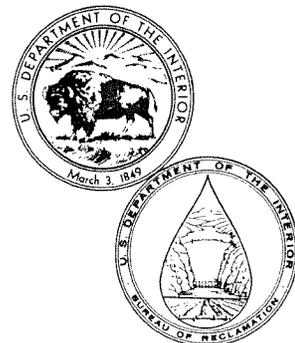


ACER TECHNICAL MEMORANDUM NO. 2
ASSISTANT COMMISSIONER - ENGINEERING AND RESEARCH
DENVER, COLORADO

FREEBOARD CRITERIA AND GUIDELINES FOR COMPUTING FREEBOARD ALLOWANCES FOR STORAGE DAMS

U.S. DEPARTMENT OF THE INTERIOR
Bureau of Reclamation
1981



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Assistant Commissioner - Engineering and Research
Denver, Colorado

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UNITED STATES DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

December 1981

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PREFACE

This document presents Bureau of Reclamation policy pertaining to freeboard allowances for storage dams. Freeboard is needed to provide assurance against overtopping of dams and represents a contingency allowance for a number of variables that cannot be fully assessed during the design process.

Part I contains freeboard criteria which have been developed for Bureau of Reclamation storage dams. Part II contains guidelines which provide suggested methods for computing freeboard requirements in compliance with Reclamation's criteria. Site-specific factors stated in the criteria which should be considered in the determination of total freeboard requirements for each project are discussed and example computations of wind setup and wave runup are presented. The appendix contains pertinent definitions that are used in freeboard determinations.



Rodney J. Vissia
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CONTENTS

	<u>Page</u>
Preface	iii
I. Freeboard Criteria for Storage Dams	
A. Introduction	1
B. General Criteria	1
C. Criteria for Existing Dams	7
II. Guidelines for Computing Freeboard Allowances for Storage Dams	
A. Wind Setup and Wave Runup	9
1. Limited Scope Studies	9
2. Detailed Studies	10
3. Examples of Computations of Wind-generated Wave Runup and Setup	29
B. Hydrologic Uncertainty	53
C. Earthquake- and Landslide-generated Water Waves	53
D. Settlement of Embankment and Foundation	55
E. Malfunction of Spillway and Outlet Works	57
F. References	59
Appendix - Definitions	61

TABLES

Table

1. Freeboard requirements for preliminary studies	9
2. Wind relationship - Water to land	12
3. Choke Canyon Reservoir - Computation of effective fetch	31
4. Over water wind computations - Choke Canyon Reservoir - Normal freeboard	31

TABLES - Continued

	<u>Page</u>
5. Wind velocity and duration data points - Choke Canyon Reservoir - Normal freeboard	32
6. Over water wind computations - Choke Canyon Reservoir - Minimum freeboard	37
7. Wind velocity and duration data points - Choke Canyon Reservoir - Minimum freeboard	37
8. Pueblo Reservoir - Computation of effective fetch	40
9. Pueblo Memorial Airport wind data	41
10. Over water wind computations - Pueblo Reservoir - Normal freeboard	44
11. Wind velocity and duration data points - Pueblo Reservoir - Normal freeboard	47
12. Over water wind computations - Pueblo Reservoir - Minimum freeboard	50
13. Wind velocity and duration data points - Pueblo Reservoir - Minimum freeboard	51

FIGURES

Figure

1. Fastest mile of record wind - Winter	16
2. Fastest mile of record wind - Spring	17
3. Fastest mile of record wind - Summer	18
4. Fastest mile of record wind - Fall	19
5. Maximum 1-hour wind velocity - Winter	20
6. Maximum 1-hour wind velocity - Spring	21
7. Maximum 1-hour wind velocity - Summer	22
8. Maximum 1-hour wind velocity - Fall	23

FIGURES - Continued

	<u>Page</u>
9. Generalized correlations of significant wave heights with related factors	24
10. Generalized relations between wave periods and related factors	25
11. Comparison of wave runup on smooth slopes with runup on permeable rubble slopes	26
12. Wave runup on smooth, impermeable slopes	27
13. Runup correction for scale effects	28
14. Computation of effective fetch length - Choke Canyon Reservoir	33
15. Plot of over water wind velocity versus duration - Choke Canyon Reservoir	35
16. Wind exceedance frequency - Pueblo Memorial Airport	43
17. Computation of effective fetch length - Pueblo Reservoir	45
18. Plot of over water wind velocity versus duration - Pueblo Reservoir	48

I. FREEBOARD CRITERIA FOR STORAGE DAMS

A. Introduction

The freeboard for a dam is the vertical distance between a specified reservoir water surface elevation and the top of the dam, without camber. Normal freeboard is defined as the difference in elevation between the top of the dam without camber and the higher of the top of conservation storage or top of joint-use storage as established from design requirements. Where there is exclusive flood control storage, refer to intermediate freeboard criteria. Minimum freeboard is defined as the difference in elevation between the top of the dam without camber and the maximum reservoir water surface that would result from routing the IDF (inflow design flood) through the reservoir. Routing assumptions for establishing maximum pool should conform to established Bureau criteria.

The objective of having freeboard is to provide needed assurance against overtopping resulting from:

- Wind setup and wave runup
- Landslide and seismic motion
- Settlement
- Malfunction of structures
- Other uncertainties in design, construction, and operation

B. General Criteria

To accomplish the previously stated objective, use the following criteria in determining freeboard allowances for storage dams.

1. These site specific factors should be considered:

- Flood characteristics such as the shape of the IDF hydrograph, peak discharge, volume, and duration and design considerations used in selecting the IDF.
- Wind characteristics such as velocity, duration, orientation, seasonal distribution, and the probability of the joint occurrence of maximum wind and the IDF.
- Topographic configuration of the reservoir.
- Effective fetch length.
- Dam characteristics such as type of dam (embankment or concrete), controlled or uncontrolled spillway, slope and protective cover on the upstream face of an embankment dam, erosion resistance of crest and downstream face, and exposure to freezing and thawing.
- Dam and foundation consolidation.
- Earthquake potential, magnitude, and effect on the dam.
- Landslide potential and consequences.
- Malfunction of spillway and outlet works, mechanical equipment, electrical equipment, automatic controls, and/or power source, and potential for plugging, lack of necessary attendance, etc.
- Shape of the spillway rating curve.

- Downstream hazard resulting from overtopping or dam failure.
- Access to dam.

2. Consideration of the preceding factors will yield components which should be included in the computation of freeboard requirements. These components are:

- Wind-generated wave height, setup, and runup
- Earthquake- and landslide-generated wave height and runup
- Settlement of embankment and foundation
- Hydrologic uncertainty
- Malfunction of spillway and outlet works
- Allowance for other site specific uncertainties

3. Both normal and minimum freeboard requirements should be evaluated in determining the elevation of the top of dam. The freeboard resulting in the higher crest elevation should be adopted for design.

4. It is highly unlikely that maximum winds will occur when the reservoir water surface is at its maximum elevation resulting from routing the IDF. Computations of wind-generated wave height, setup, and runup should incorporate the probability of combined occurrences of pool level, wind, and appropriate durations; these should be used in determining normal and minimum freeboard allowances. There are extreme cases when reservoir outflow capacity is low; however, even in those

cases, maximum water surface elevations persist only for relatively short periods of time.

5. The best wind data available that are applicable to the site should be used for computations of wind-generated wave height, setup, and runup; the sum of wind setup and wave runup should be used for determining requirements for this component of freeboard.

6. Freeboard allowance for settlement should be applied to account for consolidation of foundation and embankment materials when computational methods do not yield highly reliable values for camber design. Freeboard allowance for settlement should not be applied where an accurate determination of settlement can be made and is included in the camber.

7. Potential earthquake-generated movement and resulting seiches and permanent embankment displacements should be considered if a dam is located in an area with high potential for seismic activity.

8. Wave and volume displacement due to potential landslides which cannot be economically removed or stabilized should be considered if a reservoir is located in a topographic setting so the wave or higher water resulting from displacement may be destructive to the dam or may cause serious downstream damage.

9. The probability that some combinations of the aforesaid components will occur simultaneously is extremely low. Maximization of each component and adding them together to determine total freeboard requirements should be avoided. Only those components which can reasonably occur simultaneously for a particular water surface elevation should be

combined. Components of freeboard and combinations of these components which have a reasonable probability of simultaneous occurrence are listed in the following sections for determining minimum and normal freeboards. In special cases, combinations of components for reservoir water surfaces between conservation and maximum levels may need to be evaluated to determine which is the most critical. The crest of the dam should be established to accommodate the most critical combination of water surface and freeboard components deemed reasonable.

- Minimum freeboard combinations. - The following components when they can reasonably occur simultaneously, should be combined to determine the total minimum freeboard requirement:

- a. Wind-generated wave runup and setup for a moderate wind.
- b. Malfunction of spillway and/or outlet works during the IDF.
- c. Settlement of embankment and foundation not included in crest camber.
- d. Hydrologic uncertainties resulting from inadequate data base.
- e. Landslide-generated water waves and/or displacement of reservoir volume (only cases where landslides are triggered by the occurrence of higher water elevations and intense precipitation associated with the occurrence of the IDF).

● Normal freeboard combinations. - The most critical of the following two combinations of components should be used for determining normal freeboard requirements:

a. (1) Wind-generated wave runup and setup for maximum wind.

(2) Settlement of embankment and foundation not included in crest camber.

b. (1) Landslide-generated water waves and/or displacement of reservoir volume.

(2) Settlement of embankment and foundation not included in crest camber.

(3) Settlement of embankment and foundation from maximum credible earthquake.

● Intermediate freeboard combinations. - A reasonable combination of components should be determined on a case-by-case basis by the designer. This would apply to cases where there are exclusive flood control storage allocations.

10. For concrete dams, zero minimum freeboard is acceptable for the top of dam in most cases when a standard 3.5-foot-high solid parapet wall is constructed.

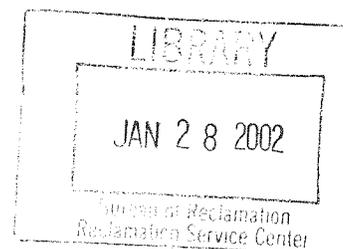
11. For embankment dams, the minimum freeboard should not be less than 3 feet. (Special consideration should be given to embankment dams with gated spillways; refer to section II-D.)

12. Use of parapet walls to provide freeboard allowances for earth dams may be considered on a case-by-case basis. However, the following safeguards must be met:

- a. The maximum water surface resulting from routing the IDF must not exceed the top of the impervious zone.
- b. The parapet wall may only replace the portion of the freeboard needed to prevent overtopping from wave runup.
- c. Future foundation and embankment settlement that would adversely affect the structural integrity of the parapet wall should be allowed to occur prior to construction of the wall or the wall design should allow for future settlement.

C. Criteria for Existing Dams

The general freeboard criteria should be applied to existing as well as proposed dams taking into account conditions that have changed since the initial freeboard design determination. For example, settlement of the embankment and landslides, with the exception of that due to seismic shaking, would probably have occurred and may not need to be considered for existing dams. Because foundation and embankment settlement is likely to have occurred, the addition of a parapet wall may be a feasible method of providing freeboard in some cases. Additionally, the risk of malfunction of spillways and outlet works should be better known than at the time of original design because of maintenance and operating experience. When assessing the risk of malfunction, known limitations to gate operation



should be considered as well as improvements in mechanical and electrical features or added provisions for skilled attendance during periods of operation. While 3 feet of freeboard has been established as the minimum criterion for proposed embankment dams, an evaluation of conditions at existing dams may indicate that some encroachment is acceptable.

II. GUIDELINES FOR COMPUTING FREEBOARD ALLOWANCES FOR STORAGE DAMS

A. Wind Setup and Wave Runup

The following computational procedures for wind-generated wave runup and setup should be used to satisfy Reclamation's freeboard criteria for storage dams.

1. Limited Scope Studies

The empirical method given in the Design of Small Dams [1, 2]*, as presented in the following table, can be used for appraisal and, in some cases, feasibility design of riprapped embankment dams where detailed wind data are not available and where funding for a preliminary design may be limited.

Table 1. - Freeboard requirements for preliminary studies

Longest fetch, miles	Normal freeboard, feet	Minimum freeboard, feet
Less than 1	4	3
1	5	4
2.5	6	5
5	8	6
10	10	7

These values were based on a wind velocity of 100 miles per hour for determining normal freeboard and 50 miles per hour for minimum freeboard. The effect of wind setup is not considered in the values shown in table 1. For embankment dams with soil cement or other smooth

* Bracketed numbers identify references listed in section II-F.

upstream faces, depending on the smoothness of the surface, the values shown in table 1 should be multiplied by a factor of up to 1.5.

2. Detailed Studies

For specifications designs and some feasibility designs, computational procedures such as those presented in the U.S. Army Corps of Engineers ETL (Engineering Technical Letter) 1110-2-221 [3] and modified in this document should be followed in determining freeboard requirements.

Wind-generated wave runup and setup are sensitive to site-specific conditions at the reservoir such as the seasonal magnitude and direction of winds concurrent with given water levels, orientation of the central radial of the effective fetch, influence of topography, shape of the shoreline and other factors. Detailed analysis of wind data for meteorologic stations located near the damsite should be used in the computations when such data are available. Transposition of wind data from meteorologic stations to the specific reservoir site should reflect the influence of changed topography and ground cover on wind direction and velocity. Designers should fully utilize detailed site-specific wind data that can be obtained from Bureau technical specialists, the National Weather Service, or other sources for use in these computations. Either detailed or generalized wind data can be used with these procedures; however, using detailed wind data will provide more reliable results.

