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WIND GENERATED WAVES AND THEIR EFFECTS ON RIVER BUNDS

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1.0 INTRODUCTION

On large rivers like Indus and Chenab the distance between flood containing levees is quite large (as much as 12 miles at some places). The depth of flood spills is also considerable during monsoon season. Quite servere wind storms are also experienced in this season. The high velocity winds generate substantially high waves which damage the bund slopes.

This note is an effort to explain the mechanics of wave damages and propose possible remedial measures.

2.0 W A V E S

Waves are generated in water by artificial or natural disturbances like explosions, moving ships, tides, earth-quaks and winds,.Only a few decades ago the Civil Engineer's wave analysis tool-kit comprised little more than stevenson's empirical formulae. Since World War II the knowledge about wave behaviour has advanced considerably, specially about the deep water waves effecting the design and construction of Mari-time structures.

The major theories discussing the wave behaviour are introduced briefly in the following paragraphs.

1) G.B.Airy advanced the theory of small amplitude waves.

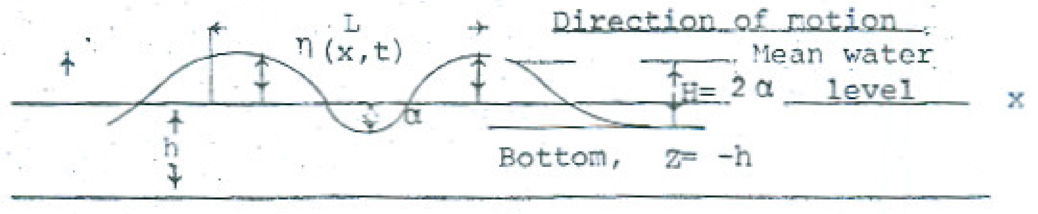

Fig =1 Small amplitude wave,system,definition sketch.

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The above sketch represents a small amplitude wave. In this figure:-

h = distance from mean water level to bottom.
$\eta(x,t)$ = instantaneous vertical displacement of water surface above mean water level.

α = wave amplitude
H = wave height = 2α for small amplitude waves
L = wave length or distance between any two corresponding positions on successive waves.

T = wave period, time interval required for motion to reoccur at a given fixed point.

C = velocity of wave propagation (phase velocity) = L/T also termed as celerity."

K = wave number = $2\pi/L$

σ = wave angular frequency = $2\pi/T$

The analysis of this wave form yields

$$C^2 = \frac{g}{K} \tan h\,(kh) \quad \text{and}$$

$$L = \frac{gT^2}{2} \tan h \frac{2\pi h}{L}$$

Wave classification according to relative depth appears in the following table:-

Range of h/L	Range of $kh = 2\pi h/L$	Types of waves
0 to 1/2	0 to π	"Shallow water" waves (long waves)
1/2 to ∞	π to ∞	"deep water" waves (short waves)

Replacing the hyperbolic functions in the earlier equations by their Asymptotes we have:

$$C^2 = \frac{g}{K} \tan h \ \ kh$$

for shallow water conditions,

$\tan h \ \ kh = kh$

Therefore $C^2 = gh$ -- (shallow water

For deep water conditions

$\tan h \ \ kh = 1$

and so, for deep water

$$C_o^2 = \frac{g}{k_o}$$

wherein

$$K_o = \frac{2\pi}{L_o} \quad \text{or}$$

$$C_o = \frac{g}{2\pi} \times T = \frac{L}{T} \quad \text{and}$$

$$L_o = \frac{g}{2\pi} \quad T^2$$

$$T = \frac{2\pi c}{g}$$

The above relations can also be expressed as

$$C_o = \frac{L_O}{T} = \frac{g}{2\pi} T = 5.12 \quad T = 2.26 \ (L)^{\frac{1}{2}}$$

$$L_o = 5.12 \ T^2 = 0.195 \ C^2$$

$$\& \quad T = \frac{2\pi c}{g} = 0.195 \quad C = 0.442 \ (L)^{\frac{1}{2}}$$

The wave energy can be expressed as average potential energy/unit area $= \frac{\gamma \alpha^2}{4}$

and kinetic energy/unit area $= \frac{\gamma \alpha^2}{4}$

Thus total wave energy/per unit area $= \frac{\gamma \alpha^2}{2}$

Where:-

γ = Specific waight = ρg
ρ = Density = mass/unit volume = $\frac{\gamma}{g}$
α = amplitude of the wave
$= \frac{H}{2} = \frac{\text{Height of wave}}{2}$

or $E_t = \frac{\gamma H^2}{8}$

2.1 The Standing Wave

When a wave encounters a vertical obstruction it gets reflected and its inter-action with another progressive wave results into a wave of double the amplitude of incident wave.

The form of standing waves may very but they appear to remain in the same position.

2.2 Finite amplitude wave theories

2.2.1 Stokes Wave Theory

Stokes presented his second order theory for waves of finite amplitude in a series form which is convergent for deep water conditions, improving upon the small amplitude theory by Airy. This paved the way for even higher order theories.

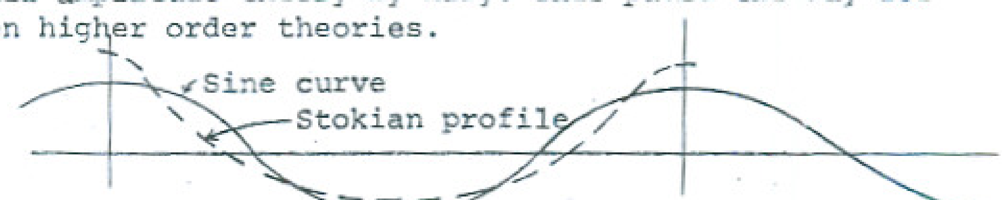

Fig.(II) Comparison of Stokian & small amplitude profiles.

In deep water if terms involving $\left(\frac{H}{L}\right)^3$ and higher powers are neglected from Fourier series relating to this analysis, we have:-

$$C = \left\{ \frac{gL}{2\pi} \left(1 + \frac{\pi^2 H^2}{L^2} \right) \right\}^{\frac{1}{2}}$$

Where

C = Wave celerity
H = Wave height
L = Wave length,

As H/L increases the crest tends to steepen and trough goes flatter than that of Airy wave, thus raising the median height. The particle orbital velocities have a superimposed forward drift of mass transport which is depicted by:-

$$U = \frac{\pi^2 H^2}{LT} = e^{4A \quad Y/L}$$

Relationships for particle velocities and wave forms become quite complex and for deep water it is usually satisfactory to adopt the simpler trochoidal theory.

2.2.2 Breaking of Waves

For any water depth and wave period there is a limit to stable wave height, beyond which the wave becomes unstable and breaks.

Stokes maintained that the particle velocity in the wave crest is equal to wave celerity in stable conditions. If the wave height increases, the particle velocity will also increase and exceed celerity and the wave would then "topple over" or "Spill".

At this stage the crest angle $\theta = 120^\circ$

The wave steepness $\left(\frac{H_o}{L_{ob}}\right) = 0.142$

Crest angle at max steepness $= 120^\circ$

Where L_{ob} = Breaking wave length in deep water.

Michell further calculated

$$\frac{L_{ob}}{L_o} = 1.2$$

While $L_o = \frac{g}{2\pi} T^2$

and $\frac{H_b}{T^2} = 0.875$

The value $\frac{H_b}{h}$ (for breaking to occur)

Varies between 0.73 (Laitone) ᵤ 0.87 (Chapplear) but the figure given by McCowan $\frac{H_b}{h} = 0.78$

is agreed to by most researchers and Reid and Bretshneider have also produced a graphical representation for a breaking wave index curve.

2.2.3 Gerstener's Trochoidal Wave

The surface form of a Gerstener wave is a Trochoid, that is the path of a point on a disc whose circumference rotates along a straight line.

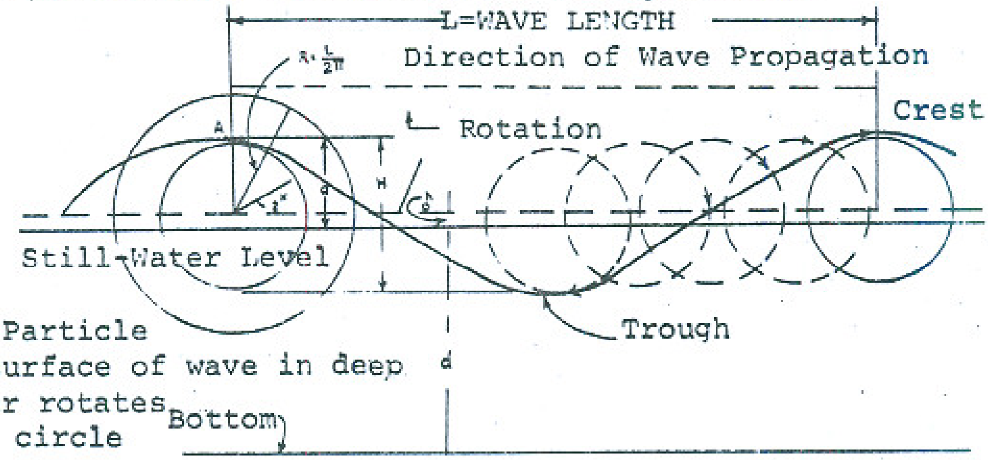

Fig.III Particle at surface of wave in deep water rotates in a circle

Considering a disc of Radius $\frac{L}{2\pi} = K^{-1}$ rolling along a horizontal line, the wave surface is the locus of a point at a radius $\frac{H}{2}$ where H is wave height.

For these waves:-

$$C = \left(\frac{gL}{2\pi}\right)^{\frac{1}{2}}$$

& r = (Radius of particle orbit) $= \frac{H}{2} e^{2\pi y/L}$

and $E_k = E_p = \frac{H^2 L}{16}\left\{1 - \frac{1}{2}\left(\frac{\pi H}{L}\right)^2\right\}$

There are some more theories like Keulegan and patterson's Cnoidal wave, Scot Russel and Boussinesq's solitary wave theory and a couple more numerical theories

Celerity of a Solitary wave $= \{g(H+h)\}^{\frac{1}{2}}$

Although a lot of work has been done both in America and Europe but no theory is so fully developed as to cater for complete range of wave forms. All the theories have limited applications in certain specific areas and some errors have to be accepted.

3.0 GENERATION OF WAVES

Waves are generated in water by artificial and natural disturbances like explosions moving ships,tides, earth-quaks and winds.For us, the wind generated waves are of real consequence and great importance because these are the ones which cause serious damages to river bunds during wind storms in floods.

3.1 Wind Generated Waves

These waves are generated by the transfer of energy from air moving over the water surface. The energy transfer is effected through tangential stresses developing between air and water due to excessive wind velocity and the water surface reacting to varying pressure in the air stream blowing past the water surface.

The development of the theory of generation of wind waves is at present at an interesting stage. Although the mechanism is not yet fully understood, certain generalization can be made regarding the generation of waves in deep water:-

a) A minimum wind velocity is required to cause surface waves (3 ft/second (1m/Sec) measured at a height of about 6 ft (2m) above still water level.

b) A spectrum of maximum levels of wave energy in specified narrow ranges of wave period will be associated with a given wind velocity, the wave period and height tending to increase with increasing wind.

c) The "fully developed" condition of point(b) requires the wind to blow over the area of developing waves for a certain minimum period; the stronger the wind, the longer is the period. Full development may be prevented on account of limited duration of exposure of the wave to the wind and this may be caused by a limited travel of the developing wave.

