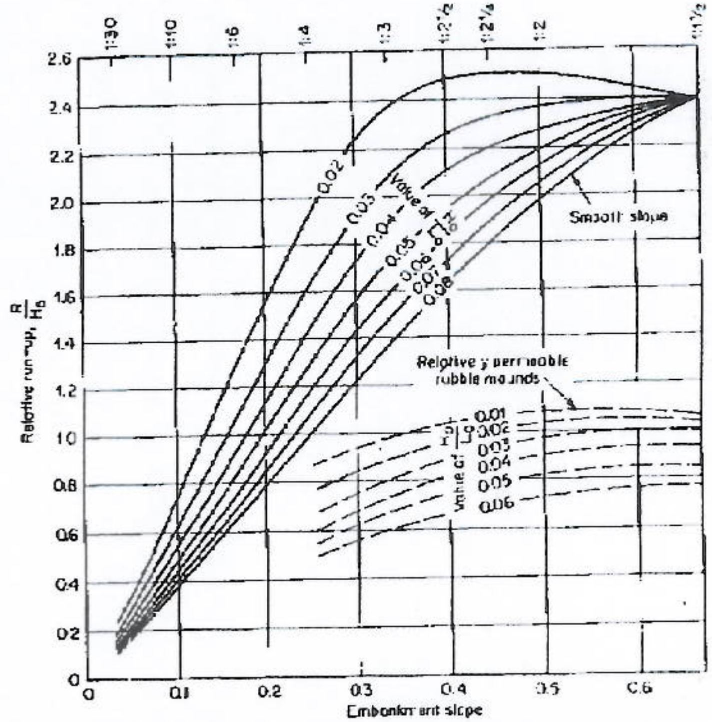

FIG. 11. Wave run-up ratios vs. wave steepness and embankment slopes. (*Trans. ASCE*, vol. 128, 317. 1963.)

the maximum wave H_{max}. Thus, in deep water the L_o determined for the wave steepness ratio can be used for any value of H between H_{max} and H_s. Deep-water conditions can be considered to be present when the depth at the toe of the slope is more than one-third of the calculated wave length.

Using the values of H_o and L_o, the effect of run-up on wave height may be computed from Fig. 11. Curves are shown for smooth slopes and for rubble mounds. Smooth slopes include surfaces such as well-graded earth embankments covered with sod and asphalt or concrete facing. Run-up on hand-placed riprap slopes approaches that computed for smooth slopes. Run-up on dumped riprap slopes can be considered to be about 50 percent of computed run-up on smooth slopes.

6.0 FREEBOARD CAPACITY FOR A 0.5 PMP STORM EVENT

The probable maximum precipitation (PMP) depth for this area in Arizona is approximately 11.2 inches based on information in the National Oceanic & Atmospheric Administration (NOAA) Hydrometeorological Report No. 49. The 0.5 PMP would be approximately 5.6 inches. All of the ponds are designed to exclude offsite runoff from entering and to maintain a minimum freeboard of 3 feet. Therefore, if the 0.5 PMP were to occur, there exists adequate freeboard to prevent discharge over the dike crest.