The computations are based on significant wave height which is the average of the highest one-third of the waves in the wave spectrum. A step-by-step computational procedure follows:

a. In inland waters, fetches are limited by land forms surrounding the body of water. Shorelines are irregular and a general method must be applied. The effective fetch at a given station can be computed by [3,4,5]:

$$F_e = \frac{\sum X_i \cos \alpha_i}{\sum \cos \alpha_i} \quad \text{[Equation 1]}$$

in which α_i = angle between the central radial and radial i
and X_i = length of projection of radial i on the central radial.

A trial-and-error method should be used in the selection of a station along the dam for determining the direction of the central radial to obtain a maximum effective fetch for a given reservoir shoreline configuration. The radials spanning 45° on each side of the central radial should be used in computing the effective fetch.

b. The generalized fastest mile (approximate 1-minute duration) and 1-hour winds can be estimated from figures 1-4 and 5-8, respectively, based on the location of the reservoir and the season of storm occurrence if detailed wind data are not available. Note previous emphasis on obtaining detailed site-specific wind data. The 2-hour wind velocity can be estimated by multiplying a factor of 0.96 times the 1-hour wind velocity determined from figures 5 through 8. These over land wind velocities have been adjusted to 25 feet above ground level. An adjustment for over water winds can be estimated by multiplying the over land winds by the velocity ratios given in table 2.

Table 2. - Wind relationship - Water to land

Effective fetch (F_e) in miles*	0.5	1	2	3	4	5 (or more)
Wind velocity ratio $\frac{\text{(Over water)}}{\text{(Over Land)}}$	1.08	1.13	1.21	1.26	1.28	1.30

*For effective fetch values between those listed, interpolate to obtain the most appropriate multiplier.

c. The relationship between wind velocity over water and wind duration at a given reservoir can be developed from figure 9 by entering the effective fetch computed in step a.

d. The intersection of the wind velocity-duration curves for given sites and fetches developed from steps b and c will determine the design wind velocity and duration.

e. The significant wave height can be estimated from figure 9 and the wave period from figure 10 by entering the design wind velocity determined in step d and the effective fetch.

f. The deep water wave length in feet can be computed by

$$L = 5.12 T^2 \quad \text{[Equation 2]}$$

in which T = the wave period in seconds from step e.

Most dams have relatively deep reservoirs compared to the wind-generated wave length and the wave is unaffected by the reservoir floor. Equation 2 is valid when the water is deeper than one half of the wave length. If reservoir depth becomes a limiting factor,

adjustments to L can be made by following procedures defined in reference 6, volume III.

g. Runup, from a significant wave, on an embankment with riprap surface is:

$$R_S = \frac{H_S}{0.4 + (H_S/L)^{0.5} \cot \theta} \quad \text{[Equation 3]}$$

in which H_S = significant wave height in feet,

L = wave length in feet from step f,

θ = angle of the upstream face of the dam with horizon.

This equation should not be used on slopes flatter than 0.2:1.

For embankment dams with soil cement or other smooth upstream faces, the runup computed by equation 3 should be multiplied by a factor of up to 1.5, depending on the smoothness of the surface.

Equation 3 should not be used for computing runup for rockfill dams. Rockfill acts more like a rubble mound structure and has a different effect on energy dissipation than riprap placed on an impervious embankment. Runup for rockfill dams may be determined from figure 11. These data were taken from figure 7-20 of reference 6, volume II.

For smooth impermeable slopes of concrete and other smooth surface dams with water depth at the dam (d_S) greater than three times the wave height (H_0), the relationship between wave runup and wave

height can be determined from figure 12. These data were taken from figure 7-12 of reference 6.

Results predicted by figure 12 are probably less than the runup on prototype structures because of scale effects due to the inability to scale roughness effects into small-scale laboratory tests. Runup values from figure 12 should be adjusted for scale effects by using a factor obtained from figure 13.

If the wave propagation direction as defined by the central radial is not normal to the dam, a correction factor should be applied to the computed runup. This factor consists of multiplying the cosine of the angle between the wave propagation direction and a line normal to the dam times the computed runup as long as the angle is less than about 50° [7].

h. The wind setup in feet is:

$$S = \frac{U^2 F}{1400 D} \quad \text{[Equation 4]}$$

in which U = the design wind velocity over water in miles per hour from step d.

F = wind fetch in miles, normally equals 2 F_e

D = average water depth along the central radial in feet.

i. The minimum freeboard requirement for wind-generated waves is the sum of wave runup and wind setup and should be determined using moderate winds. These represent winds in terms of velocity, duration, direction, and seasonal distribution that may reasonably occur concurrently with maximum pool levels. If the response time between the design storm and the resulting maximum pool elevation is short, high winds that are sometimes associated with storms may not have subsided and must be considered in determining freeboard requirements. If the response time is longer than the storm period, a lower, more moderate wind would be appropriate. If adequate data for probability analyses are available, a moderate wind on the order of a 10-year event is appropriate.

The normal freeboard computation follows the same procedures as that for the minimum freeboard except that the significant wave height in equation 3 should be replaced by the average of the highest 10 percent of the waves which is 1.27 times the significant wave height [5,8] and maximum expected wind values should be used. These represent the most severe winds in terms of velocity, duration, direction, and seasonal distribution that are reasonably characteristic of the region where the reservoir is located. This determination generally includes results of meteorologic studies and probability analyses of recorded wind data. The values selected should exceed 100-year winds determined by probability analyses and generally should exceed maximum recorded winds.