The most recent theory has been proposed by phillips. This improves upon the earlier theories by Helmholtz, jeffreys, and Sverdrup and Munk. Where as Sverdrup and Munk consider that the wind is constant in velocity in order to develope their theory, Phillips considers the fact that the wind is rapidly fluctuating about some mean value. It is very true that winds blowing over water do not consist of streams of air in steedy and uniform motion but rather of an irregular series of "puffs" and "lulls" carrying eddies and swirls distributed in a disordered manner. The atmospheric eddies, or random velocity fluctuations in the air, are associated with random stress fluctuations on the surface, both pressure (i.e. normal stresses) and tangential stresses. The eddies are borne forward by the mean velocity of the wind, and at some time they develop, interact, and decay, so that the associated stress distribution moves across the surface with a certain convection velocity dependent upon the velocity of the wind, and also evolves in time as it moves along.

These pressure fluctuations upon the water surface are responsible for the early generation of waves. The tangential stress is not considered, but phillips states in some cases that the shear stress action might not be negligible. The theory is in agreement with wave observations during the early stages of generation, but as C/U approaches unity there are other wave generating processes to take into account, such as sheltering and the effects of variation in shear stresses. Although this theory tends to an under-estimation of wave heights for C/U close to unity, it may be considered as a great advance in wave generation theory in sofar as the initial birth and growth of waves are concerned.

All the above mentioned theories are useful in attempting to understand the mechanism involved in the generation of waves. They are also useful when formulating empirical wave forecasting relationships.

4.0 FORECASTING,WAVE HEIGHT AND LENGTH

The parameters of a wave for a particular location depend on the Fetch of water, Direction,Duration and Velocity of the Wind responsible for wave generation.

Thomas Stevenson in 1864 established the following empirical formulae for wave forecasts .

1) $H= 1.5(F)^{0.5}$ ----------(A) For 'F' greater than 30 Nautical Miles

$H= 1.5(F)^{0.5} + 2.5-(F)^{0.25}$---(B) For "F' less than 30 Nautical Miles.

The above relations were developed from observations on lakes and checked on North Sea. They do not include the wind velocity Variable and cater for only the maximum velocity.

Captain Gaillard of U.S.Army Corps. of Engineers reported some observations on the height of Ocean Wave in supporting Stevenson's formulae. D.A. Molitor modified Stevenson's formulae to include the wind velocity as below:-

$H = 0.17\ (UF)^{0.5}$ ----------(C) for'F'greater than 20 Statute Miles.

$H = 0.17\ (UF)^{0.5} +2.5 - (F)^{0.5}$----(D)

where U = Wind velocity (Miles per Hr)
F = Fetch Statute Miles
H = Wave Height in feet.

The ratio of wave length to height depends on the wind velocity, duration of the storm, depth of water, and character of the bottom.According to observations made by Captain Gaillard, the ratio L/H for inland lakes, in comparatively shallow depth, is between 9 and 15, and for Ocean waves L/H is between 17 and 33. The ratios are nearly inversely proportional to the wind intensity; the smaller ratios are for the stronger wind intensities.

From equation (D) the wave height when wind velocity is Zero comes to a positive value of about 2 feet. This renders the scope of applicability of this equation limited. Creager and Justin in their publication Hydro-Electric Hand Book propose the following equation which is some what more rational:-

$$H = \frac{F^{0.37} \times U^{0.48}}{3.41} \qquad \text{-----(E)}$$

The above formulae do not take the duration of storm into account, which is of great importance specially for deep water.

Darbyshire and Draper prepared a graphical relationship for forecasting parameters for wave in deep water, which appears as Figure (IV).

4.1 Significant Height

It was established by Bretschneider that in a wave train the heights of individual waves very greatly and periodically a very high wave comes up which has almost double the height than average waves in the train. The average height of the highest one third of the waves for a given interval is termed as the "significant height", and it is found that the maximum wave height in a train is about 1.87 times the significant height.

i.e. $H_{max} = 1.87\ H_s$

The manual on "Design of Break waters and Jetties" of U.S.Army Corps of Engineers, gives the following components of a typical wave train spectrum as below:

Significant height	1.0
Average height	0.6

NOTE for Fig.IV

The wave heights indicated on the diagram relate to the anticipated maximum wave for a period of observation of 10 minutes (H max 10)..The period is of statistical convenience only and is unrelated to the duration of the storm that gives to the waves. The table below permits H_{max} for longer storm durations to be estimated.

Th	1	2	3	6	12	18	24	36	48
f	1.17	1.24	1.27	1.34	1.39	1.42	1.45	1.47	1.49

Table of wave height factors (f) against duration of storm T_h (hours (H max = H max 10 x f)

The maximum wave height may be determined by the fetch or by the duration of the storm, depending upon which limiting criterion is first encountered by an ordinate representing wind speed, traced from left to right. At this point H max 10 and the wave period T may be read off the diagram.

Example 1) 30 -knot wind blowing for 6 hours over a fetch of 20 miles. A line on 30 knots ordinate encounters 20 miles before intersecting 6 hours line and thus sea is fully developed (9ft max. wave ht.) after about 2½ hours. For a storm lasting 6 hours the table indicates f=1.34, i.e. H max = 12 feet (3.5m) T= 5½ secs.

Example 2) 40- knot wind blowing for 2 hours over a fetch of 50 miles. A line on 40 knots cuts 2 hours line before encouncering 50 miles and sea would only be fully developed after about 4½ hours, H max 10=13 feet and f=1.24, thus H max = 16 feet. (5m) T=6 sec.

Note: Data relate to depths of water of about 100-150 feet. (30-45m)

Height of simple sine waves having same energy content as actual wave train	-------------0.8
Height not exceeded more than 20% of the time.	-------------0.9
Height not exceeded more than 10% of the time.	-------------1.1
Height not exceeded more than 5% of the time.	-------------1.2
Height not exceeded more than 3% of the time.	-------------1.3
Height not exceeded more than 1% of the time.	-------------1.6
Average height of highest 1%	-------------1.7

4.2 Short Fetches and High Wind Speeds

Bretschneider established from the data collected by U.S.Army Corps. of Engineers on winds and waves in lake Okeechobes Frorida, that:-

$$H_s = 0.0555\ (U^2 F)^{0.5} \qquad \text{----------(G)}$$

$$T_s = 0.50\ (U^2\ F)^{0.25} \qquad \text{----------(J)}$$

and

$$\frac{F_{min}}{t_{min}} = 0.57\ (U^2\ F)^{0.25} = 1.14T \qquad \text{--------(M)}$$

where H_s = significant height in feet

T_s = significant period in seconds

U_s = wind speed in knots

F = fetch in nautical Miles

Figure(V) provides necessary curves for determining the wave heights and period with reference to fetch, wind speed and duration of the storm.The results obtained from the graphical representation in this figure differ from those calculated from above equations but are a better approximation.

4.3 Wave in Shallow Water

Wave motion of water particle, in shallow depth (h less than $\frac{L}{2}$) is effected by the bottom. Un-like the orbital motion in deep water the orbital velocity in shallow water is greater in a horizontal plane than in vertical. The celerity C in shallow water is $C=C_o x (\frac{b}{a})^{0.5}$ where $\frac{b}{a}$ is the ratio of vertical to horizontal axis of the elliptical orbital motion.

Where $h = \frac{L}{2}$, $\frac{b}{a} = 1$ and $C = C_o$

The conversion co-efficients for deep to shallow water are given below:

d/L	c	u	d/L	c	u
0.05	0.552	1.814	0.30	0.977	1.023
0.10	0.746	1.340	0.35	0.988	1.013
0.15	0.858	1.165	0.40	0.994	1.007
0.20	0.922	1.085	0.45	0.997	1.004
0.25	0.958	1.044	0.50	0.998	1.002

The depth of water affects the growth of the waves, initially on account of modification to wave celerity in shallow water, subsequently on account of increased energy Losses and, ultimately, by virtue of the limitation on maximum wave height. For example, Bretschneider indicates that, for Lake Okeechobee, Florida, with a fetch of 16 Km(10 miles), the significant wave heights for wind speeds between 30 and 70 knots were only about 60-65% of its deep water value in a water depth of 3.5m (12 ft).

He also established a numerical procedure for computing wave parameters in shallow water taking bottom friction into account and assuming a friction factor of f= 0.01. A relationship for forecasting waves in shallow water of constant depth, appears as Figure VI.

When deep water wave enters shallow water where the botton slopes upwards, two counter-acting influences are exerted on the wave height. One is the bottom friction which tends to reduce the wave height and the shoaling effect which tends to increase the wave length.

4.4 Effect of Fetch Width

The effect of fetch width in limiting wave growth has long been recognised but in case of sea waves the effect being very minor, is neglected. However, in inland water like lakes and rivers in floods, fetokes of great length in comparison to their width are frequently observed and the width effect becomes quite important resulting in generation of waves substantially lower than would be expected under same generating conditions in more open waters (Effective fetch Fe=Width factor xF_{actual}).

A graphical method for determining the width effect has been proposed by Saville which appears as Figure VII.

All the methods of determining wave parameters described above are based on theories, most of which are semi-empirical nature and not fully developed and so the results given by one method do not agree with the other. However, they do give a general idea of the magnitude of wave parameters and thus provide a reasonable basis for design of Maritime structures.

A summary of empirical formulae proposed for determination of wave heights is given in an American Society of Civil Engineering paper "Review of slope protection methods from which the following table has been extracted:-

Fetch,miles	Wind Velocity miles per hour.	Wave height,foot
1	50	2.7
1	75	3.0
2.5	50	3.2
2.5	75	3.6
2.5	100	3.9
5	50	3.7
5	75	4.3
5	100	4.8
10	50	4.5
10	75	5.4
10	100	6.1

5.0 EFFECT OF WAVES ON RIVER EMBANKMENTS

On large rivers like Indus and Chenab the distance between flood containing levees is quite large (as much as 12 miles at some places). The depth of flood spills is also considerable during monsoon season. Quite severe wind storms are also experienced in this season. The high velocity winds generate substantially high waves which damage the bund slopes.

When a wave strikes the sloping face of an embankment, it breaks or is partially reflected. The slopes of our river embankments (usually 3:1) are sufficiently mild to ensure effective breaking. The impact of breaking wave is explained below:-

1) The dynamic pressure is applied perpendicular to the slope, the magnitude of which can be approximately determined from the relations $P_{V,\alpha} = P_V . \sin^2 \alpha$. This force though quite large does not in any way seriously affect the stability of earthen embankment of our type design.

2) Continuous wave Run up on the sloping surface causes drift of earth mass from the slope towards deep water thus causing a beach to be created with a very flat slope below the wave action line. The slope of this·breach depends on the median diameter of the earth particles and other factors enumerated in the following paragraphs.

5.1 Beach Slopes

As explained above wave action removes the earth from the slope of the embankment and creats a beach with flat slope in continuation of the embankment slope. The Irrigation, Drainage and Flood Control Council of Pakistan sponsored a study in this behalf which was carried out at the Soil Mechanics and Hydraulic Laboratory at Hyderabad. In the report on the experiments conducted under this study in which the form of beach slope was analysed, a set of two formulae for determining the beach slope has been established.

These are:-

i) $$S_t = 0.00022 \left(\frac{H}{L} \times \frac{MD_o}{L} \right)^{-0.5}$$

ii) $$S_t = 1.59 \left(\frac{f}{100} \times S_i \times \frac{H}{L} \times \frac{MD_o}{L} \right)^{0.2}$$

The above report also suggested that relationship at (ii) gives a better verification with model results and may thus be adopted. The Notations used in this relationship are:-

ST = Final Beach slope.
SI = Initial slope of the embankment before wave-action.

f = Actual frequency of wave in cycles per Minute.

H = Height of wave in ft.
L = Length of wave in ft.
H/L= Wave factor
MD_o= Median particle diameter. (mm)

Some observations on the prototype at Taunsa Barrage indicate that the stable beach slopes for the soil used in the embankment at Taunsa Barrage comes to 1 in 12 while the formula mentioned above gives a slope of 1 IN 8.8 (Calculation sheet attached). Similar is the case with the Baluchwah Flood Bund at the left bank of river Chenab where beach slope of 1 in 10 was observed after the floods of 1976. The beach slope as calculated from the above formula comes to 1 in 8. This indicates that the subject will need some further study and co-relation of the results from the model studies with the prototypes at different places to arrive at a final conclusion regarding establishment of a firm relationship for calculation of beach slopes specially so when the embankments are normally made with heterogeneous soil mass, laminated structure with varying qualities against erosion and the variation in the degree of compaction of the embankment earth work.

5.2 Free Board

In the design of water retaining structures the crest of the embankment/structure is to be kept higher than the level of water to be contained. This margin is necessary for safety against overtopping due to rise in water levels on account of disturbances in water like, waves, wind set up, river set in case of river ponds and seiches. Some small margin is also necessary for contingent requirement as factor of safety. In some cases the depth of frost zone may be the controlling factor.

5.2.1 Wave Effect

When a wave strikes an embankment with a sloping front face as is the case with our river embankment, it breaks completely or is partially reflected in which case standing waves (clapotis) are encountered which we want to avoid on account of their large height (twice that of an incident wave). We therefore have to provide a slope which ensures complete breaking of the maximum number of waves in a wave train. For ensuring this the front slope of the embankment must satisfy the equation:

$$\operatorname{Tan} \alpha \approx \frac{B}{T} \left(\frac{H_1}{2_g} \right)^{\frac{1}{2}}$$

As on rivers we have to deal with mostly the shallow water conditions, there is no change in the height of waves i.e.

$$H_1 = H_o \text{ and } L_o = \frac{gT^2}{2}$$

$$\therefore \frac{H_o}{L_o} \simeq \frac{\tan^2 \alpha}{5.1} \text{ -----(After Iribarren \& Nogales)}$$

The run up (ride) R" of a breaking wave, measured vertically above the mean water surface level, is given by 'HUNT' as

$$\frac{R}{H_1} = \frac{K \tan \alpha}{\frac{8}{T} \left(\frac{H_1}{2g} \right)^{\frac{1}{2}}}$$

Where R = Wave run up (Ride)
H = Wave height
K = Surface roughness co-efficient for the embankment slopes which is usually assumed as 2.3' for smooth facing.

For friction-less slopes, in the case of a surging wave:-

$$\frac{R}{H_1} > 3 \quad \&$$

$$\frac{R}{H_1} = \left(\frac{\pi}{2\alpha}\right)^{\frac{1}{2}} \quad \text{for} \quad \frac{\pi}{4} < \alpha < \frac{\pi}{2}$$

In the case of composite slopes the analysis becomes somewhat more complex but normally we do not have such cases on rivers.

5.2.2 Wind/Wave set up

Tidal movements in inland waters are almost imperceptible the maximum for the Great Lakes being less than 1". However, an appreciable rise in water level may be caused on one shore of a lake, reservoir or pond by wind action particularly in shallow water. For deep water and small area this effect is negligibly small. Such rise above the water levels in the undisturbed conditions of the lake is called 'Set Up'. The best available means of estimating this set up is the ZUI-DER-ZEE-FORMULA reproduced below:-

Set Up $$S = \frac{U^2 F}{1400\ D} \text{Cos 'A'}.$$

Where
S = The Set up in feet. above still pond level.
U = The wind velocity in miles per hour.
F = Fetch in miles.
D = Average depth of water in feet.
A = The angle of wind and fetch.

The effect of this set up when combined with the storm wave action results in the total rise in water level which equals this set plus two third of the maximum height of the storm wave.

5.2.3 River Set

At curves (see Fig. IX) the deepest point of the cross section is near the concave bank and the water surface there (A) is higher than at the convex bank (B). The main current too, hugs the outer or concave bank. The difference in elevation between the water surface at the outer and inner bank caused by the centrifugal force consequent upon the curvature of flow is called the "set" of the river.

A Schoklitsch in his work "Hydraulic Structures". Vol: 1, P_2-151 states:-

$$h = 2.30 \frac{V^2}{g} \times \log \frac{R_2}{R_1}$$

Where V is the velocity at the cross-section, R_1 and R_2 are the radii of curvature of the inneer (convex) and outer (concave) banks respectively, and 'g' is the force due to gravity.

The experience in Sind indicates that 'river set' may cause a "superelevation" of the water surface at the concave bank of as much as 2 ft.

Schoklitsch points out that in the above formula, the "set" is computed on the assumption that the velocity over the entire cross-section is uniform and equal to V "the velocities at the bottom are, however, smaller than V, and the bottom water particles on the concave side are diverted towards the zones of low pressure, i.e, towards the convex shore, by the surface water which flows to the bottom when it impinges on the concave shore. In this way a spiral flow develops scouring the bed on the concave side".

On River Indus at Taunsa Barrage the prevalent wind direction during the monsoon period is from North to South or from North West to South East. All the spur shanks have East-West alignment and length of fetch on the Up Stream is over 10 miles. T.e width of the fetch is also around 10 miles and thus the fetch is not effected by the width. The depth of water during high floods ranges between 8 to 12 ft.

The record of wind storm data for Taunsa Barrage for the monsoon period of 11 years from 1967-77 was collected and appears as Table 1 to 4.

From this data it is established that severe and very severe intensity wind storms are experienced generally during the month of July and the duration varies from 30 minutes to 4 Hrs. During such storms wind velocities of the order of 50 miles per hr or more are not uncommon.

Analyses for wave parameters free board and Beach slope follow, wherein the existing provisions have been compared with the computed values.

6.0 ANALYSIS FOR WAVE PARAMETERS AGAINST EMBANKMENTS

At Taunsa Barrage (River Indus)

Available Data:

Fetch (F) = 10 Miles
Wind Velocity (U) = 50 miles/hr
Average Depth of Water (h) = 10 feet

Duration of Storms = 4 hours.

i) From Creager and Justin Formula:-

$$\text{Wave Height (H)} = \frac{F^{0.37} \times U^{0.48}}{3.41}$$

$$= \frac{(10)^{0.37} \times (50)^{0.48}}{3.41} = 4.5'$$

ii) From Bretschneider Formula:-

$$H_s = 0.0555(U^2\ F)^{0.5}$$

$$= 0.0555\left[\left(\frac{50}{1.15}\right)^2 \times \frac{10}{1.15}\right]^{0.5}$$

$$= 0.0555\left[(43.5)^2 \times 8.7\right]^{0.5}$$

$$= 0.0555(16400)^{0.5} = 7.1 \quad \text{(For deep water)}$$

$\therefore$ H for Shallow water $= H_s \times \frac{65}{100} = \frac{7.1 \times 65}{100} = 4.6'$

Period $(T_s) = 0.5\ (U^2\ F)^{0.25} = 5.65$ Sec.

$\therefore$ Frequency (f) = 10.8 $\approx$ 11 c/s

iii) From U.S.B.R.Table in "Design of Small Dams":-

H = 4.5'

iv) From Fig (VI) for shallow water.

$h_s = 3.5'$ -

correcting for duration of storm - $H_{max} = 3.5 \times 1.3 = 4.55'$

& T = 4.7 sec. or Frequency (f) = 13 c/s

6.1 ANALYSIS FOR BEACH SLOPE FOR EMBANKMENTS

Taunsa Barrage (River Indus)

from IDFCR Council Formula

$$S_f = 1.59 \left(\frac{f}{100} \times S_i \times \frac{H}{L} \times \frac{MD_o}{L}\right)^{0.2}$$

where $f = 12$

$S_i = 1/3$

$H = 4.5$

$L = 5.12 \times T^2 = 5.12 \times 25 = 128$ feet

MD_o = median size of particle (in mm) = 0.17 mm

$$\therefore S_f = 1.59 \left(\frac{12}{100} \times 1/3 \times \frac{4.5}{128} \times \frac{0.17}{128}\right)^{0.2}$$

$$= 1.59 \left(\frac{0.153}{81200}\right)^{0.2} = 1.59 \left(\frac{1}{532000}\right)^{0.2}$$

$$= \frac{1.59}{10 \times 1.4} = \frac{1.59}{14} = \frac{1}{8.8}$$

i.e. 1 in 8.8

But actual observed on Ist Defence Bund at Taunsa Barrage = 1 in 12 1968 - 69

Hence some more data from prototypes must be analysed to arrive at a firm standard. On Balochwah Bund on Left Side of River Chenab, the Beach Slope after 1976 floods was observed to be 1 in 10 under similar conditions.

This has better relationship with the above figure calculated from IDFCR formula.

It thus transpires that the wave action can be successfully counter-acted by providing, a front slope in conformity with the stable beach slope, adequate free board as shown in analysis, and other protective measures. It may not be possible in so many places to construct the embankment with such mild slopes and some suitable protective measures will have to be adopted.

6.2 ANALYSIS FOR FREE BOARD OF EMBANKMENTS

Taunsa Barrage (River Indus)

i) Wave Run Up:-

Available Data:-

$H = 4.5$ ft.

(T) Period = 5 Sec.

$f = 12$/minute

Depth = 10 ft.

Front face slope = 1 in 3

$$\text{Tan}\ \alpha = \frac{8}{T}\left(\frac{H_t}{2g}\right)^{0.5}$$

$$= \frac{8}{5}\left(\frac{4.5}{64.4}\right)^{0.5} = \frac{8}{5}\left(\frac{1}{14.35}\right)^{0.5}$$

$$= \frac{1.6}{3.8} = 0.42$$

i.e. Slope required for Breaking = 1 in 2.4

but actual slope = 1 in 3

Hence breaking is ensured.

Now $$\frac{R}{H_1} = \frac{K \tan\alpha}{\frac{8}{T}\left(\frac{H_1}{28}\right)^{0.5}}$$ Where K = 2.3 for a smooth surface Adopted -1.8 for earthen slope

$$\therefore R = \frac{4.5 \times 1.8 \times 1 \times 3.8}{1.6 \times 1 \times 3} = 6.4'$$

Neglecting wind set and River set which can raise the Free board requirement by a minimum of 2.0'

The Free Board should be = 6.4 + 1 = 7.4
Say 7.5 ft.

This is confirmed from Fig. XII from "Design of Small Dams" by U.S.B.R

But free board actually provided is 5.0' which is too insufficient.

This is also confirmed from personal experience in 1967-71 that during very severe wind storms the waves were found riding not only to crest but breaking splash would also go over the embankment and cause damage to land side slopes in the form of local flow cuts. But for small duration of storms, serious mishaps could have occured.

This calls for revision of existing free board design from 5 feet to 7.5 feet, similarly the analysis for Trimmu on River Chenab and left embankments in Multan and Shujabad Tehsil indicates the necessity for revision of free board standard of five feet to seven feet.

7.0 PROTECTION OF UP STREAM SLOPE

One of the major causes of failure of earthen Bunds is the damage caused by wave action,during floods when the embankments hold substantial depth of spill water. Protection of the Up Stream slope,therefore, becomes necessary for safety of the embankment. The methods employed for protection of Up Stream slope are introduced in the following paragraphs:-

7.1 Temporary (Limited Duration) Measures

7.1.1 Khaji Mats.

Khaji Mats are stitched together with 6 inches over laps and the large mat so prepared is laid on the bank slope and anchored in place with small stakes. Figure XIII (A). This method is used mainly during emergency and un-expected storms as delaying measure only and can at best simply reduce the damages.Some times gunny bags filled with earth are also used in extreme emergency.

7.1.2 Fascine Covering

Pilchi, Sarkanda or brush wood is woven in the form of a mat and this mat is then laid on the bank slope and anchored with rope and stakes. Figure XIII(A). This is quite a cheap protection but can only reduce the damages. Its life is also limited and it losses its usefulness just after one or two storms.