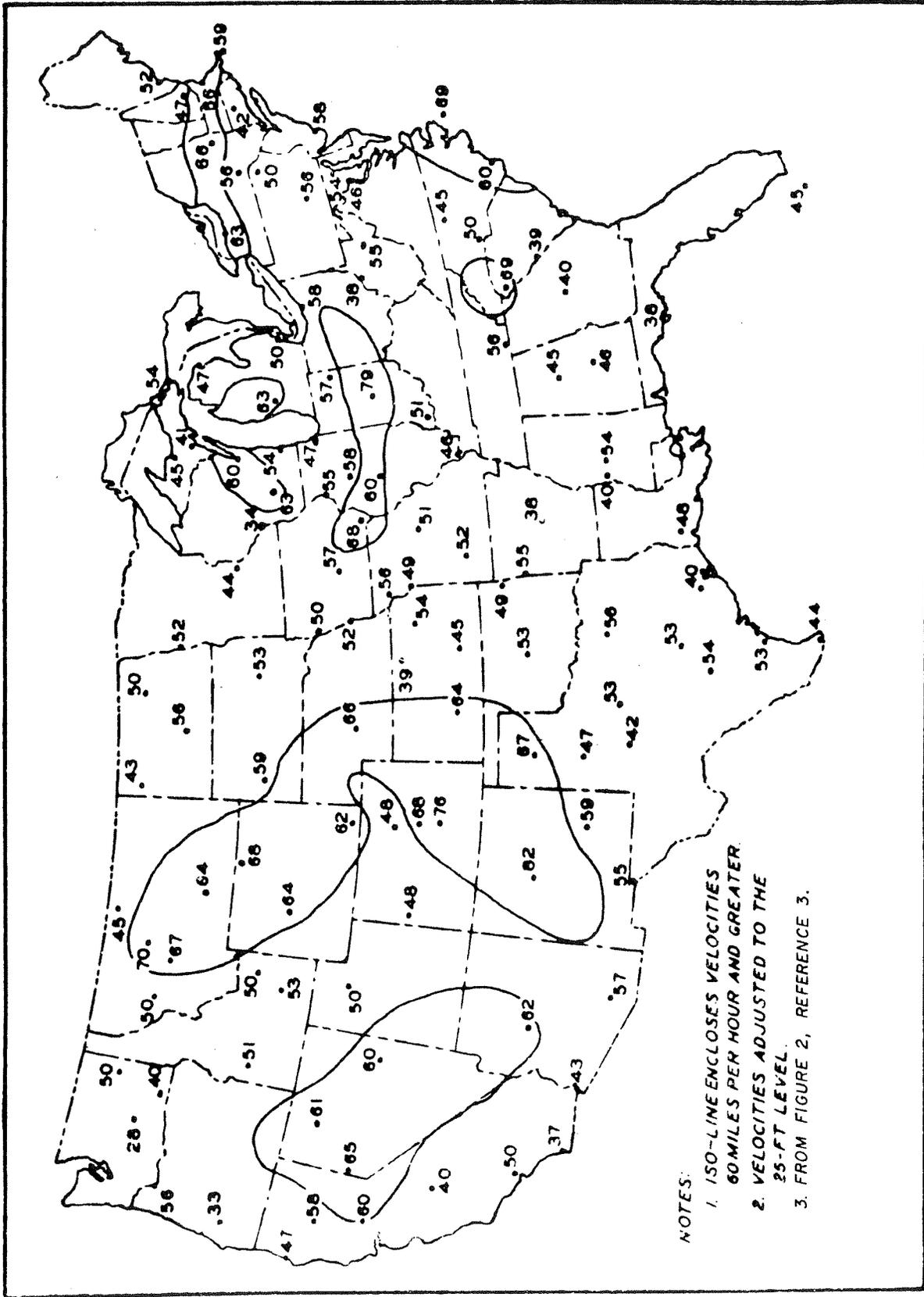


FIGURE 1.-FASTEST MILE OF RECORD - WINTER

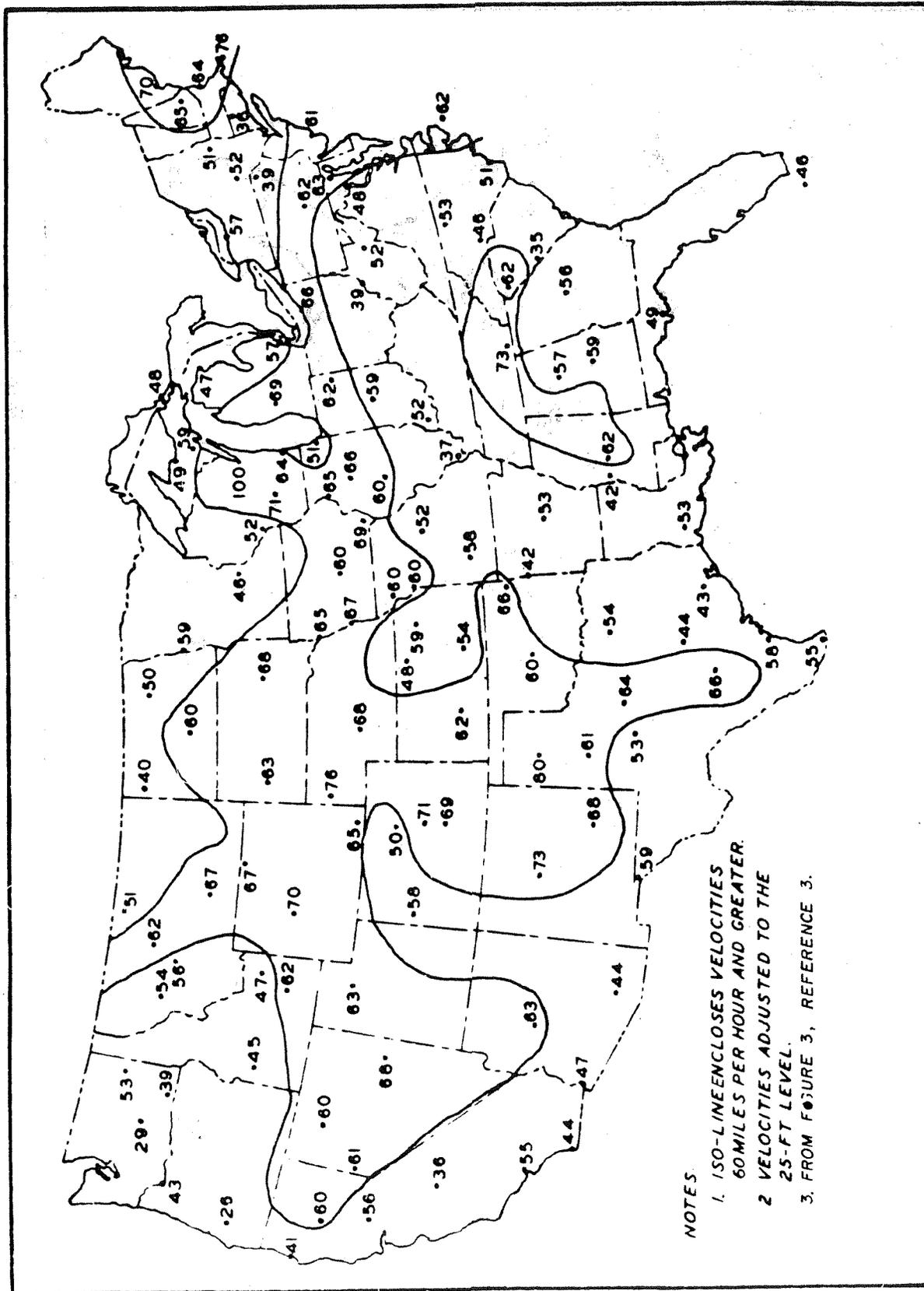


FIGURE 2.-FASTEST MILE OF RECORD - SPRING

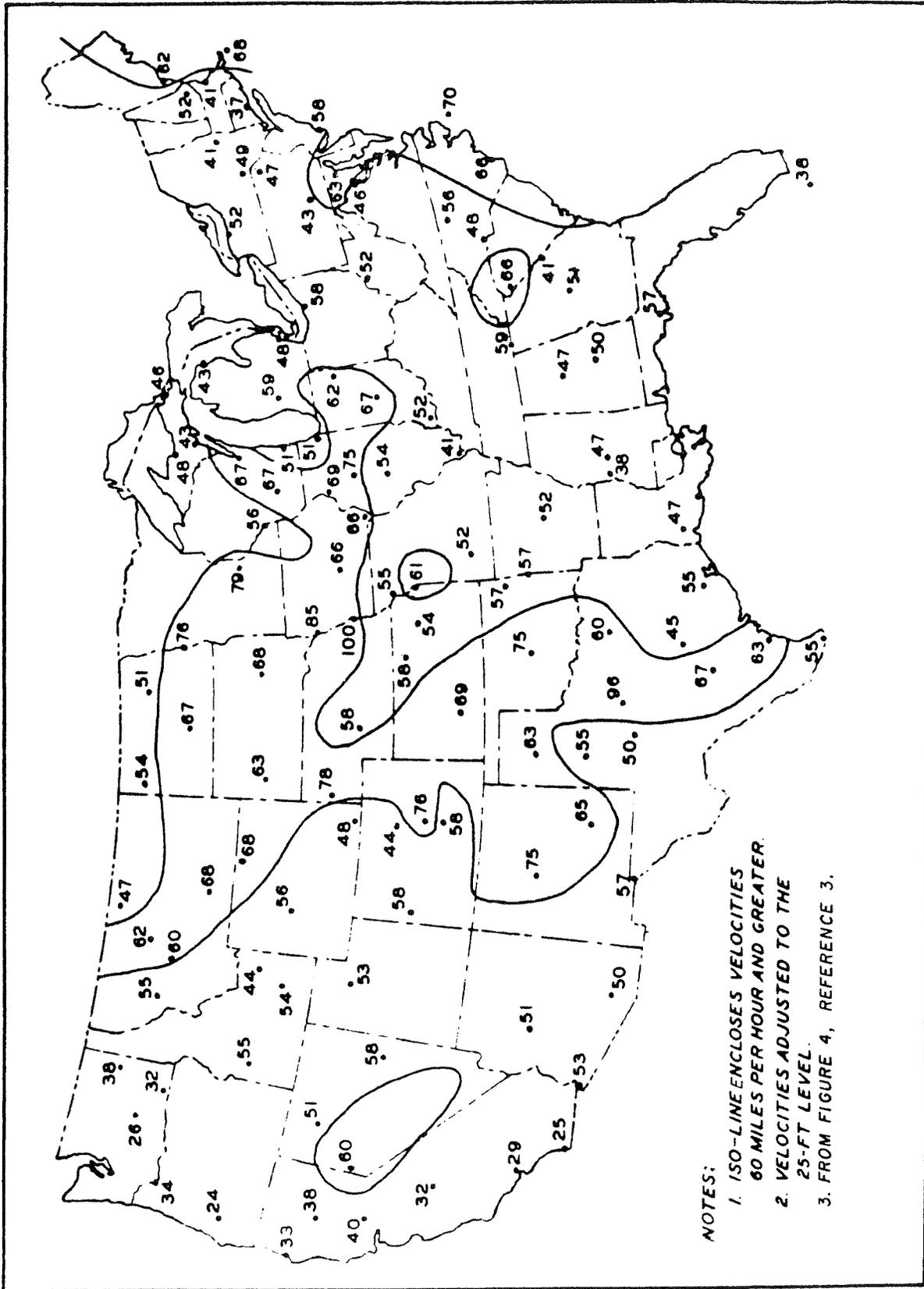


FIGURE 3.-FASTEST MILE OF RECORD - SUMMER

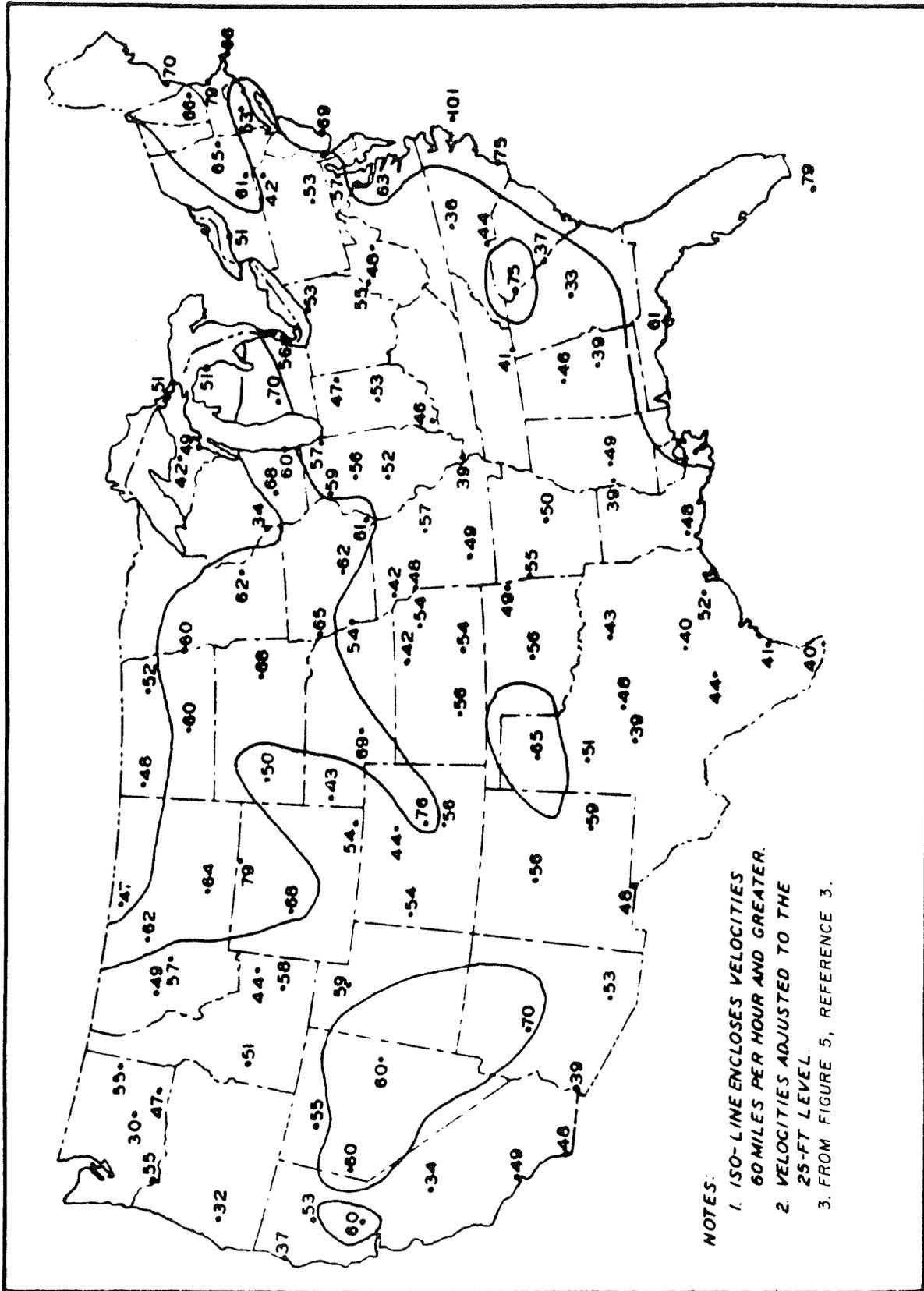


FIGURE 4.-FASTEST MILE OF RECORD - FALL

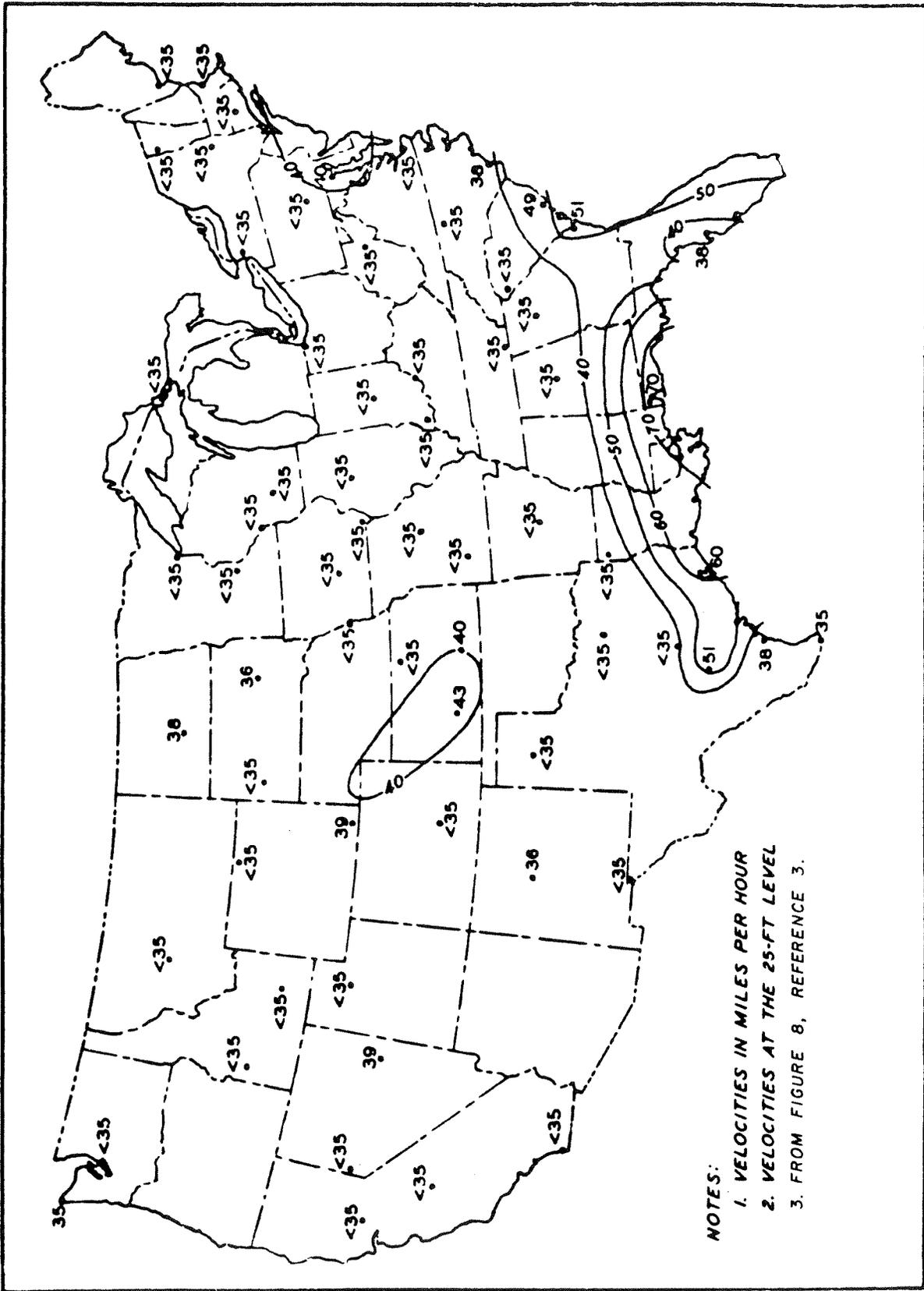
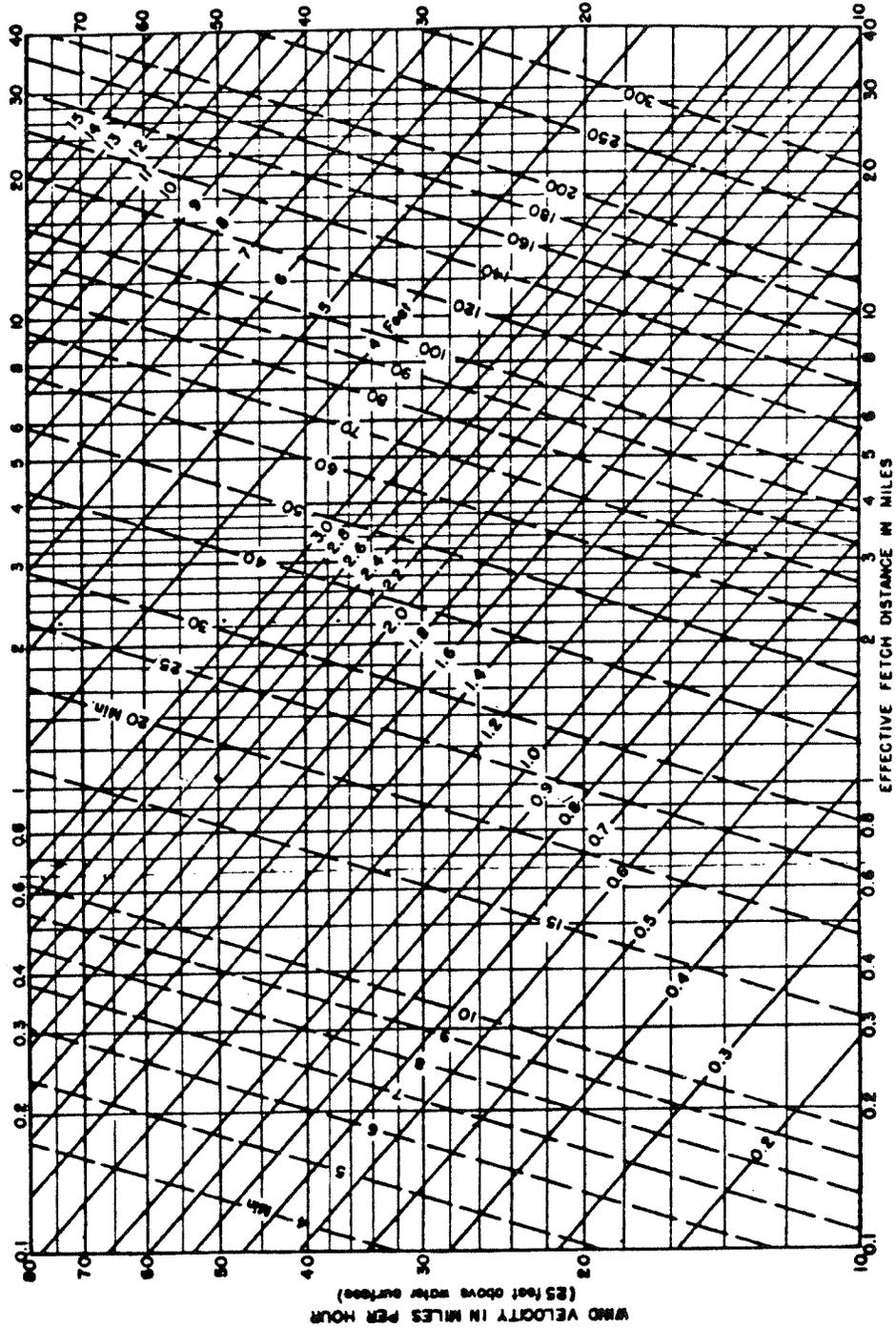


FIGURE 7.-MAXIMUM ONE HOUR VELOCITY - SUMMER

LEGEND:

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.



**FIGURE 9. - GENERALIZED CORRELATIONS OF SIGNIFICANT WAVE HEIGHTS (H_s) WITH RELATED FACTORS
-DEEP WATER CONDITIONS (FROM FIGURE 11, REF. 3)**

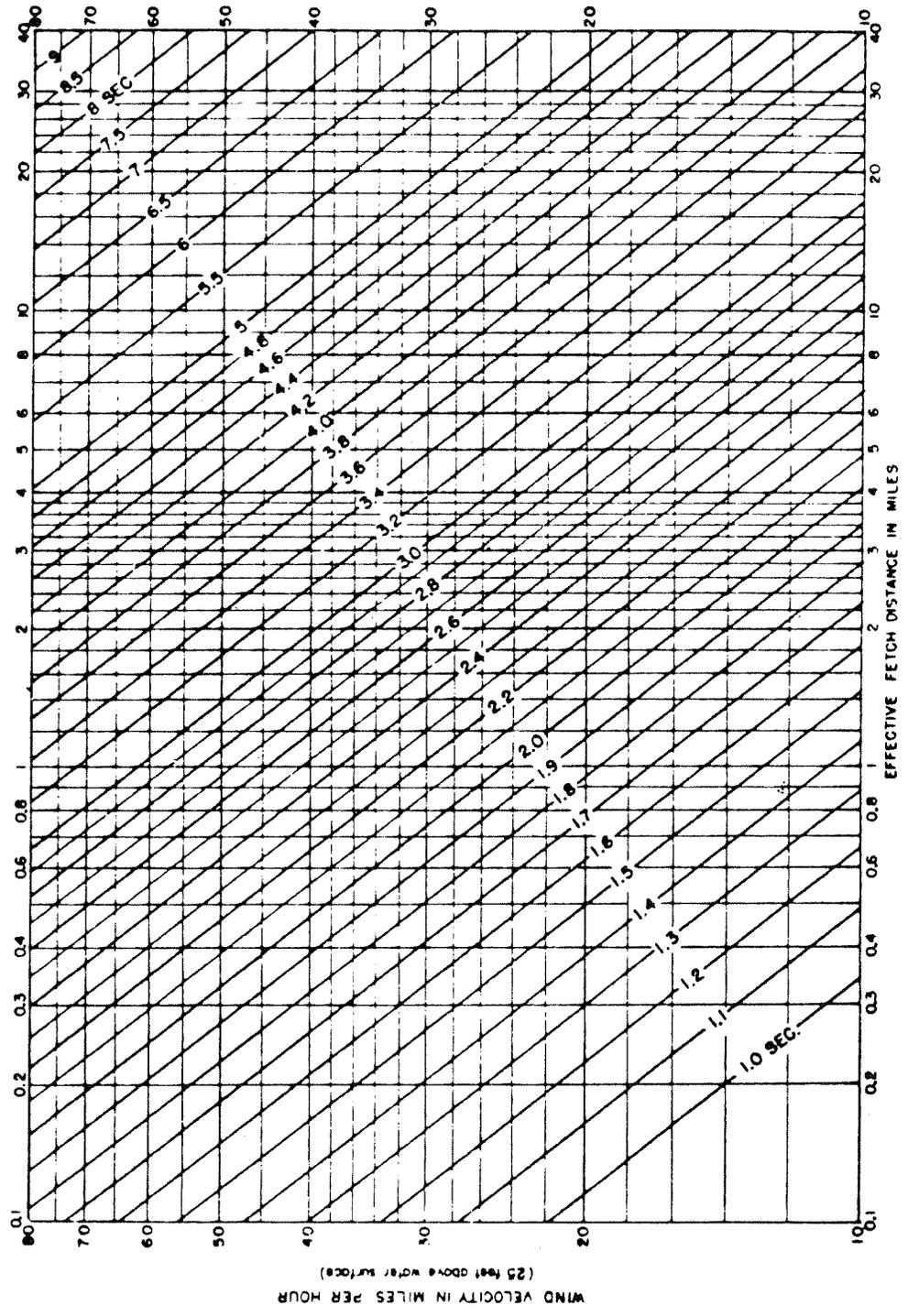


FIGURE 10.-GENERALIZED RELATIONS BETWEEN WAVE PERIODS AND RELATED FACTORS
 -DEEP WATER CONDITIONS (FROM FIGURE 12, REF. 3)

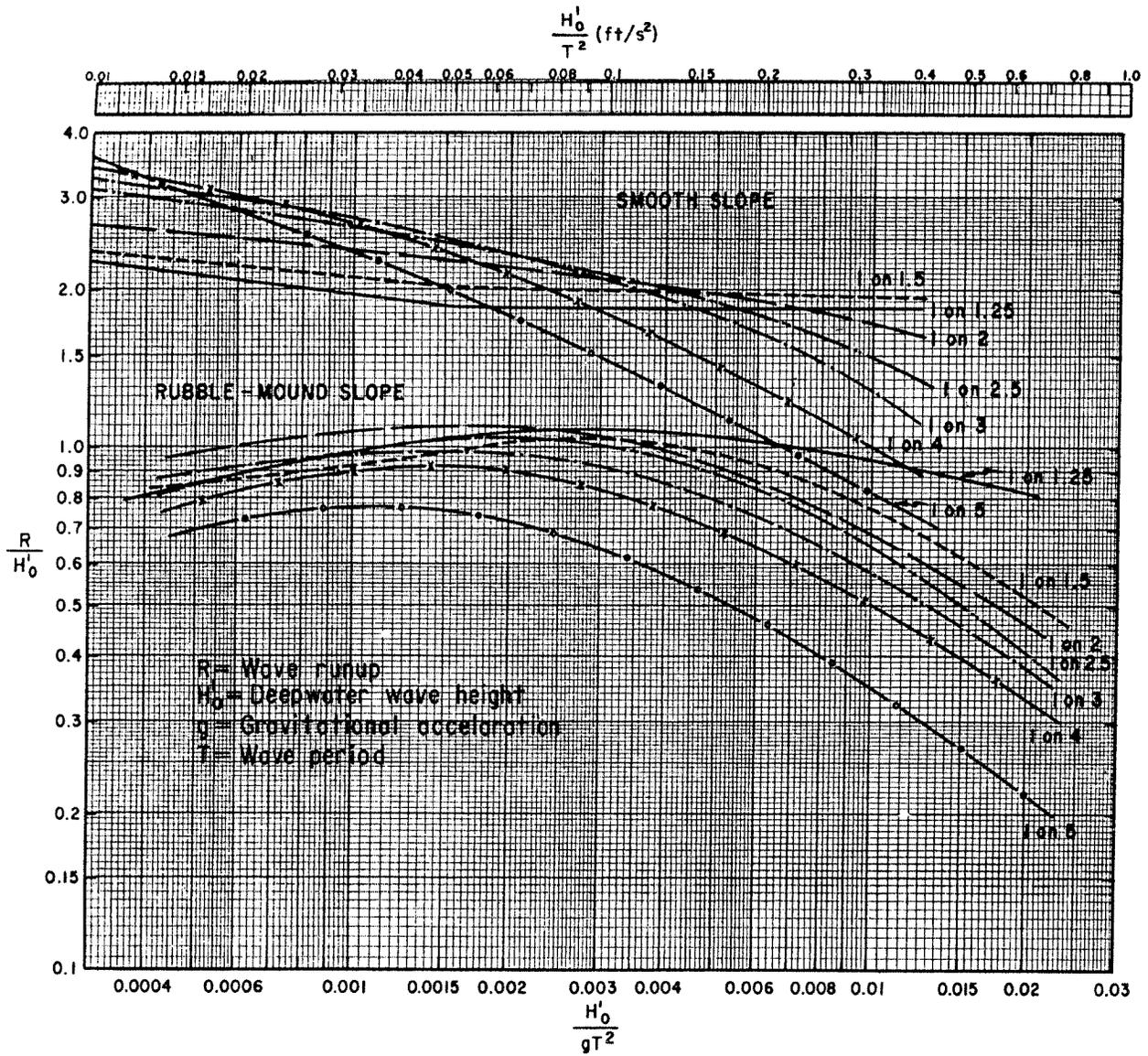


FIGURE 11.- COMPARISON OF WAVE RUNUP ON SMOOTH SLOPES WITH RUNUP ON PERMEABLE RUBBLE SLOPES (DATA FOR $d_s/H_0' > 3.0$) (FROM FIGURE 7-20, REF. 6)

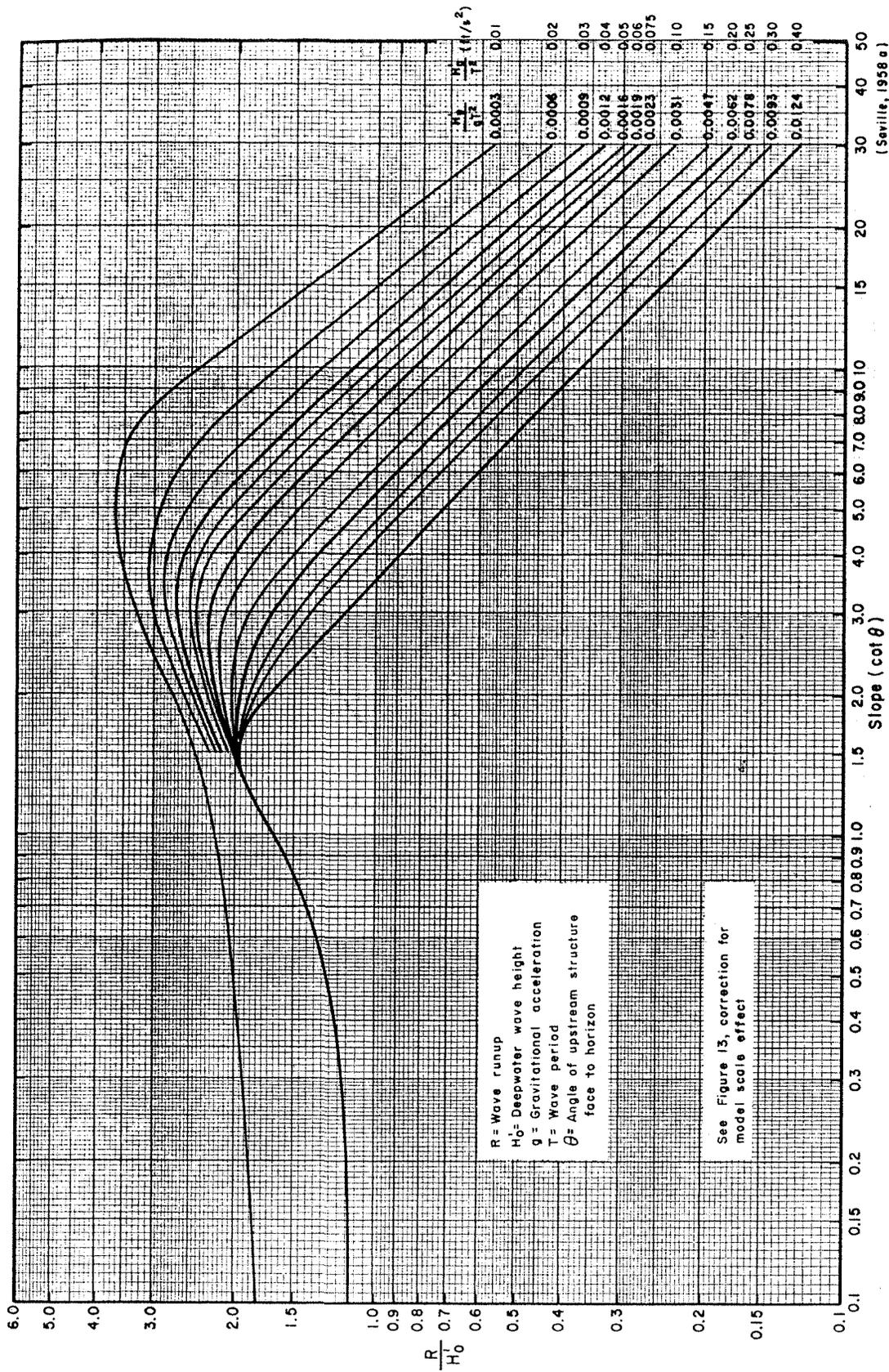


FIGURE 12. - WAVE RUNUP ON SMOOTH, IMPERMEABLE SLOPES, $d_s/H_0 \geq 3.0$
 (FROM FIGURE 7-12, REF. 6)