7.1.3 Pilchi Rolls

Pilchi, Sarkanda or Brush Wood is tied into around bundles with diameter varying between 1 ft. to 2½ ft, and these rolls are placed on the slope at water level parallel to the embankment and anchored with timber stakes. Figures XIII (A). This type is more useful than the earlier two alternatives but is more costly and lasts only for a few storms of moderate wind velocities. The author has seen such rolls broken and churned up into pulp due to pounding action of waves during high velocity storms.

7.1.4 Longitudinal Stakes and Bushing protection

This consists of one or two rows of stakes with brush wood or pilchi rolls of 6 inches dia whettled in between. Figure XIII (A). In sind province this type is termed as MUHARI.

7.1.5 Pilchi pitching (Revetment)

This is a semi permanent type of protection and is widely used. Figure XIII (A). However, the extent of protection afforded to the embankment slope sometimes does not justify the cost. It also gets damaged,(most of the time irrepairably) under intense action and usually losses its usefulness in about 2 years time.

7.2 PERMANENT MEASURES

7.2.1 Brick pitching

Brick pitching with brick laid on end

over a flat course has been tried at some places. Figure XIII(A). It deteriorates with time and lets the water seep through to earth backing. This creates hydrostatic pressure behind the brick pitching which results in buldging and finally spawling off and specially so when the spill depth reduces rapidly (Quick Draw Down Case).

A similar protection was provided to 2nd defence bund along right bank of D.G.Khan Canal at Taunsa Barrage, but it collapsed in the very first season, it had to face the full fury of wave action during monsoon of 1967, and was later replaced with Stone pitching.

The Brick pitching also has a disadvantage that the wave ride increases due to smoothness of the pitched surface.

7.2.2 Dumped Stone Rip-Rap

It is one of the best methods for protection of water face slopes. It consists of angular Stone dumped randomly over a properly placed graded filter consisting of rock material ranging between $\frac{3}{16}$ inch to $3\frac{1}{2}$ inch ring. Figure XIII (B). The efficacy of this type depends on:-

1) Quality of Stone
2) Weight, Size and shape of individual Stones. Angular but not flaky,pieces are better than rounded boulders.
3) Thickness of the protection and the slope on which laid.
4) Behaviour of the filter behind the dumped Stone.

The individual stones must be of sufficient weight to resist displacement by wave action which is not necessarily a function of the height of the embankment. It is wrong to suppose that large size stones are needed only on higher structures, while smaller size will afford enough protection for low embankment without considering the wind velocity, direction, depth and the fetch causing wave action. The required weight of the individual pieces can be determined theoritically by methods given in 13 which provide that the force exerted on Rip-Rap Stone cannot be greater than that of a current flowing at velocity equal to that of the particles in the wave. The stone sizes so calculated compare favourably with experience on prototypes studied by U.S.Bureau of Reclamation. The gradation limits of Rip-Rap Stone on 3:1 slope is given in the following table used by USBR:-

THICKNESS & GRADATION LIMITS OF RIPRAP ON 3:1 SLOPE

Reservoir fetch (miles)	Normal thick-ness (inches)	Maxi-mum size	Gradation, percentage of stones of Various weights (Pounds)		
			40 to 50 percent grea-ter than	50 to 60 : 0 percent From- to-	: -10 percent less than
2.5 & less	30	2,500	1,250	75-1250	75
More than 2.5	36	4,500	2,250	100-2250	100

Note:- 1) Sand and rock shall be less than 5 percent by weight of the total riprap material.

2) The percentage of this size material shall not exceed an amount which will fill the voids in larger rock.

Sherad et al in their publication Earth and rock dams (19) recommend the following data for rip-rap and filter beneath,

RECOMMENDED RIPRAP DESIGN CRITERIA

Maximum wave height (feet)	Minimum Average Rock size (D_{50}) (inches)	Layer thickness (inches)
1	2	3
0-2	10	12
2-4	12	18

1	2	3
4-6	15	24
6-8	18	30
8-10	21	36

MINIMUM THICKNESS OF SINGLE LAYER FILTER UNDER RIPRAP BLANKETS

Computed wave height (feet)	Minimum Filter Thickness (inches)
0-4	6
4-8	9
8-10	12

Dumped rip-rap is, however, very costly and should be provided where absolutely necessary.

7.2.3 Hand placed Rip-Rap (Stone pitching)

Stone pitching consists of Stone Laid by hand and packed properly over a graded filter similar to the one required for dumped rip-rap. The usual thickness adopted varies between 1.5 ft. to 3 ft, 2 ft. is the usual average adopted. The usual thickness of filter is 0.7 ft: and the balance 1.3' is occupied by top Layer of large stones. Figure XIII (B).This is quite effective in protecting the earthen slope but calls for a larger free board due to increased wave run-up on account of smoother surface.

7.2.4 Soil Cement Cover

Of late, soil Cement is proving to be an economical facing material for embankments where the cost of Stone protection is too high. A reasonable firm foundation is required so that deformation after placement is insignificant. It is generally placed and compacted in horizontal step-like layers of 8 ft: width. Each succeeding layer is stepped back by a distance equal to the product of the compacted layer thickness (Usually six inches) and the embankment slope. (Figure XIII (B).Soil containing 10 to 25% material passing No.200 sieve (ASTM) can be safely used by adding one barrel of cement per Cyd. of compacted layer.

7.2.5 Cement Concrete Paving

Concrete paving can be used successfully where the Sub Grade is suitable and settlement in the body or foundation of the embankment can be taken as negligible. Figures XIII (B) Monolithic pavement behaves better than that laid in the form of pannels, with expansion joints in between. The water finds its way through the joints to the back of the pavment and damages it through hydro-static pressure. Settlement in the Sub Grade also results in cracks and similar failure. Usually a six inches layer suffices if the Sub Grade is firm. In case of panelled construction, all joints have to be sealed and panels reinforced. In such cases adequate provision for back drainage must also be made.

7.2.6 Asphaltic Concrete

Asphaltic concrete being comparatively flexible than cement concrete is a better substitute, but in this case also provision for back drainage is a must to eliminate up lift in case of rapid draw down. The usual thickness of asphaltic concrete is taken as 9 to 12 inches. Figures XIII (B).

7.2.7 Porous Concrete slab

Porous concrete has also been in use for quite sometime.When properly made it is comparatively strong and almost equally pervious as crushed stone protection. It consists of coarse aggregate 3/4 inch to ¼ inch with one barrel of cement per Cd. This is best suited for mild climates and where stone rip-rap is excessively expensive. Figure XIII (B).

8.0 CONCLUSIONS

The following conclusions are drawn:-

1) The existing criteria for river face slopes of earthen embankments need revision. We may either construct the Water face slopes in conformity with stable beach slope or the existing slopes may be provided with proper protection.

The slope protection measures may be executed in two stages:

i) Trees with good foliage and medium height, or other plants like Gulabasi may be grown in a thick belt in front of the embankments at a distance of 10 ft from the toe, but Gulabasi if grown or other bushes may not be allowed to spread on to the slopes.

ii) Permanent protection on Up Stream slope in the form of stone pitching with proper filter or a layer of cement concrete (1:3:6) 6" thick may be provided but in case of concrete the free board will have to be increased by 0.5 ft specially on account of reduction in friction co-efficient of slope protection and consequent increase in wave run up.

The existing criterion for free board may also have to be revised. The following standards are recommended:-

a) On River Indus = 7.5 feet
b) On River Chenab and Ravi = 7.0 feet
c) On River Jhelum & Sutlej = 6.0 feet

It is hoped that the above recommendations will find favour with Engineers concerned and will go a long way in ensuring the safety of Earthen embankments against wave action.

Paper No. 436

TABLE-1

STATEMENT SHOWING WIND STORMS DATA FOR THE MONSOON PERIOD OF 1967 AT TAUNSA BARRAGE

Sr. No	Date	Intensity	Direction	Duration From	Duration To	Hrs	Mts	Remarks
1.	3.5.67	Severe	North to South	19.00	20.00	1	00	
2.	17.7.67 to 18.7.67	Severe	North to South	23.30	02.30	3	00	
3.	24.7.67	Moderate	N-W to S-E	18.00	22.00	4	00	
4.	26.7.67	Moderate	North to South	08.30	11.30	3	00	
5.	30.7.67	Severe	N-E to S-W	14.30	15.30	1	30	
6.	8.8.67	Severe	North to South	16.00	17.30	1	30	
7.	25.8.67	Severe	North to South	17.30	18.30	1	00	
	Year 1968							
1.	25.5.68	Severe	North to South	17.00	22.30	5	30	
2.	27.6.68	Severe	North to South	15.00	16.00	1	00	
3.	9/10.7.68	Severe	N-W to S-E	23.00	02.00	3	00	
4.	23.7.68	Severe with rain	N-W to S-E	00.30	04.30	4	00	
	Year 1969							
1.	14.6.69	Severe	West to East	17.00	19.00	2	00	
2.	18.6.69	Severe	North to South	17.00	18.30	1	30	
3.	1.7.69	Severe	West to East	15.30	16.15	0	45	
4.	14.7.69	Severe	East to West	17.30	18.00	0	30	

TABLE-2

STATEMENT SHOWING WIND STORM DATA FOR THE MONSOON PERIOD OF 1970 AT TAUNSA BARRAGE

Sr. No.	D a t e	Intensity	Direction	Duration. From	To	Hrs	Mts	Remarks
1.	27.4.70	Moderate	North to South	07.00	14.00	7	00	
2.	5.5.70	Severe	North to South	10.00	19.00	3	00	
3.	8.5.70	Severe	North to South	17.30	21.00	3	30	
4.	29.5.70	Very Severe	N-E to S-W	16.00	19.30	3	30	
5.	2.6.70	Severe	North to South	16.40	19.50	3	10	
6.	2.7.70	Very Severe	N-W to S-E	17.00	19.00	2	00	
7.	22.7.70	Severe	North to South	05.00	06.00	1	00	
8.	4.9.70	Severe	North to South	19.00	20.00	1	00	

Year 1971

Sr. No.	D a t e	Intensity	Direction	Duration. From	To	Hrs	Mts	Remarks
1.	20.5.71	Severe	N-E to S-W	21.30	23.30	2	00	
2.	27.5.71	Moderate	N-W to S-E	22.00	03.00	5	00	
3.	30.5.71	Severe	N-W to S-E	22.00	23.30	1	30	
4.	31.5.71	Severe	N-W to S-E	22.00	01.00	3	00	
5.	15.6.71	Moderate	N-E to S-W	19.00	24.00	5	00	
6.	19.6.71	Severe	West to East	20.00	21.00	1	00	
7.	24.6.71	Severe	N-E to S-W	19.00	20.00	1	00	
8.	27.6.71	Severe	North to South	18.00	20.00	2	00	
9.	4.7.71	Severe	North to South	18.00	20.00	2	00	
10.	18.7.71	Severe	N-W to S-E	20.00	21.00	1	00	

Year 1972

Sr. No.	D a t e	Intensity	Direction	Duration. From	To	Hrs	Mts	Remarks
1.	5.6.72	Severe	N-W to S-E	04.30	06.30	2	00	
2.	5.5.72	Severe	North to South	05.00	11.00	6	00	
3.	11.7.72	Severe	N-W to S-E	05.00	07.00	2	00	

TABLE-3

STATEMENT SHOWING WIND STORM DATA FOR THE MONSOON PERIOD OF 1973 AT TAUNSA BARRAGE

Sr. No.	Date	Intensity	Direction	Duration From	Duration To	Hrs	Mts	Remarks
1.	30.5.73	Severe	North to South	18.30	20.00	1	30	
2.	5.7.73	Severe	North to South	12.05	16.00	3	55	
3.	3.9.73	Severe	N-E to S-E	14.10	16.00	1	50	
			Year 1974					
1.	11.6.74	Severe	S-E to N-W	15.40	16.10	0	30	
2.	22.6.74	Severe	N-E to S-W	19.00	23.00	4	00	
3.	3.7.74	Severe	N-E to S-W	18.45	20.45	2	45	
4.	9.7.74	Severe	West to East	18.45	19.20	0	35	
5.	10.7.74	Severe	West to East	19.30	20.15	0	45	
6.	11.7.74	Severe	West to East	01.30	02.30	1	00	
			Year 1975					
1.	21.4.75	Severe	N-W to S-E	07.45	09.00	1	15	
2.	24.4.75	Severe	N-W to S-E	17.30	18.15	0	45	
3.	1.6.75	Severe	N-W to S-E	16.10	17.30	1	20	
4.	3.6.75	Severe	N-W to S-E	16.50	18.10	1	20	
5.	7.6.75	Severe	N-W to S-E	10.50	12.00	1	10	
6.	21.7.75	Severe	N-W to S-E	14.30	15.00	0	30	
7.	22.7.75	Severe	N-W to S-E	22.00	23.00	1	00	
8.	23.7.75	Severe	N-W to S-E	9.15	10.00	0	45	
9.	20.9.75	Severe	N-W to S-E (again E to W)	20.10	21.40	1	30	

TABLE-4

STATEMENT SHOWING WIND STORMS DATA FOR THE MONSOON PERIOD OF 1976 AT TAUNSA BARRAGE

Sr. No.	Date	Intensity	Direction	Duration From	Duration To	Hrs	Mts	Remarks
1.	6.5.76	Severe	N-W to S-E	17.45	19.00	1	45	
2.	19.6.76	Severe	N-E to S-W	03.00	0[illegible].30	0	30	
3.	30.7.76	Very Severe	N-E to S-W	22.30	00.30	2	00	
4.	12.8.76	Severe	N-E to S-E	20.00	22.30	2	30	

Year 1977

Sr. No.	Date	Intensity	Direction	Duration From	Duration To	Hrs	Mts	Remarks
1.	30.4.77	Severe	North to South	16.30	17.30	1	00	
2.	2.5.77	Severe	N-E to S-W	18.30	19.00	0	30	
3.	10.5.77	Severe	N-E to S-W	21.00	22.00	1	00	
4.	13.5.77	Severe	N-E to S-W	17.00	18.00	1	00	
5.	23.5.77	Severe	S-W to N-E	17.10	19.55	2	45	
6.	8.6.77	Moderate	North to South	17.30	20.00	2	30	
7.	16.6.77	Moderate	S-W to N-E	19.00	19.15	0	15	
8.	8.7.77	Severe	North to South	12.00	13.00	1	00	
9.	29.8.77	Medium	North to South	19.45	20.30	0	45	
10.	23.9.77	Very Severe	North-W to S-E	17.10	19.00	1	30	

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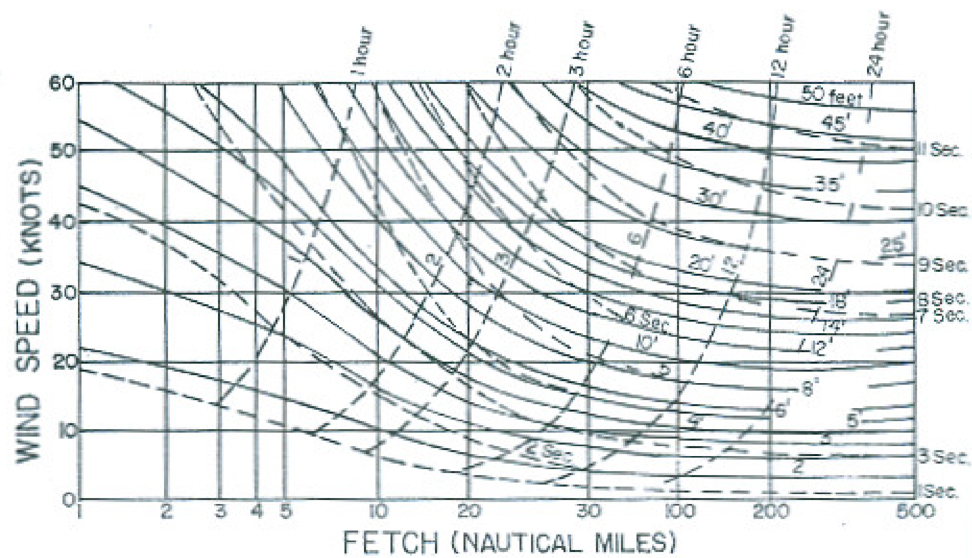
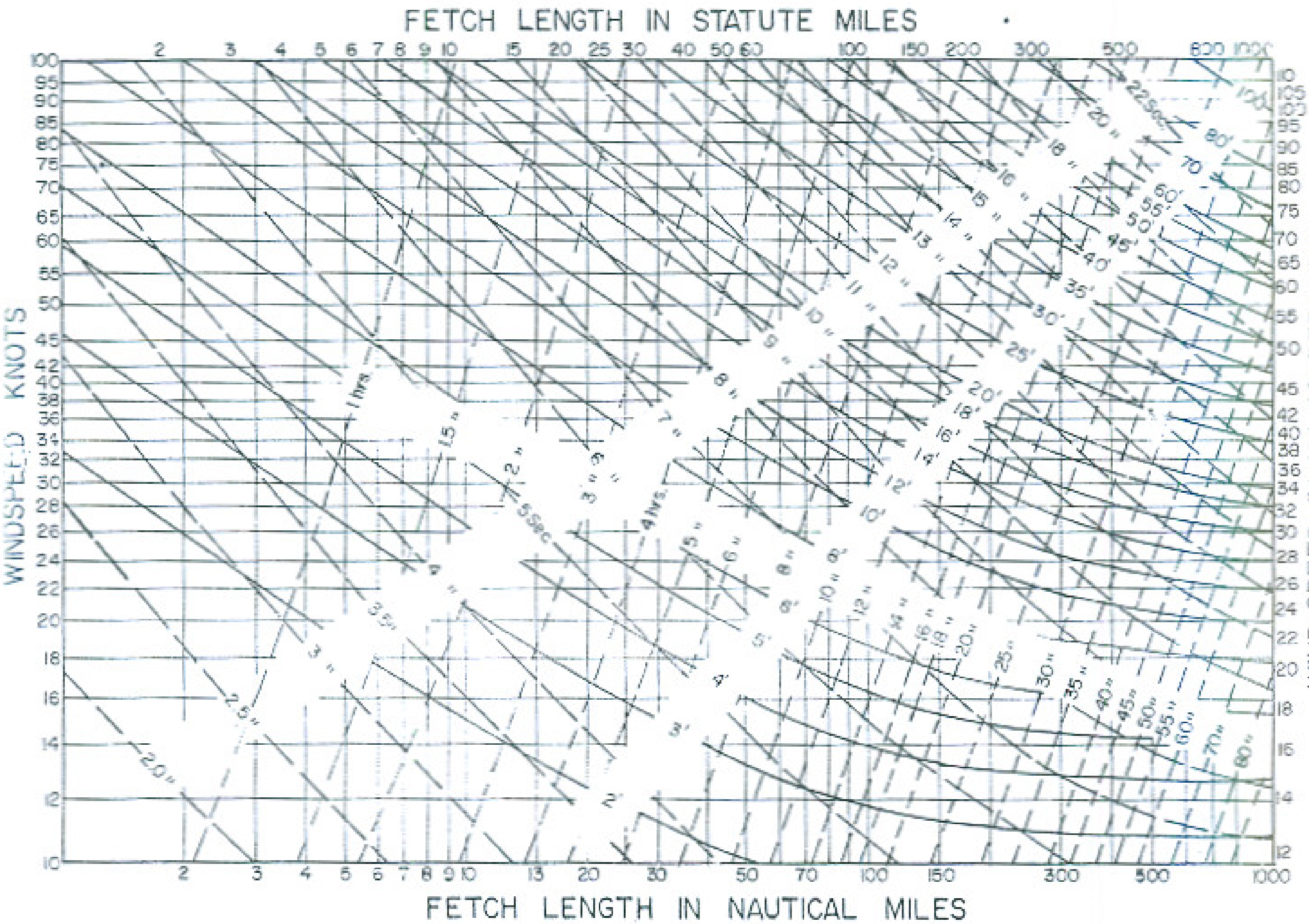


WAVE HEIGHTS AND PERIODS IN COASTAL WATERS
(AFTER DARBYSHIRE & DRAPER)



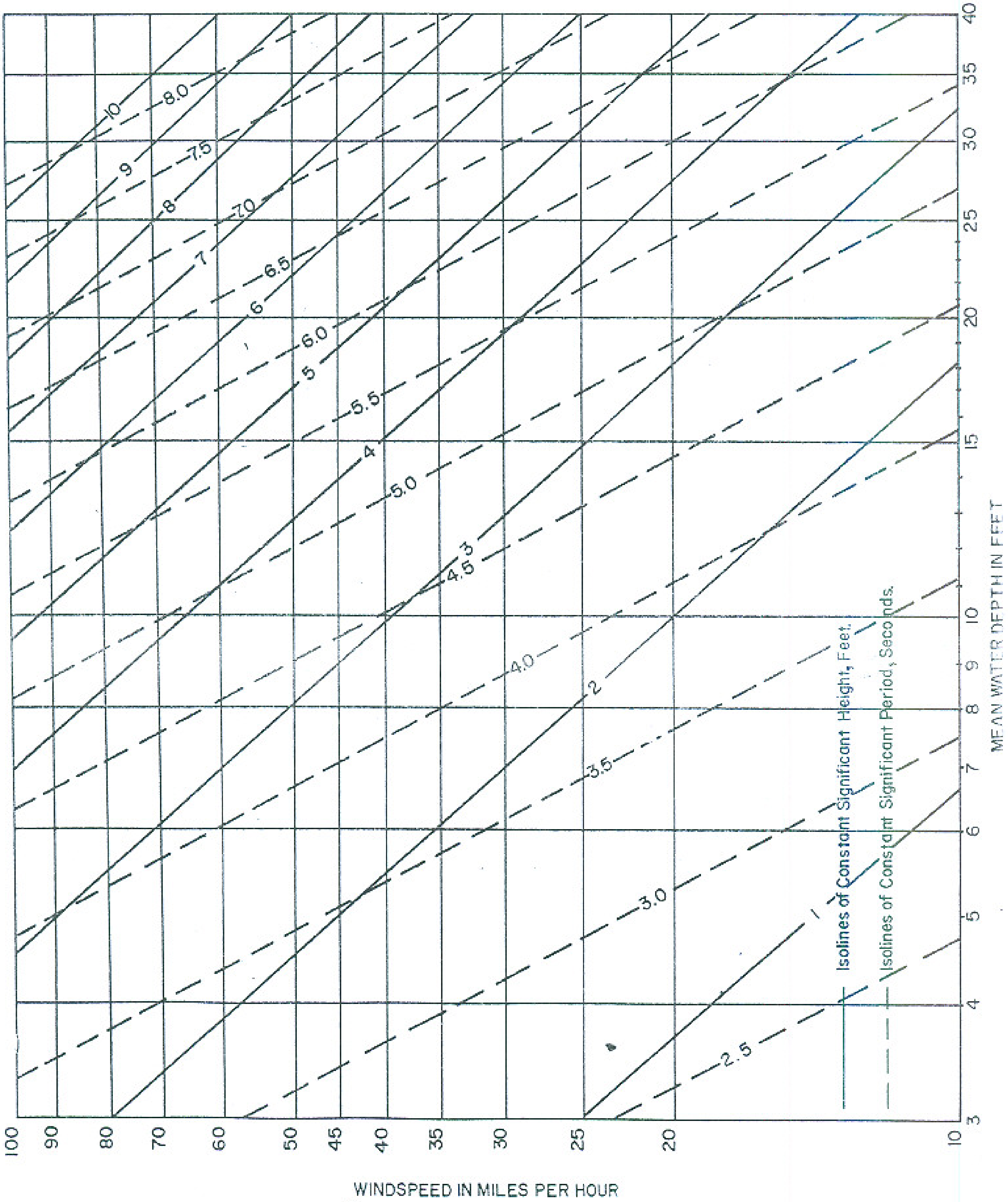
——— Signifies distance in feet — — — Signifies period in seconds - - - - Min. duration in hours