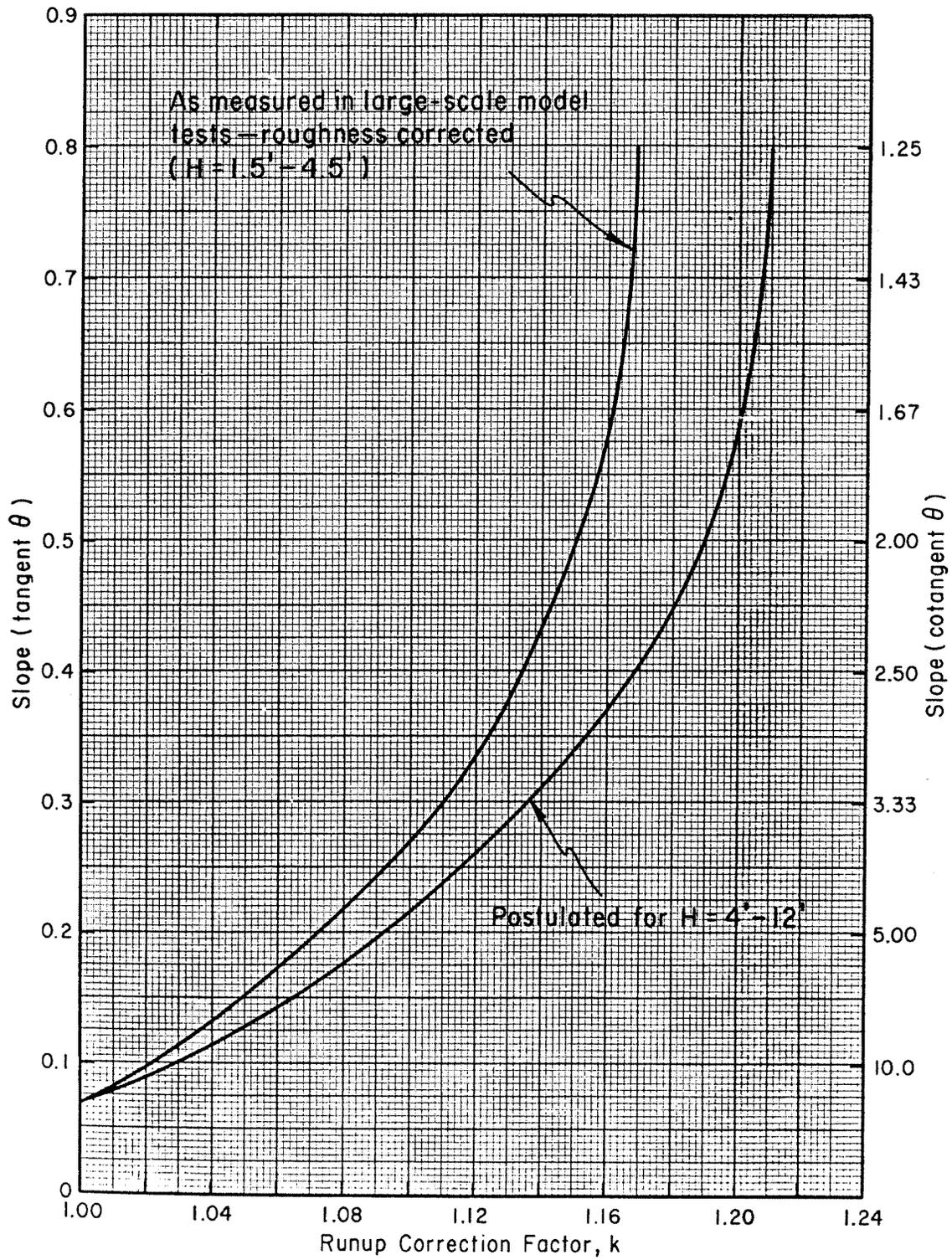


FIGURE 13.- RUNUP CORRECTION FOR SCALE EFFECTS
(FROM FIGURE 7-13, REF. 6)

3. Examples of Computations of Wind-generated Wave Runup and Setup

a. Choke Canyon Dam and Reservoir. -

Given:

- Wind data at or near the damsite are not available. For this example, it can be assumed that the generalized fastest mile and 1-hour maximum winds as shown in figures 3 and 7 can be used to determine maximum winds. Although the prevailing wind is from the southeast, high winds can occur from any direction, and values from the figures will be used as maximum winds in this example. However, those values are too severe for use as moderate winds.
- The dam is an irregular-curved-zoned earthfill embankment with an upstream slope of 0.33:1. The upstream slope is protected by soil cement.
- The pool elevation at the top of the active conservation pool is 220.5 feet. The maximum pool elevation under IDF conditions is 232.8 feet.
- The top of dam elevation without camber is 241.1 feet. A camber with 1.3 feet maximum height was added to the top of the dam for the anticipated settlement.
- The angle between the central radial of maximum effective fetch and a line normal to the dam is 23°.

- The average reservoir depth along the central fetch radial with water at the top of conservation pool is about 50 feet.

Find:

- Effective fetch lengths for maximum and top of conservation pools.
- Design winds and wave heights for determining minimum and normal freeboards.
- Wave runup and wind setup for minimum and normal freeboards.
- Minimum freeboard requirement.
- Normal freeboard requirement.

Normal Freeboard - Top of Conservation Pool

Maximum winds as defined in the appendix should be used in this computation. From figure 3, the fastest mile (about 1-minute duration) summer wind velocity is about 60 mi/h at 25 feet above ground.

From figure 7, the maximum 1-hour summer wind velocity is about 35 mi/h at 25 feet above ground.

Using table 3 and equation 1, the effective fetch length was computed to be 4.67 miles. The layout of radials are shown on figure 14. While a 6° spacing was used in this example, other spacings could be used. The estimated wind velocity over water can be obtained by multiplying the wind velocity over land by a factor of 1.3 (table 2).

Table 3. - Choke Canyon Reservoir - Computation of effective fetch

α_j	$\cos \alpha_j$	Unadjusted X_1 (top of conservation pool)	Unadjusted X_2 (maximum W.S.)	$X_1(\cos \alpha_j)^2$	$X_2(\cos \alpha_j)^2$
42	0.743	2.25	2.94	1.24	1.62
36	0.809	3.50	3.62	2.29	2.37
30	0.866	2.90	3.03	2.17	2.27
24	0.914	2.40	2.52	2.00	2.11
18	0.951	2.20	5.25	1.99	4.75
12	0.978	7.63	7.83	7.30	7.49
6	0.995	7.58	9.80	7.50	9.70
0	1.000	8.50	10.50	8.50	10.50
6	0.995	5.72	9.68	5.66	9.58
12	0.978	5.65	8.08	5.40	7.73
18	0.951	6.00	6.30	5.43	5.70
24	0.914	5.98	6.15	5.00	5.14
30	0.866	5.15	5.29	3.86	3.97
36	0.809	4.23	5.04	2.77	3.30
42	<u>0.743</u>	3.55	3.90	<u>1.96</u>	<u>2.15</u>
Total	13.512			63.07	78.38

$F_{e1} = 63.07/13.512 = 4.67$ miles (top of conservation pool)
 $F_{e2} = 78.38/13.512 = 5.80$ miles (maximum water surface)

Table 4. - Over water wind computations - Choke Canyon Reservoir - Normal freeboard

Wind duration (mi/h)	Wind velocity over land (mi/h)	Wind velocity over-water (mi/h) (conservation pool)
1	60	78
60	35	46
120	34 (0.96 x 35)	44

From figure 9, the over water wind velocity duration relationship for a number of arbitrary points and a 4.67-mile effective fetch follows:

Table 5. - Wind velocity and duration data points -
Choke Canyon Reservoir - Normal freeboard

Wind velocity over-water (mi/h)	Wind duration (min)
40	57
50	52
60	48
70	46

A plot of wind velocity and duration based on the data from tables 4 and 5 is shown in figure 15.

From figure 9, the significant wave height (H_s) = 4.3 feet, therefore

$$H = 1.27 \times 4.3 = 5.5 \text{ feet for top of conservation pool.}$$

From figure 10, the significant wave period (T) = 4.2 seconds.

From equation 2, the wave length (L) = $5.12 T^2 = 5.12 \times (4.2)^2 = 90.3$ feet. Reservoir depth is greater than one half of 90.3 feet, therefore the deepwater equation is valid.

For wave runup on smooth soil cement, use a factor of 1.4 (selected by the designer).