DEEP WATER WAVE FORECASTING CURVES AS A FUNCTION OF WIND SPEED, FETCH LENGTH AND WIND DURATION

(Reproduced from "Estuary and Coastline Hydrodynamics" by A.T. Ippen)

FIGURE-VI

PAPER NO. 436



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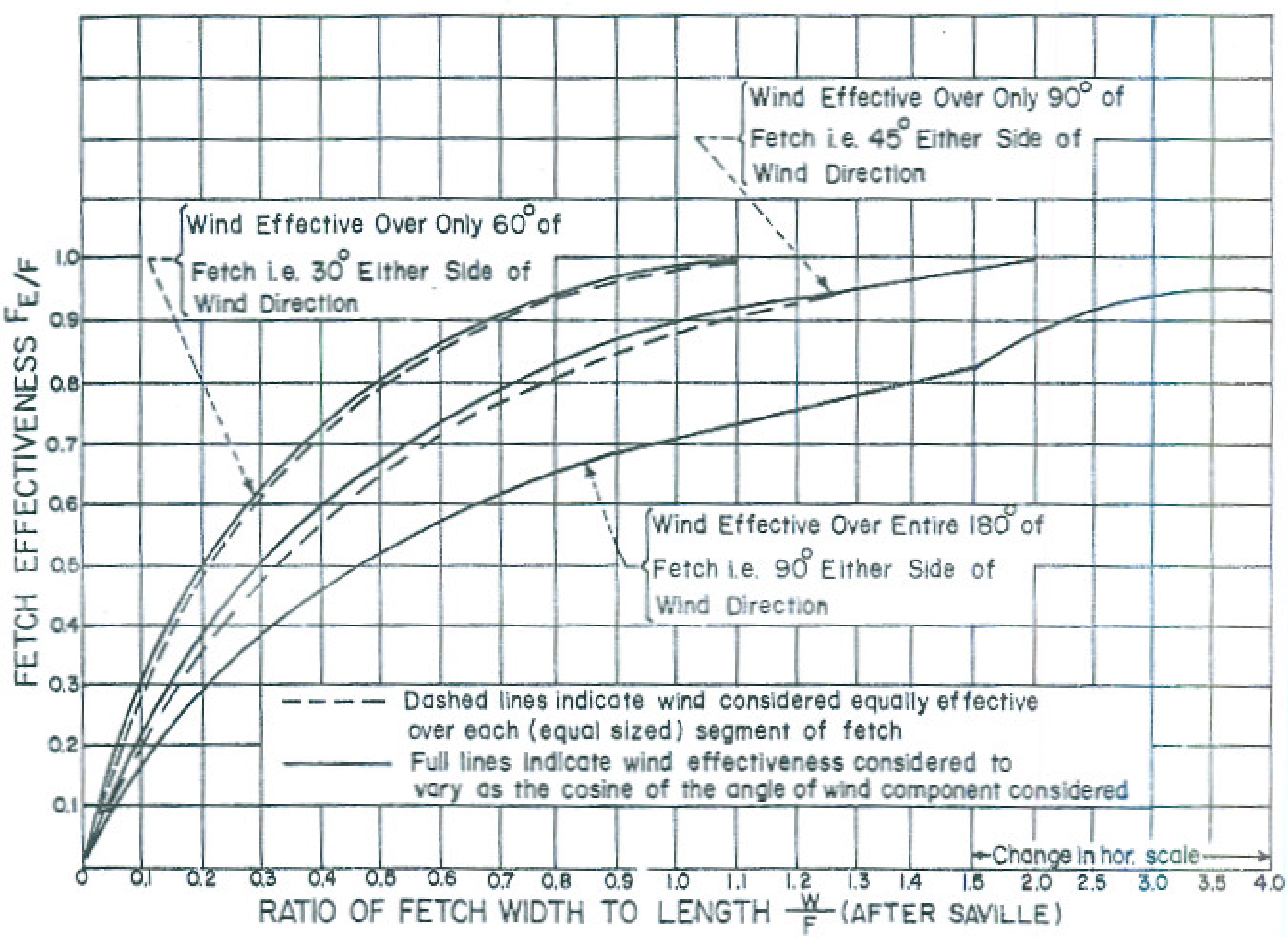
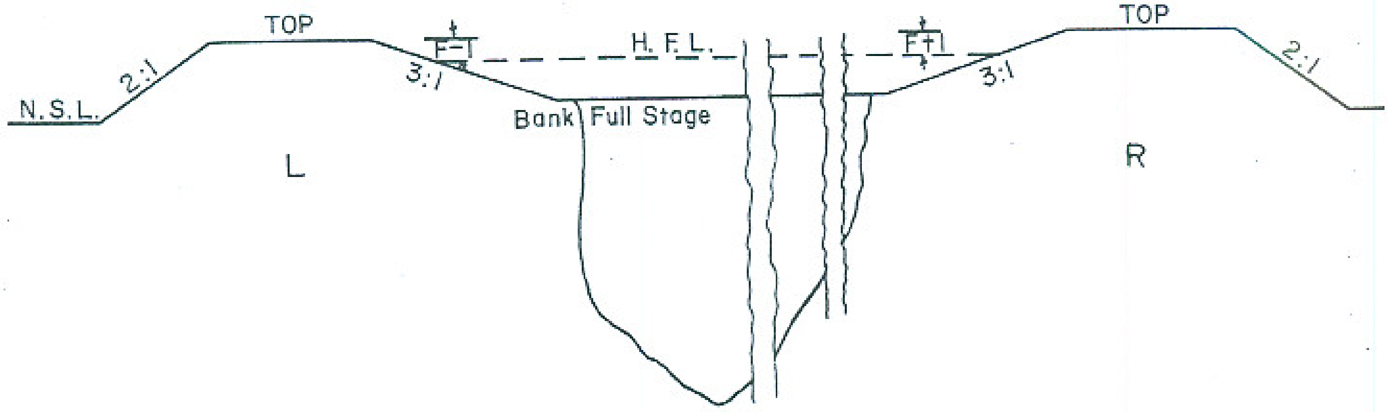
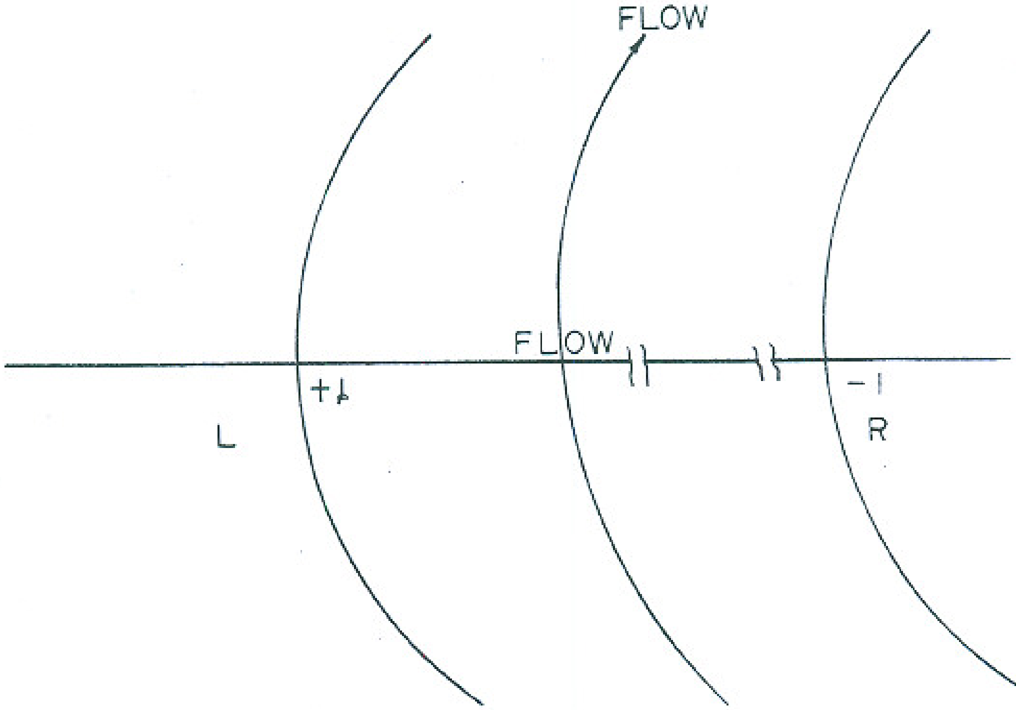


RELATION OF EFFECTIVE FETCH TO WIDTH–LENGTH RATIO FOR RECTANGULAR FETCHES

FIGURE - IX
PAPER NO. 436



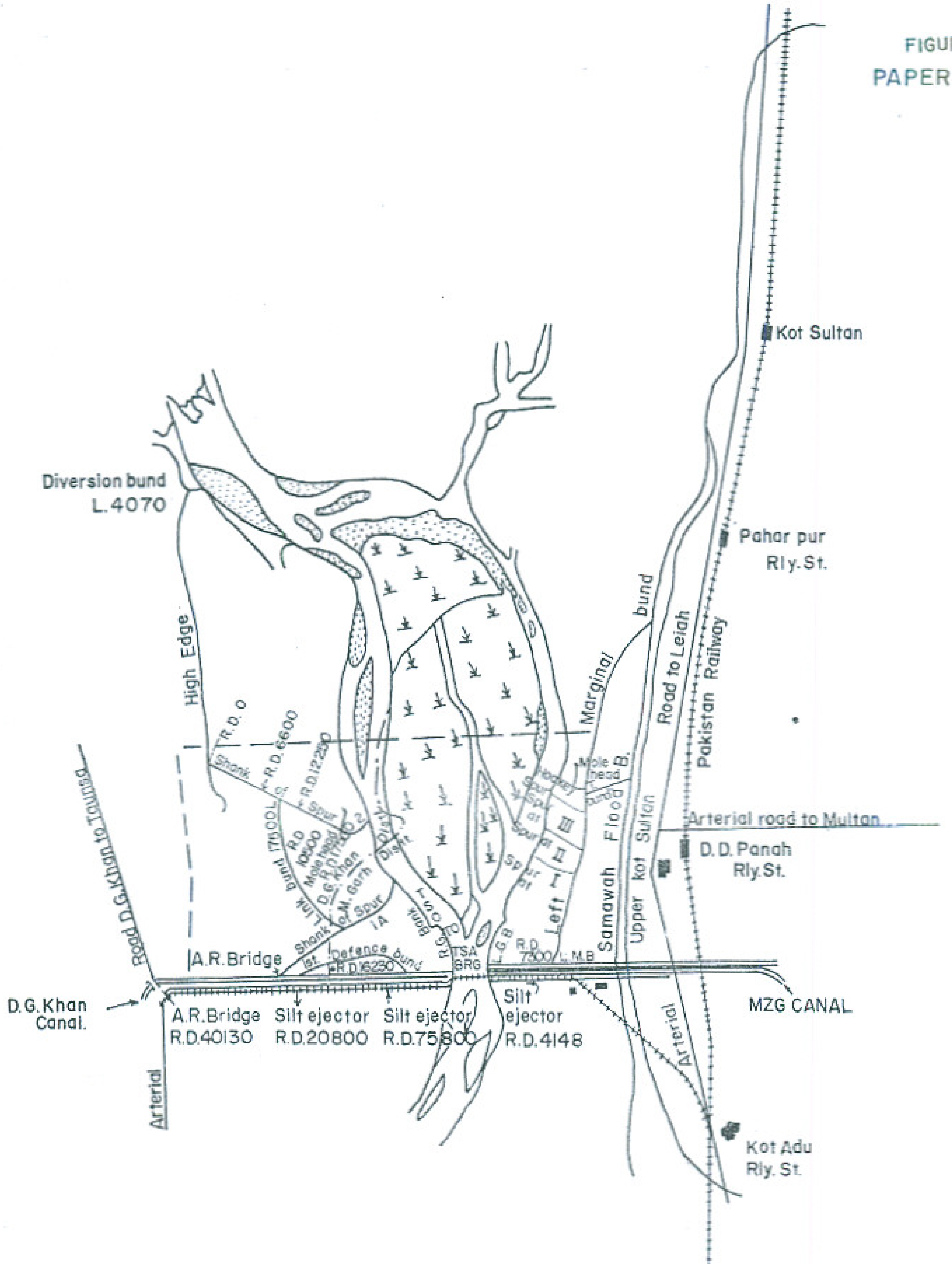
PLAN



RIVER SECTION DEPICTING CURVED COURSE & SET

FIGURE-X

PAPER NO.436



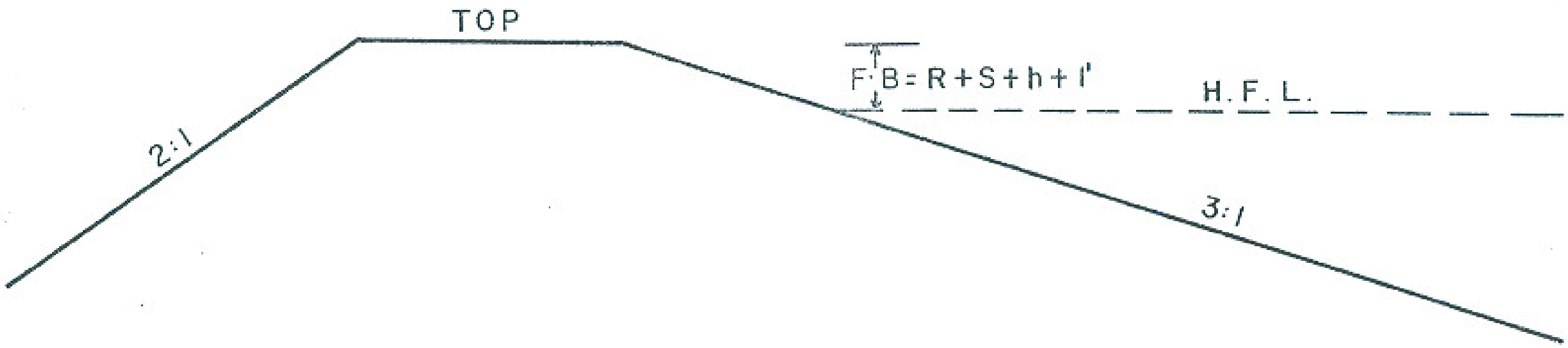
INDEX PLAN

OF

TAUNSA BARRAGE

(SCALE = 1/4" = 1 MILE)

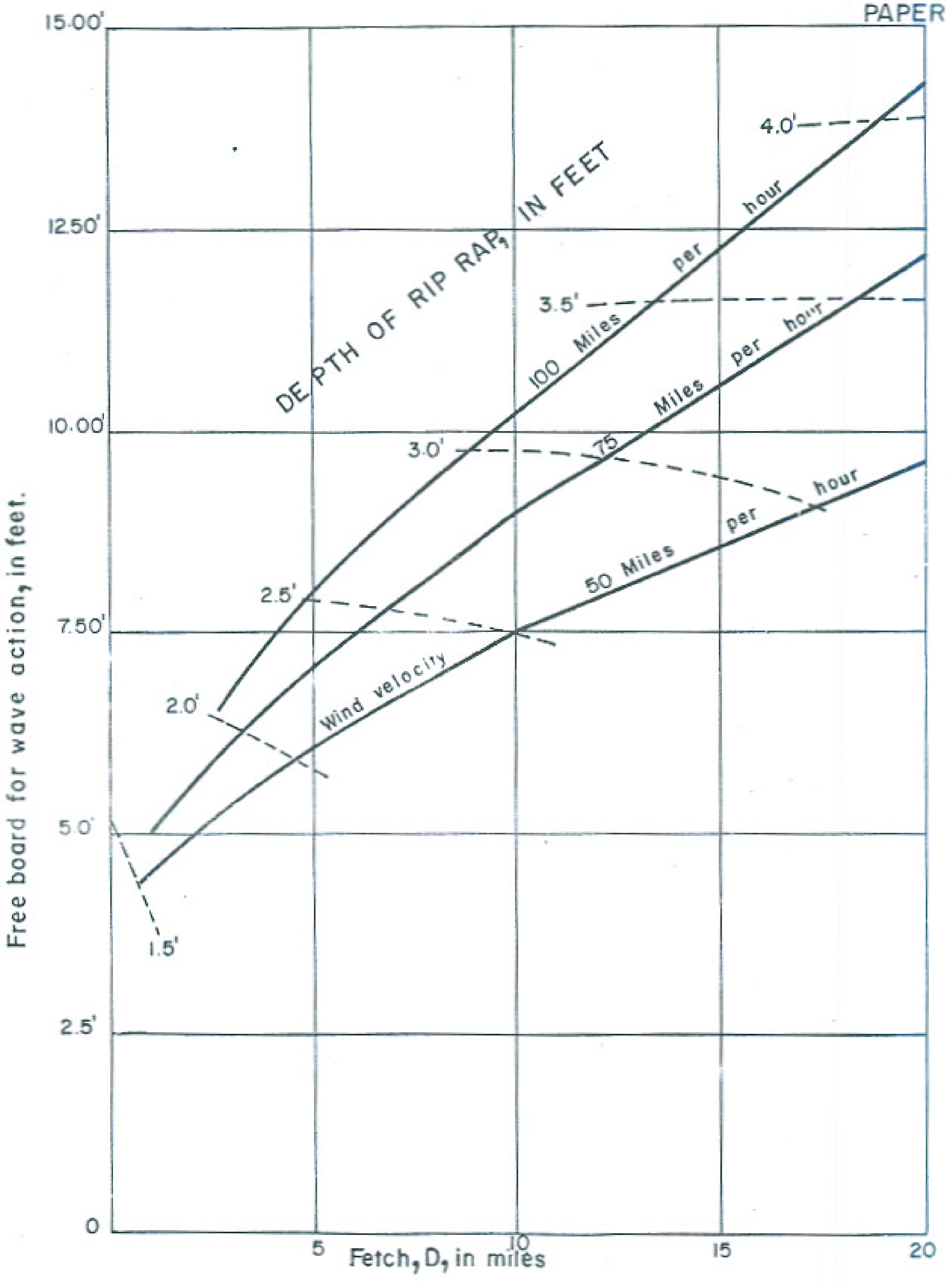
FIGURE-XI
PAPER NO.436



F= FREE BOARD
R= WAVE RIDE
S= WAVE SET
h= RIVE SET
1' FACTOR OF SAIFTY

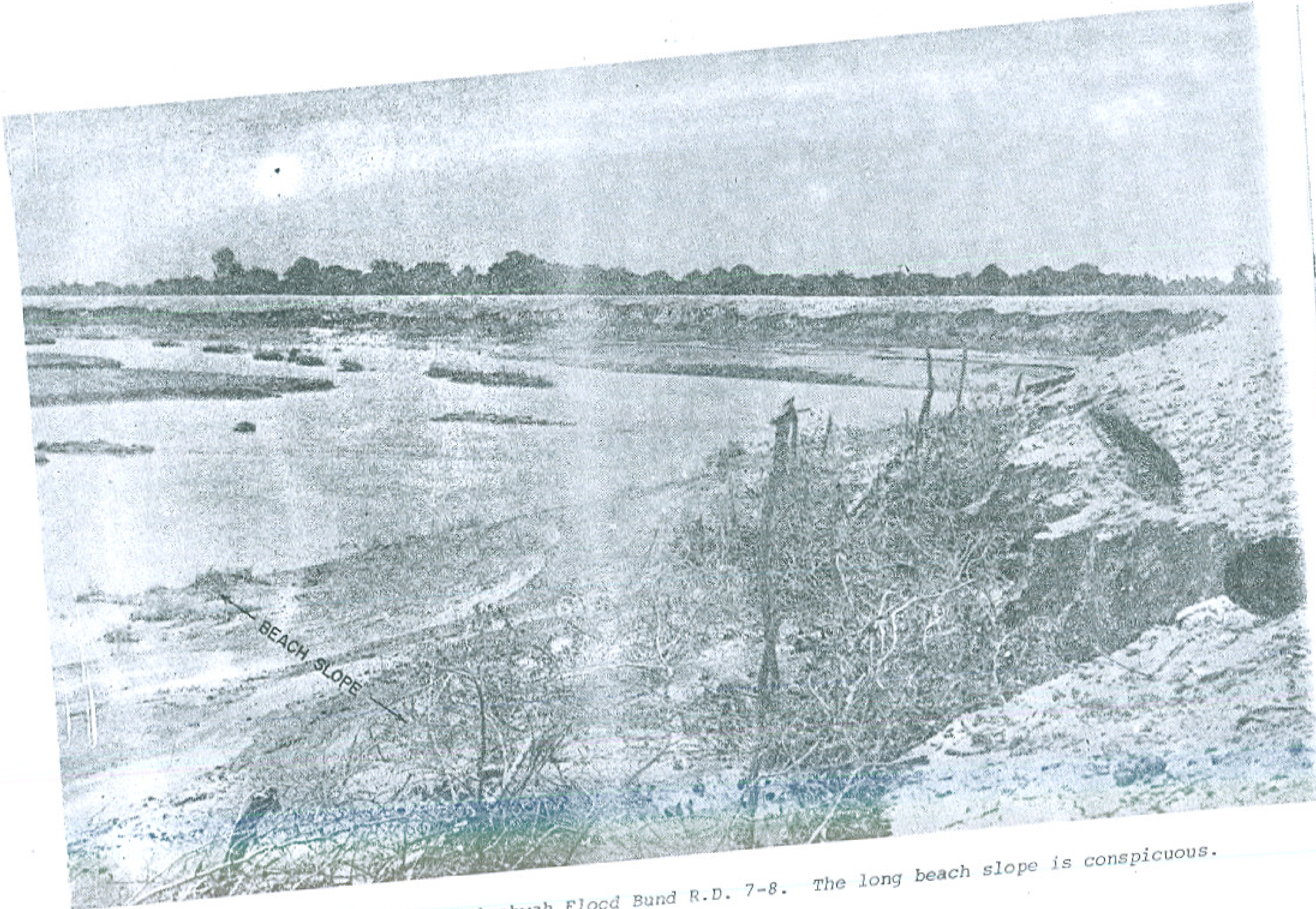
DESIGN OF FREE BOARD

FIGURE-XII
PAPER NO.436



RELATION SHIP BETWEEN FETCH BOARD FOR WAVE ACTION, AND THICKNESS OF DUMPED RIP RAP FOR SLOPE PROTECTION.

(Reproduced from hand book of applied hydraulics C.V. DAVIS)



Wave damages floods 1976, Balochwah Flocd Bund R.D. 7-8. The long beach slope is conspicuous.