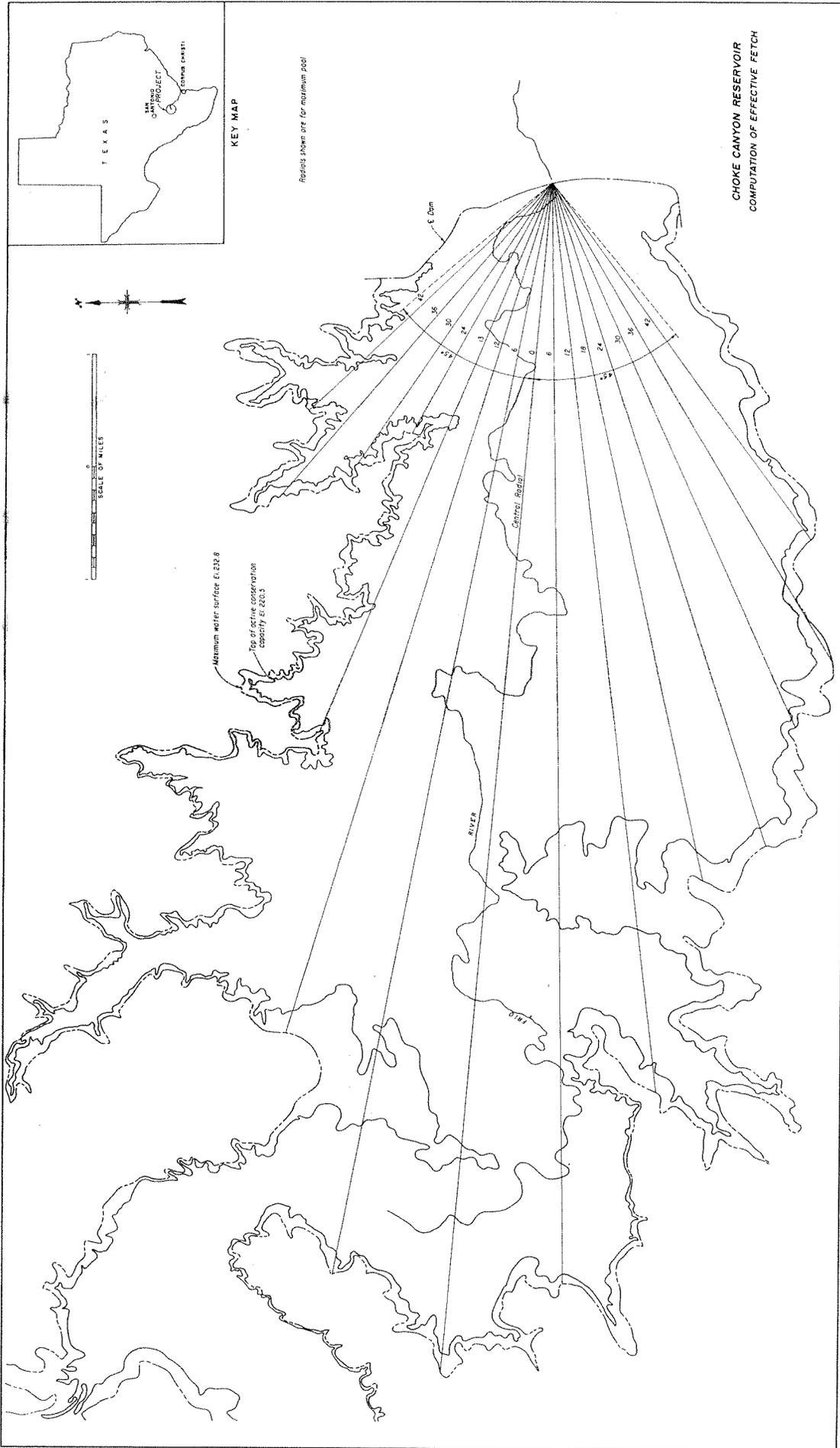


FIGURE 14.- COMPUTATION OF EFFECTIVE FETCH LENGTH - CHOKE CANYON RESERVOIR

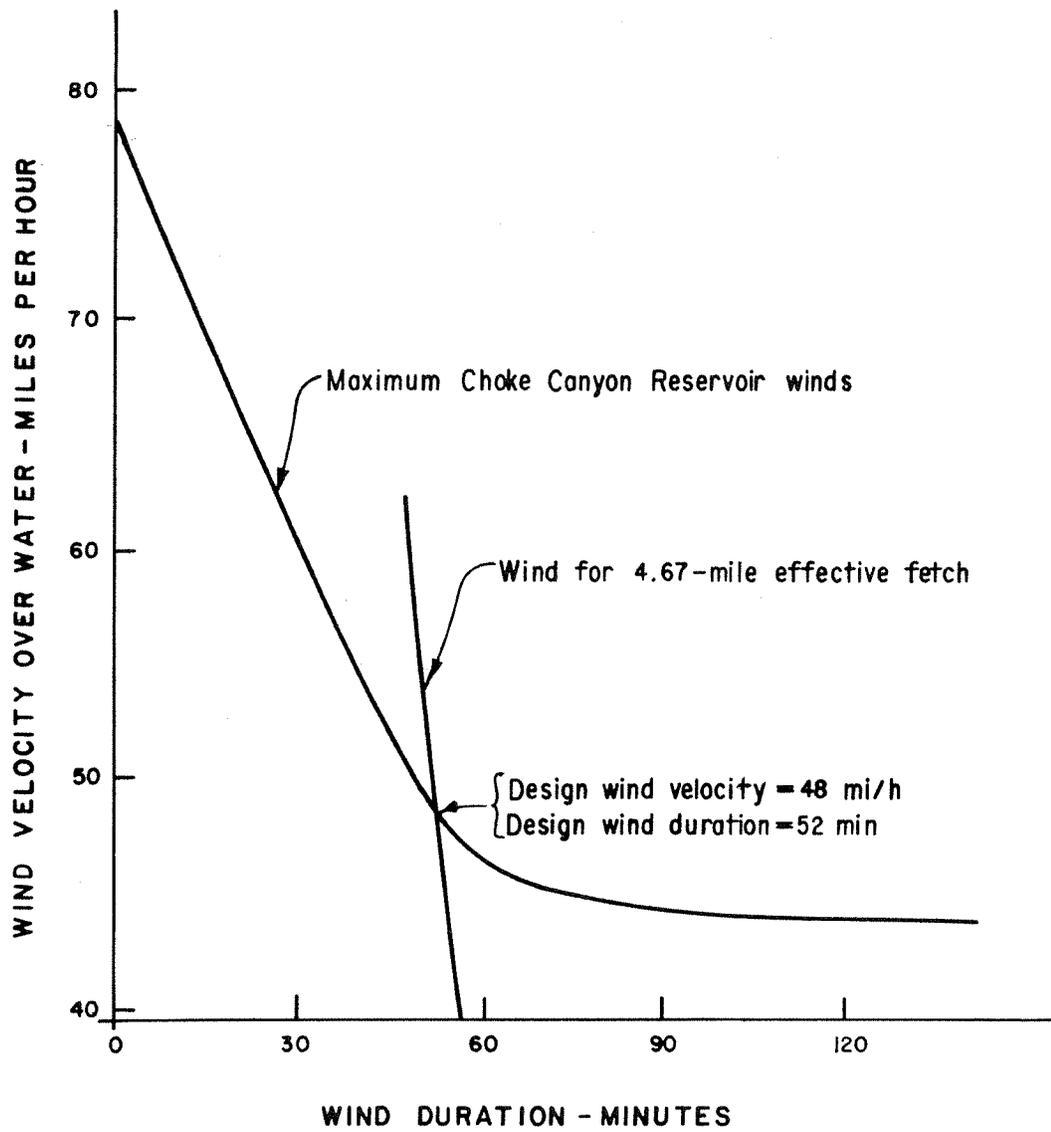


FIGURE 15. - PLOT OF OVER WATER WIND VELOCITY VERSUS DURATION - CHOKO CANYON RESERVOIR

From equation 3:

$$R = \frac{1.4 H}{0.4 + (H/L)^{0.5} \cot \theta} = \frac{1.4 \times 5.5}{0.4 + (5.5/90.3)^{0.5} (3)} = 6.8 \text{ feet}$$

A correction factor should be applied to the computations because the direction of wave propagation is not normal to the embankment but is at an angle of 23°. The actual wave runup

$$= 6.8 \text{ feet} \times \cos 23^\circ = 6.3 \text{ feet}$$

Wind setup from equation 4:

$$S = \frac{U^2 F}{1400 D} = \frac{48^2 (2 \times 4.67)}{1400 \times 50} = 0.3 \text{ feet}$$

Normal freeboard requirement equals the sum of wave runup and wind setup.

$$\text{Normal freeboard} = 6.3 + 0.3 = 6.6 \text{ feet}$$

The dam crest elevation should equal at least normal freeboard plus top of active conservation pool.

$$6.6 + 220.5 = 227.1 \text{ feet}$$

The existing top of dam elevation of 241 feet is sufficient for this condition.

Minimum Freeboard - Maximum Pool

The effective fetch length is 5.80 miles (table 3). Moderate winds as defined in the appendix should be used for these computations.

Because detailed data are not available, 80 percent of maximum winds will be used for illustrative purposes. The estimated wind velocity over water can be obtained by multiplying the wind velocity over land by a factor of 1.3.

Table 6. - Over water wind computations -
Choke Canyon Reservoir - Minimum freeboard

Wind duration (min)	Wind velocity over land (mi/h)	Wind velocity over water (mi/h) (maximum pool)
1	48 (60 x 0.8)	62
60	28 (35 x 0.8)	36
120	27 (34 x 0.8)	35

From figure 9, the wind velocity duration relationship for a 5.80-mile effective fetch is shown in table 7.

Table 7. - Wind velocity and duration data points -
Choke Canyon Reservoir - Minimum freeboard

Wind velocity over water (mi/h)	Wind duration (min)
30	76
40	67
50	60

A plot of the wind velocity versus wind duration curves for the data in tables 6 and 7 results in a design wind velocity of 36 mi/h and a duration of 70 minutes. (This plot is not included in this report.)

From figure 9, the significant wave height (H_s) = 3.6 feet.

From figure 10, the design wave period (T) = 3.9 seconds.

Wave length (L) = $5.12 T^2 = 77.9$ feet.

Wave runup:

$$R_s = \frac{1.4H_s}{0.4 + (H/L)^{0.5} \cot \theta} = \frac{1.4 \times 3.6}{0.4 + (3.6/77.9)^{0.5} (3)} = 4.8 \text{ feet}$$

The actual wave runup after correction for wave direction

$$= 4.8 \text{ feet} \times \cos 23^\circ = 4.4 \text{ feet.}$$

Wind setup:

$$S = \frac{U^2 F}{1400 D} = \frac{36^2 (2 \times 5.80)}{1400 \times 62} = 0.2 \text{ feet.}$$

Minimum freeboard requirement

$$= 4.4 + 0.2 = 4.6 \text{ feet.}$$

The dam crest elevation without camber should equal at least minimum freeboard plus maximum water surface.

$$4.6 + 232.8 = 237.4 \text{ feet}$$

The existing dam crest elevation of 241 feet provides sufficient freeboard allowance.

Comparison and Comment

From the paragraphs on limited scope studies for a fetch length of 10.5 and 8.5 miles, the minimum and normal freeboard requirements are

about 7 and 9 feet, respectively, which are greater than the computed 4.6 and 6.6 feet. Considering soil cement as a smooth surface and the application of a factor of 1.4 to the wave runup computation appears to result in reasonably conservative freeboard requirements.

b. Pueblo Dam and Reservoir. -

Given:

- The only wind data that are readily available near this reservoir site is information on the fastest mile of wind from the meteorological station located at Pueblo Memorial Airport.
- The dam is a composite dam consisting of 8,480-foot embankment and 1,750-foot concrete sections. They have upstream slopes of 3:1 and 0.4:1, respectively. The embankment sections are protected by 3 feet of riprap. The top of the embankment section and top of the parapet wall on the concrete section are at El. 4925.
- The maximum reservoir water surface elevation is 4919.0 feet and the top of the joint use pool elevation is 4893.8 feet.
- The water depth from the top of the joint-use pool at the damsite is about 130 feet and the average depth along the central fetch radial is about 112 feet.

Find:

- Effective fetch lengths for the maximum and top of joint-use pools.
- The design winds and wave heights for determining minimum and normal freeboards.
- Wave runups and wind setups for minimum and normal freeboards.
- Minimum freeboard requirement.
- Normal freeboard requirement.

Table 8. - Pueblo Reservoir - Computation of effective fetch

α_j	$\cos \alpha_j$	Unadjusted X_1 (top joint-use pool)	Unadjusted X_2 (maximum W.S.)	$X_1(\cos \alpha_j)^2$	$X_2(\cos \alpha_j)^2$
42	0.743	1.18	1.27	0.65	0.70
36	0.809	1.20	1.42	0.79	0.93
30	0.866	1.42	1.45	1.06	1.09
24	0.914	1.38	1.39	1.15	1.16
18	0.951	3.08	3.71	2.79	3.36
12	0.978	3.67	4.03	3.51	3.85
6	0.995	4.43	4.52	4.39	4.47
0	1.000	4.85	4.98	4.85	4.98
6	0.995	1.97	3.05	1.95	3.02
12	0.978	1.82	2.05	1.74	1.96
18	0.951	1.43	1.44	1.29	1.30
24	0.914	1.38	1.41	1.15	1.18
30	0.866	1.29	1.39	0.97	1.04
36	0.809	1.20	1.28	0.79	0.84
42	<u>0.743</u>	1.21	1.25	<u>0.67</u>	<u>0.69</u>
Total	13.512			27.75	30.57

$F_{e1} = 27.75/13.51 = 2.05$ miles (top of joint-use pool)
 $F_{e2} = 30.57/13.51 = 2.26$ miles (maximum water surface)

Table 9. - Pueblo Memorial Airport wind data

Date	Anemometer elevation (feet)	Annual fastest mile windspeed and direction (recorded at anemometer elevation)				Calculated fastest mile windspeed at 10 meters above ground (corrected speed)			
		All				All			
		directions	NW	W	SW	directions	NW	W	SW
9-7-41	36.	56.	N			55.			
6-3-42	36.	57.	NW			56.			
1-21-43	36.	59.	NW			58.			
11-25-44	36.	57.	N			56.			
3-26-45	36.	63.	SW			62.			
10-29-46	36.	59.	SW			58.			
3-23-47	36.	57.	W			56.			
4-10-48	36.	57.	W			56.			
5-20-49	36.	59.	W			58.			
1-17-50	36.	80.	W			79.			
12-22-51	36.	56.	W	56	W	55.	55		
3-12-52	36.	61.	NW	61	NW	60.	60		
2-15-53	36.	56.	W	56	SW	55.	55		
3-18-54	36.	63.	W	63	W	62.	62		
4-12-55	34.	72.	N	63	W	72.	63		
3-27-56	34.	55.	NW	55	NW	55.	55		
3-27-57	34.	68.	N	44	W	68.	44		
4-22-58	34.	49.	N	47	NW	49.	47		
11-4-59	34.	61.	N	42	W	61.	42		
4-16-60	34.	66.	W	66	W	66.	66		
4-18-61	34.	72.	N	66	NW	72.	66		
8-15-62	21.	68.	N	52	W	74.	57		
4-15-63	21.	64.	SW	64	SW	69.	69		
7-28-64	21.	73.	NE	72	W	79.	79		
1-31-65	21.	59.	W	59	W	64.	64		
3-22-66	21.	56.	N	56	W	61.	61		
5-19-67	21.	59.	N	57	SW	64.	62		
4-3-68	21.	49.	N	43	NW	53.	47		
6-24-69	21.	54.	NW	54	NW	59.	59		
3-24-70	21.	52.	N	49	NW	56.	54		
11-13-71	21.	63.	NW	63	NW	69.	69		
8-2-72	21.	59.	N	55	NW	64.	60		
12-12-73	21.	64.	NW	64	NW	70.	70		
3-2-74	21.	59.	W	59	W	64.	64		
5-12-75	21.	70.	N	67	NW	76.	73		
2-20-76	21.	63.	N	41	SW	68.	45		
3-11-77	21.	66.	N	52	NW	72.	57		
4-9-78	21.	58.	N	40	NW	62.	44		
10-29-79	21.	55.	N	32	NW	59.	35		
1-25-80	21.	56.	NE	51	SW	61.	56		

The sample number of annual observations = 40.00
 The sample mean = 62.80
 The sample standard deviation S = 7.44
 The sample coefficient of skew G = 0.54

Normal Freeboard - Top of Joint Use

Maximum winds as defined in the appendix should be used in this computation. An analysis of fastest mile wind data for the Pueblo Memorial Airport is shown on figure 16. The wind data used in the statistical analysis were adjusted to a common base anemometer level of 10 meters. The results of those computations should be adjusted to a level of 25 feet to be consistent with wave runup data. This adjustment can be accomplished using the following equation [9]:

$$\frac{U(z)}{U(10)} = \frac{\ln \frac{z}{z_0}}{\ln \frac{10}{z_0}} \quad \text{[Equation 5]}$$

where z = height above the ground in meters

z_0 = roughness length, assume 0.05 m for open terrain

U = windspeed

A 200-year wind determined from the NW, W, and SW data has a velocity of 83 mi/h at an elevation of 10 meters. Adjusting this velocity to the 25-foot level:

$$U_{25ft} = 83 \frac{\ln \frac{(25 \times 0.3048)}{0.05}}{\ln \frac{10}{0.05}} = 79 \text{ mi/h}$$

The historic maximum wind at the Pueblo station has a velocity of 80 mi/h from the west, which is the general direction of the effective fetch and is close to the value determined from equation 5.