FIGURE- XIII (B)
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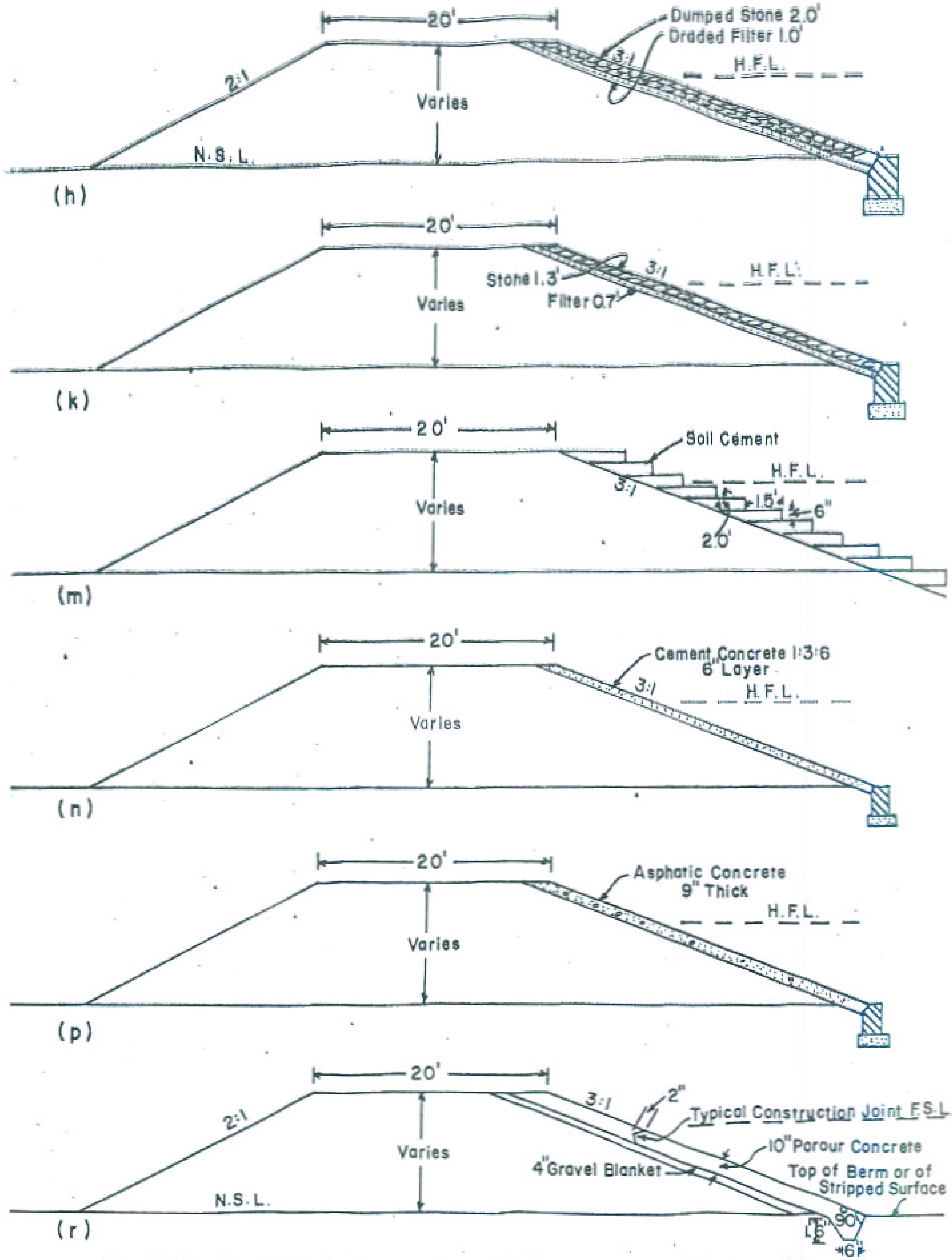
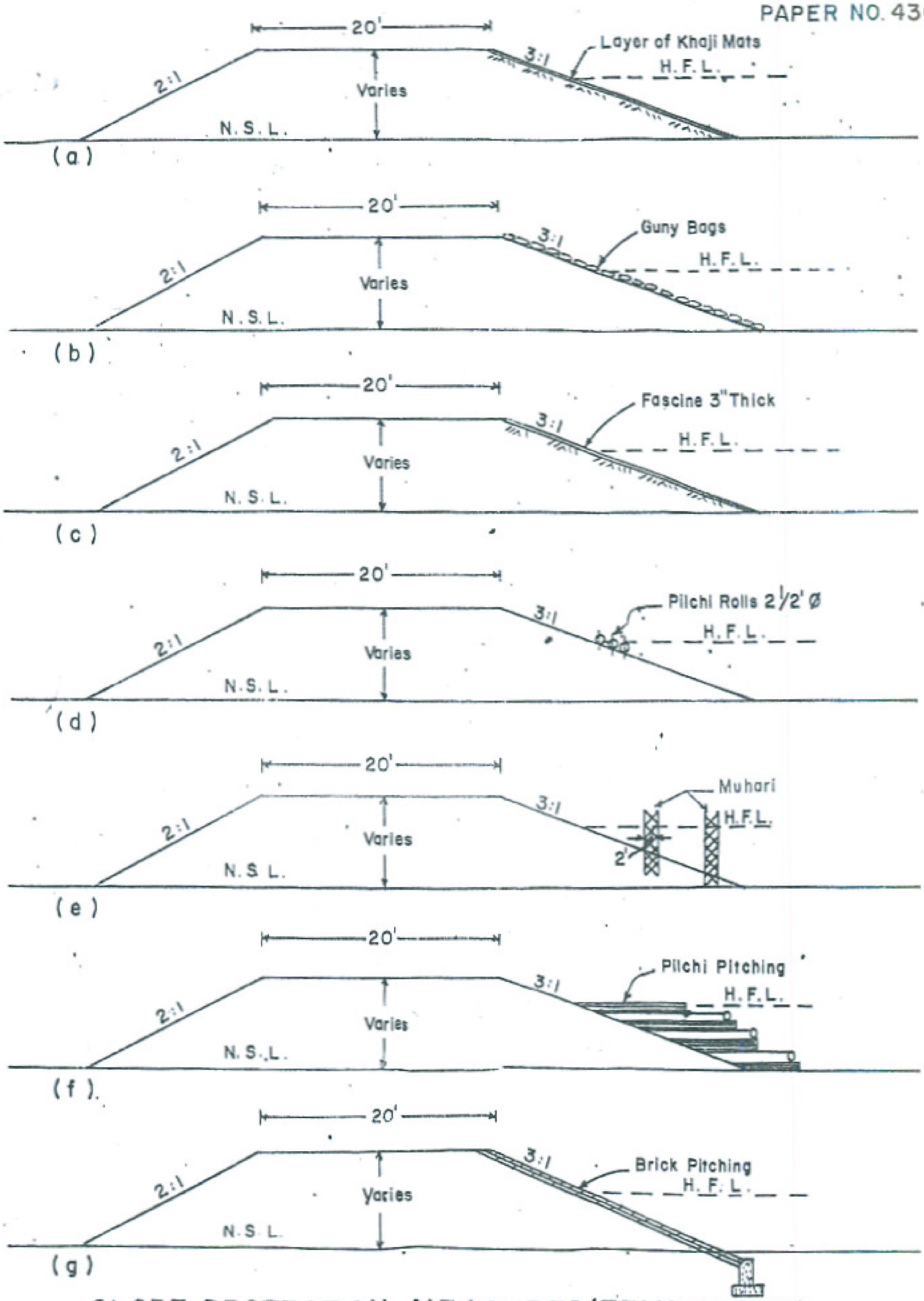
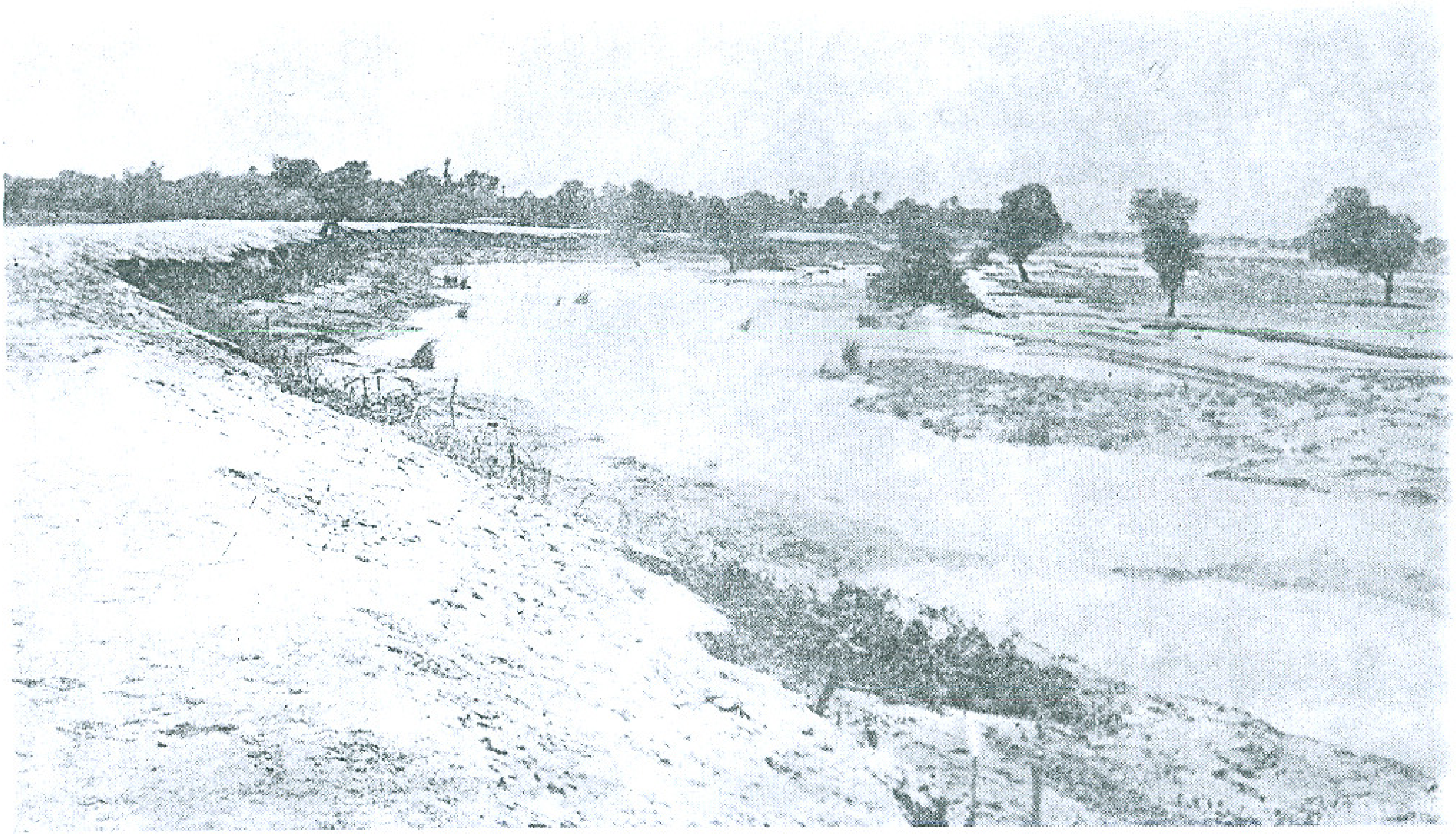


SLOPE PROTECTION MEASURES (PERMANENT)



SLOPE PROTECTION MEASURES (TEMPORARY)

Balochwah Flood Bund R.D.10-11 after Floods 1976. See what it has gone through in spite of permeable Bushing protection



Balochwah flood Bund R.D. 9-10 after Floods 1976. Please note the excessive damage in the concave.bend.

.Chenab Flood Bund after floods 1976.
Please note extent of damage and length of beach slope.