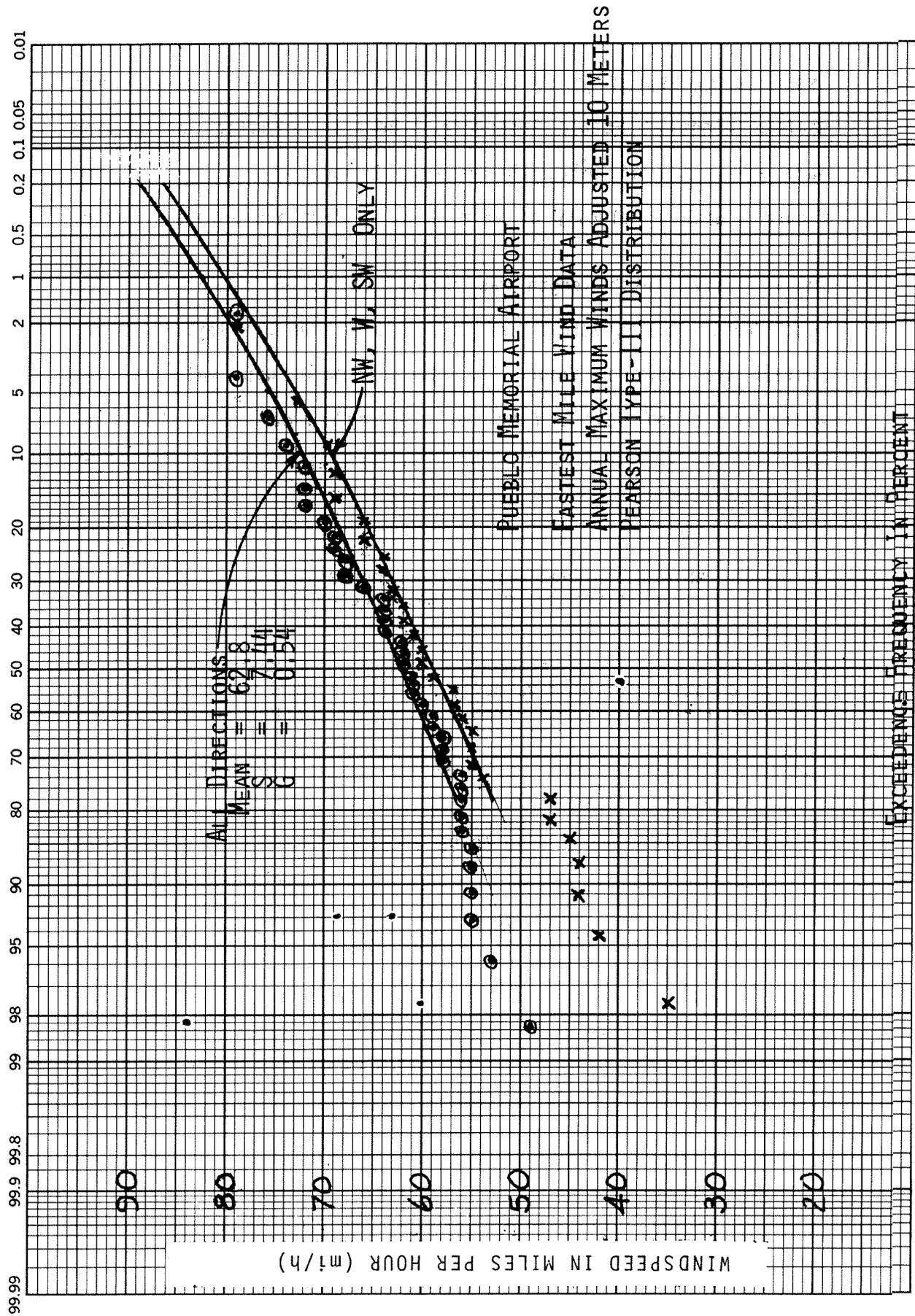


FIGURE 16. - WIND EXCEEDENCE FREQUENCY -- PUEBLO MEMORIAL AIRPORT

Assume a windspeed of 80 mi/h for the fastest mile represents maximum winds at Pueblo Reservoir. Because detailed hourly wind data are not available, the maximum 1-hour wind velocity from figure 6 with a velocity of 55 mi/h at 25 feet above land is used.

The effective fetch length was determined to be 2.05 miles. The fetch length computations are in table 8 and the layout of the radials are shown on figure 17. The estimated wind velocity over water for an effective fetch of 2.05 miles can be obtained by multiplying the wind velocity over land by a factor of 1.21 (table 2).

Table 10. - Over water wind computations -
Pueblo Reservoir - Normal freeboard

Wind duration (min)	Maximum wind velocity over-land (mi/h)	Maximum wind velocity over-water (mi/h)
0.75	80	97
60	55	67
120	53 (0.96 x 55)	64

From figure 9, the over-water wind velocity duration relationship for a 2.05-mile effective fetch follows:

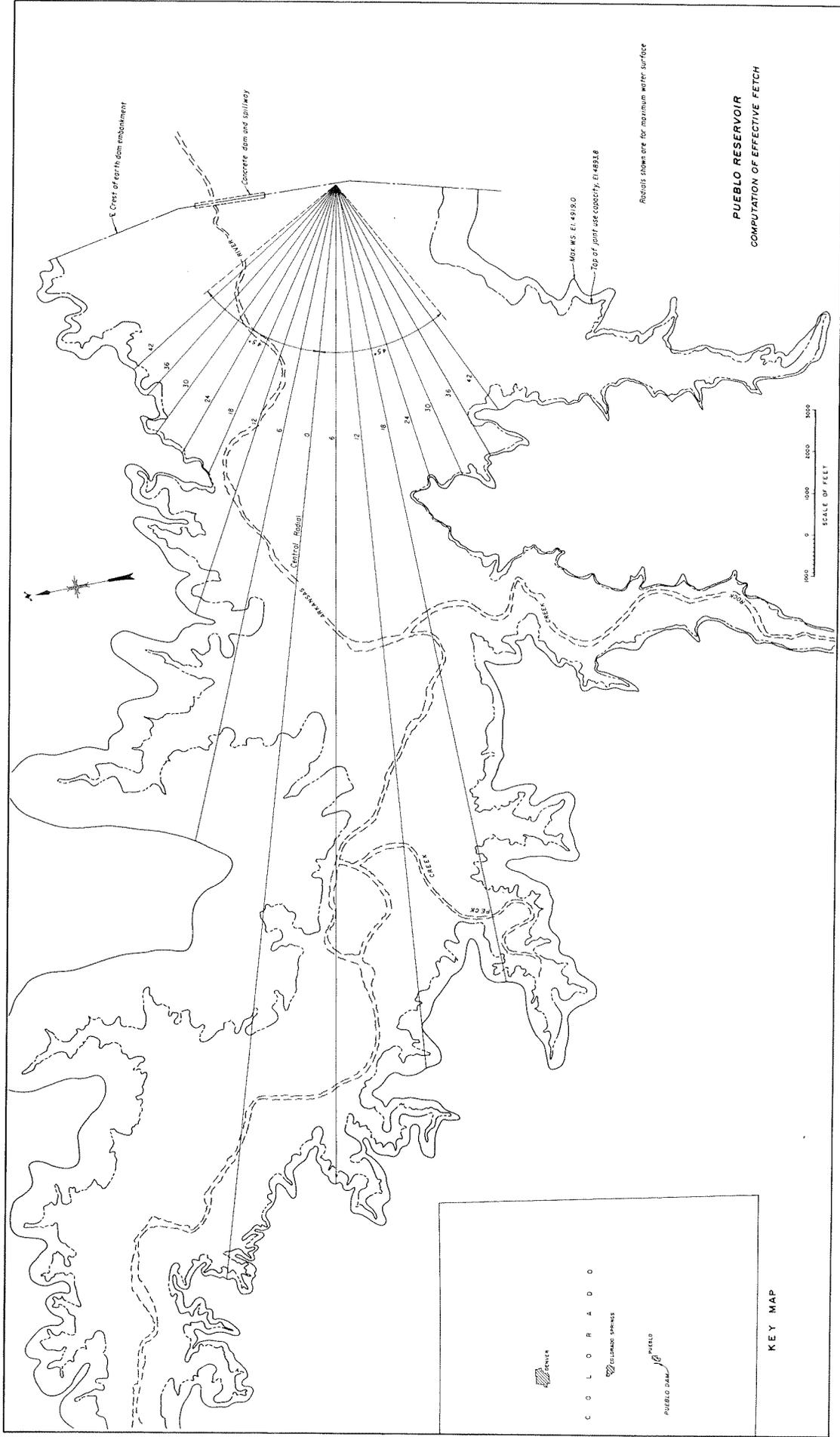


FIGURE 17.-COMPUTATION OF EFFECTIVE FETCH LENGTH - PUEBLO RESERVOIR

Table 11. - Wind velocity and duration data points - Pueblo Reservoir - Normal freeboard

Wind velocity over water (mi/h)	Wind duration (min)
50	29
60	27
70	25
80	24

A plot of the wind velocity versus duration curves based on the results from tables 10 and 11 is shown in figure 18.

From figure 9, the significant wave height (H_s) = 4.9 feet,

$$H = 1.27 \times 4.9 = 6.2 \text{ feet for joint-use pool.}$$

From figure 10, the wave period (T) = 4.1 seconds.

From equation 2, the wave length (L) = $5.12 T^2 = 86.1$ feet.

Reservoir depth is greater than 43.05 (i.e., $86.1/2$) so the deep-water computation is valid.

Normal Freeboard Requirement, Concrete Section

From figure 12 with $H/T^2 = 6.2/4.1^2 = 0.37$, $\cot \theta = 0.4$, and $D/H = 130/4 > 3$; $R/H = 1.25$. This value is estimated to be just above the lower limit curve.

$$\text{Wave runup (R)} = 1.25 \times H = 1.25 \times 6.2 = 7.8 \text{ feet.}$$

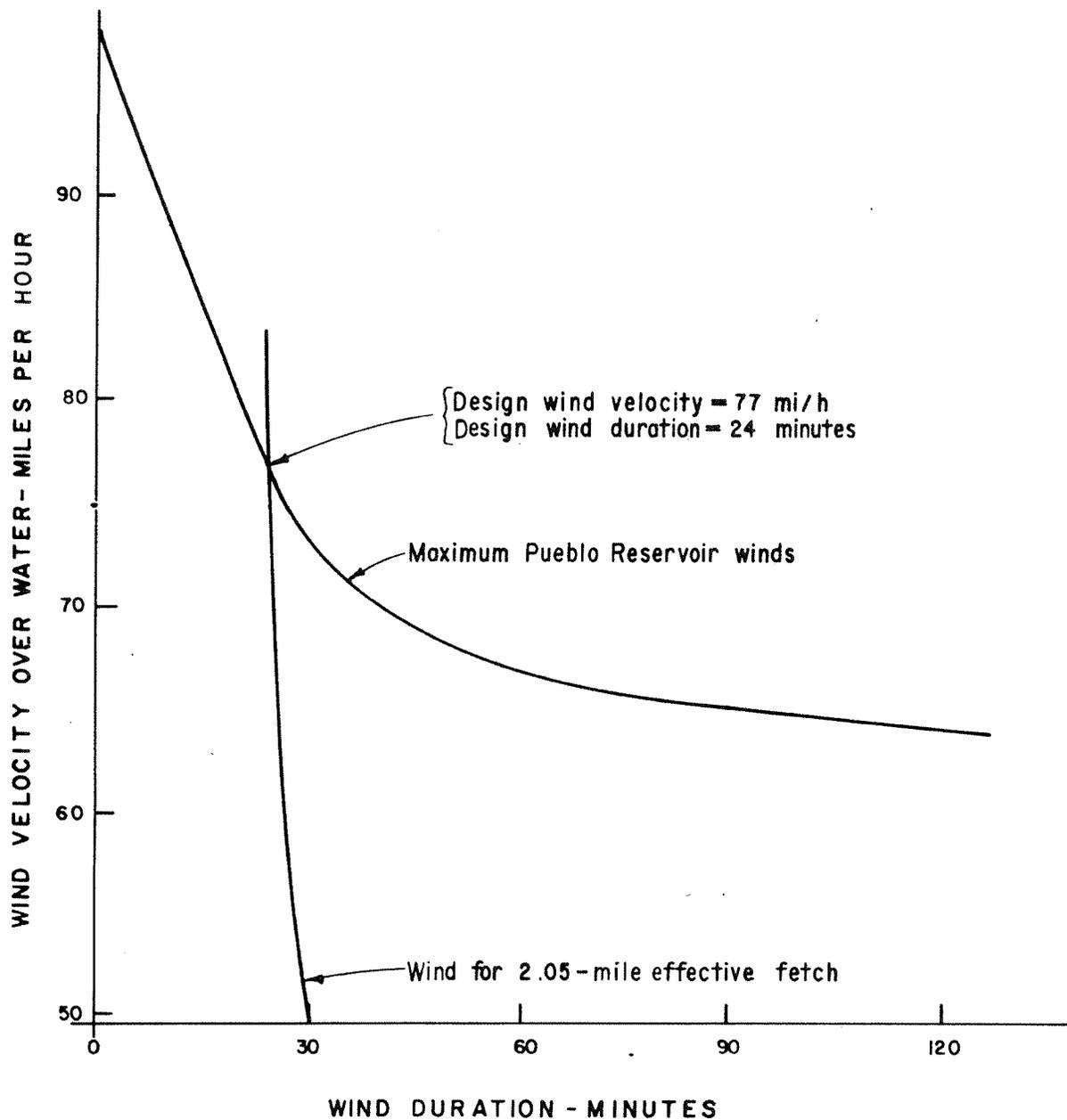


FIGURE 18.- PLOT OF OVER WATER WIND VELOCITY
VERSUS DURATION-PUEBLO RESERVOIR

From figure 13, runup correction factor is 1.21 so adjusted runup =
 $1.21 \times 7.8 = 9.4$ feet.

Using equation 4, the wind setup is:

$$S = \frac{U^2 F}{1400 D} = \frac{(77)^2 (2 \times 2.05)}{1400 \times 112} = 0.16 \text{ feet.}$$

Normal freeboard requirement is the sum of wave runup and wind
setup = $9.4 + 0.2 = 9.6$ feet.

Minimum top of parapet wall elevation required is $9.6 + 4893.8 =$
 4903.4 feet which is 21.6 feet below the top of the existing parapet
wall. (Elevation 4893.8 is top of joint-use pool.)

Normal Freeboard Requirement, Riprap Embankment Section

Wave runup from equation 3 for an embankment section armored with
riprap:

$$R = \frac{H}{0.4 + (H/L)^{0.5} \cot \theta} = \frac{6.2}{0.4 + (6.2/86.1)^{0.5} (3)} = 5.1 \text{ feet}$$

Wind setup = 0.2 foot.

Total requirement = $5.1 + 0.2 = 5.3$ feet.

The minimum dam crest elevation required $5.3 + 4893.8 = 4899.1$ feet
which is 25.9 feet below the existing dam crest elevation.

Minimum Freeboard - Maximum Pool

Moderate winds as defined in the appendix should be used for these
computations. The analyses of fastest mile wind data in table 9,

presented in figure 16, show that a 10-year wind value from the general direction of the effective fetch (NW, W, and SW) would be about 69.5 mi/h. Adjusting that windspeed from a level of 10 meters to 25 feet using equation 5 results in a design windspeed of 66 mi/h. This is about 82 percent of the maximum historic wind at that site. Assume the 1- and 2-hour maximum winds are reduced to 82 percent of maximum winds to reflect moderate winds.

The effective fetch for maximum pool was determined to be 2.26 miles (table 8). For a 2.26-mile fetch, the estimated wind velocity over water is obtained by multiplying the wind velocity over land by 1.22 (table 2).

From figure 9, the over-water wind velocity duration relationship for a 2.26-mile effective fetch is shown in table 13.

Table 12. - Over water wind computations -
Pueblo Reservoir - Minimum freeboard

Wind duration (min)	Moderate wind velocity over land (mi/h)	Moderate wind velocity over water (mi/h)
0.91	66 (0.82 x 80)	79
60	45 (0.82 x 55)	54
120	43 (0.82 x 53)	52

Table 13. - Wind velocity and duration data points - Pueblo Reservoir - Minimum freeboard

Wind velocity over water (mi/h)	Wind duration (min)
50	31
60	29
70	27
80	25

A plot of wind velocity-duration curves based on data presented in tables 12 and 13 results in a design wind velocity of 62 mi/h and a wind duration of 29 minutes.

From figure 9, the significant wave height (H_s) = 4.1 feet.

From figure 10, the significant wave period (T) = 3.8 seconds.

From equation 2, the wave length (L) = $5.12 \cdot T^2 = 5.12 \times 3.8^2 = 73.9$ feet.

Minimum Freeboard Requirement, Concrete Section

From figure 12 with $H_s/T^2 = 4.1/3.8^2 = 0.28$, $\cot \theta = 0.4$, and $D/H = 130/4 > 3$; $R/H_s = 1.30$.

Wave runup (R) = $1.30 \times 4.1 = 5.3$ feet.

From figure 13, runup correction factor is 1.17, therefore, the adjusted runup is $1.17 \times 5.3 = 6.2$ feet.

Wind setup from equation 4:

$$S = \frac{U^2 F}{1400 D} = \frac{(62)^2 \times 2 \times 2.26}{1400 \times 137.2} = 0.1 \text{ foot.}$$

Total requirement = 6.2 + 0.1 = 6.3 feet.

The minimum top of dam elevation required is 6.3 + 4919 = 4925.3 feet which is 0.3 foot above the top of the parapet wall. (El. 4919 is the maximum water surface under IDF conditions.) Flood routings of the IDF show the pool would be within 0.3 foot of maximum level for only about 4 hours.

Minimum Freeboard Requirement, Riprap Embankment Section

Wave runoff from equation 3:

$$R_s = \frac{H_s}{0.4 + (H_s/L)^{0.5} \cot \theta} = \frac{4.1}{0.4 + (4.1/73.9)^{0.5} (3)} = 3.7 \text{ feet.}$$

Wind setup = 0.1 foot.

Total requirement = 3.7 + 0.1 = 3.8 feet.

The minimum dam crest elevation required is 3.8 + 4919 = 4922.8 feet which is 2.2 feet below the existing crest.

Comparison and Comment

From the paragraphs on limited scope studies, section II-A1, for maximum fetch lengths of 4.85 and 4.98 miles, the normal and minimum freeboard requirements for a riprap embankment are about 8.0 and 6.0 feet, respectively, which are greater than the computed 6.3 and 3.8 feet.

B. Hydrologic Uncertainty

Hydrologic uncertainty involves the condition of being unsure about the value of some of the parameters used in hydrologic computations. The value of some parameters must be inferred from a random sample which might not, and probably does not, represent all of the future possibilities accurately. Consequently, estimates of parameters contain some degree of uncertainty and resulting errors do not necessarily compensate each other. The impact of errors in one direction due to uncertainty can be quite different from the impact of errors in the other direction. The confidence level in computing the IDF in terms of reliability of data for developing the design storm and snowmelt runoff and other hydrologic parameters are factors that could impact on freeboard determinations. This becomes evident when the adequacy of existing IDF's are reevaluated based on all currently available data and this results in increased flood magnitudes. Problems related to inadequate hydrologic data should be resolved to the extent feasible during derivation of the IDF. If conditions exist that justify including a freeboard allowance for hydrologic uncertainty, the value should be based on the judgment of those responsible for developing the IDF.

C. Earthquake- and Landslide-generated Water Waves

Seiches or earthquake-generated water waves can develop when the resonance of a reservoir equals the resonance of the seismic shaking. Also, a wave develops in a reservoir when water rushes in to fill a "void" caused by faulting in the form of either vertical displacement or tilting under or adjacent to the reservoir. A large seiche wave (1 foot or more) is considered very remote because the time of shaking is almost always less than

what is needed to get a large oscillating wave started. Very often an earthquake quite distant from the reservoir may have more chance of creating a seiche in a reservoir than an earthquake near the reservoir. A fault displacement wave spreads radially from the point of maximum vertical displacement. If the water depth does not exceed the displacement, the wave breaks and dissipates rapidly. However, a displacement wave (bore wave), in which the water piles up behind a vertical front, is not affected by reservoir shape. If the dam is on the downdrop side of the fault, the "lowered" crest height increases the chance of a wave overtopping the dam. Methods and theories of seiche analysis can be found in references 10, 11, and 12 listed in section II-F. For fault displacement waves, applied hydraulics is used to evaluate wave height and propagation.

Earthquakes and other factors, principally rapid reservoir drawdown, may trigger landslides in a reservoir. The height of landslide-generated waves is dependent on several factors. The mass and velocity of the slide and its orientation to the reservoir probably are the most significant factors for evaluating landslide-generated waves. Of these three, velocity is the most critical. The height of a reservoir wave from a landslide can vary from a minimum disturbance to "Vaiont size." Some methods exist for estimating the approximate size of landslide-generated water waves. A starting point for this analysis can be found in a chapter entitled "Occurrences, properties, and predictive models of landslide-generated water waves" [13]. Another useful paper, scheduled for publication in the 14th ICOLD Conference in Rio de Janeiro (1982), is "Prediction of Landslide-generated Water Waves" by C. A. Pugh and D. W. Harris [14].

Earthquakes and landslides are site specific. The waves generated by these events in a reservoir must individually be analyzed as to their potential maximum height and their attenuation characteristics in the reservoir before reaching the dam. In some cases, a freeboard component for "large" waves may be beyond the economics or realities of any project. When a real danger of wave overtopping exists for a proposed or an existing dam, then an evaluation is required which may indicate a need for freeboard or other mitigating measures.

D. Settlement of Embankment and Foundation

Allowance for consolidation or settlement of an embankment dam and its foundation caused by static loading will generally be made by providing camber to the design top of dam and the impervious zone. Camber is not included as part of the freeboard requirements.

It may still be desirable to include some freeboard allowance for settlement in the embankment or foundation from static loads if there are unusual site or construction conditions. Some conditions that may justify this are:

- Unfavorable and/or poorly understood foundation conditions
- Embankment materials with unusual properties that are not well understood
- Cases where good construction control may not be exercised
- Excessive height of the maximum pool above the normal water surface
- Probability of infrequent inspection and survey of the dam crest

- Possibility of storage in the surcharge zone above the normal water surface for unusually long periods
- Access or configuration that would inhibit emergency measures such as sand bagging or diking low areas
- Unfavorable conditions in areas where the embankment contacts abutments or appurtenant structures

Any of these conditions that could cause settlement should generally be resolved during design and should be included in camber.

If conditions exist that justify including allowance for static load settlement in the freeboard, the amount would have to be based on judgment, but would rarely be more than 0.5 to 1.0 foot. Such an allowance would recognize the difference between entirely adequate and marginal foundation conditions, embankment materials, and construction methods. In reviewing the freeboard requirement for existing dams, an initial uncertainty with respect to settlement may no longer exist because of a record of satisfactory performance under conditions of high reservoir levels. Under such conditions, a settlement component would no longer be appropriate.

Settlement caused by earthquake loading (dynamic loading) is also a possibility and should be considered in determining freeboard requirements. However, the probability of a large magnitude earthquake and large flood event occurring simultaneously is extremely remote. Therefore, allowance for settlement caused by earthquake loading should only be

included in normal freeboard and possibly intermediate freeboard determinations. The magnitude of settlement caused by the MCE (maximum credible earthquake) should be included in normal freeboard determinations. If necessary, as determined on a case-by-case basis, intermediate freeboard should also include allowance for settlement caused by earthquake loading.

E. Malfunction of Spillway and Outlet Works

Operation and maintenance factors should be given careful consideration in the determination of freeboard requirements. Malfunction of the spillway and outlet works, either due to operation error, mechanical and electrical failure, or as a result of plugging with debris could cause the reservoir to rise above levels considered in the design.

Ungated spillways are less affected by and, in most cases, free from improper maintenance and operation problems. Freeboard allowance for malfunction is not required for most dams with ungated spillways except for those reservoirs which depend on the outlet works to discharge flood flows. When shaft spillways are used, particular attention should be given to potential loss of discharge capacity as a result of plugging the inlet by debris. The effect of debris would depend upon the location of flow control in the shaft spillway system. Some freeboard allowance to account for potential loss of discharge capacity as a result of debris may be warranted in some cases. Where a large gated flood outlet is used in place of a spillway or results in a smaller overflow spillway, the gated spillway freeboard allowance given below should be used.

Even with regular maintenance of equipment and adequate attendance by an operator, the possibility of malfunctions of gated spillways and outlet works due to mechanical and electrical power failure or operational error should be recognized. In determining freeboard allowances for malfunction of gated spillways, the following site-specific conditions should be considered:

- Reliability of gate operations from actual experience
- Sensitivity of gate operation to IDF characteristics and flood storage capability
- Training, experience, and physical condition of the dam tender
- Distance between the dam and dam tender's residence
- Availability of a substitute dam tender
- Road condition and accessibility of the gates and control center during floods
- Size and complexity of the gate structure and its operation
- Number of gates - chances of mechanical-type failure adversely affecting outflow is usually reduced as the number of gates increases, especially in going from one to two or from two to three gates
- Reliability of commercial and auxiliary power supplies
- Availability of emergency materials and equipment
- Availability of warning and communication systems

- Remoteness of the damsite

The designer should make an assessment of the foregoing site-specific conditions, making quantitative evaluations where possible. For example, determine the change in maximum water surface resulting from failure of one of three gates to open. For some reservoirs with large surface areas, the change in maximum water surface might be small, while for reservoirs with small surface areas, the result of losing outflow capacity from one of three gates might result in overtopping the dam. The characteristics of the flood hydrograph would also be a factor that influences the severity of the outcome of a malfunction.

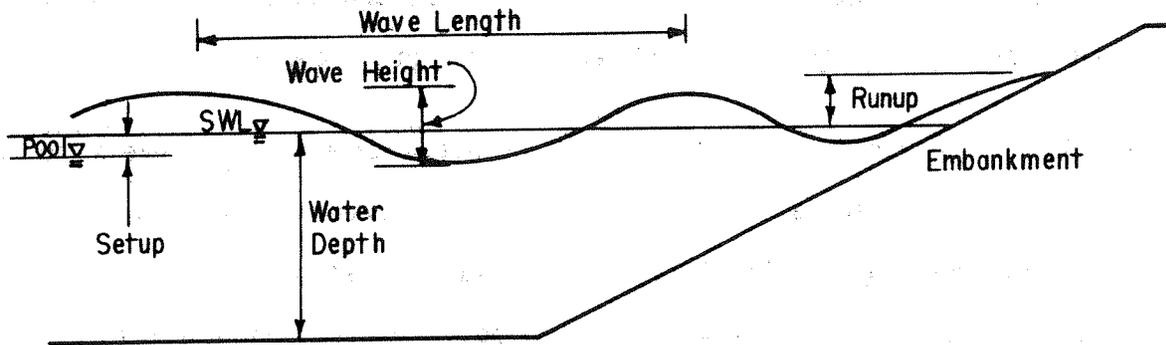
In most cases, a minimum freeboard allowance of 1 foot is considered necessary to account for the malfunction of gated spillways; however, 3 or 4 feet may be required in some cases where a valid combination of adverse conditions could reasonably be expected to occur.

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APPENDIX - DEFINITIONS



SKETCH ILLUSTRATING TERMS

CREST OF WAVE. - The highest part of a wave.

DEEP WATER. - Water so deep that surface waves are little affected by the lake bottom. Generally, water deeper than one-half the surface wavelength is considered deep water.

EFFECTIVE FETCH. - An average horizontal distance in the general direction of the wind over water, corrected for reservoir plan geometry, over which a wind acts to generate waves.

MAXIMUM WINDS. - Winds used in combination with normal pool levels to determine normal freeboard requirements. They represent the most severe winds in terms of velocity, duration, direction, and seasonal distribution that are reasonably characteristic of the region where the reservoir is located. This determination will generally include results of meteorologic studies and probability analyses of recorded wind data. The values

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WAVE PERIOD. - The time for two successive wave crests to pass a fixed point.

WAVE RUNUP. - The movement of water up a structure or beach on the breaking of a wave. The amount of runup is the vertical height above stillwater level that the water reaches.

WIND SETUP. - The vertical rise in the stillwater level on the leeward side of a body of water caused by wind stresses on the surface of the water.