STUDIES OF WAVE LOADS ON CONCRETE SLOPE PROTECTIONS OF EARTH DAMS

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SUMMARY

A description is presented of a wave flume with a pneumatic wave productor, which can generate waves up to 2 m high. The principal experimental results are outlined of dynamic wave action on the slope protection of an earth dam, with the slope protection formed of slabs with open joints placed on an artificial filter layer. Data are cited on wave pressure distribution over the upper surface of the slope protection, and on the uplift pressure occurring under the slabs.

The effect of scale modelling on the wave pressure values in the breaking zone is estimated based on a series of scaled experiments, with wave heights ranging between 5 and 125 cm.

The erection of large modern hydraulic river projects is closely connected with the construction of large reservoirs where wind waves of over 3 m high may occur. Under these conditions the problem of protection from wave action of upstream slopes of earth dams and dykes, as well as protection of the natural shoreline from erosion and scour assumes a special importance.

The following types of slope and bank protection for hydraulic structures are extensively used at present in the construction practice of the U.S.S.R.:
1. Concrete protections:
   a) monolithic slabs concreted in situ,
   b) protection constructed of prefabricated slabs.
2. Asphalt concrete impervious coatings.
3. Stone protections:
   a) rock facing,
   b) dumped rockfill.

The main problems in designing the strength and stability of slope protection can be reduced to establishing two values:
a) the maximum hydrodynamic wave pressure on the slope protection
b) the maximum uplift pressure under the slab protection.

In the Soviet Union three main trends can be distinguished in the research into wave loads on slope protections.
1. Theoretical studies for determining the mean value of the hydrodynamic pressure in the breaking zone and establishing regularities in the propaga ·

tion of the impact under the slab, both for open and closed joints between the slabs.

2. Models and special stands were used for investigations into the dynamic wave action on slabs, in particular, the establishment of relationships between wave parameters and wave loads under conditions of varying slope steepness, head, soil characteristics under the slabs, and the permeability factor of the protection.

3. Verifying of the experimental findings and the conclusions drawn at special experimental areas in the field.

It should be noted that the possibilities of theoretical solutions in designing slab strength and stability are essentially limited at present by the lack of data characterizing the complex mechanism of the interaction between the waves and the slope, in particular, such important characteristics as the duration of the wave impact, pressure pulsations in the impact zone, turbulence parameters in the run-up zone on the slope, the frequency of free oscillations of slabs, with virtual masses of water and soil taken into account, the decrement of damping of slabs oscillations et al. The above characteristics can be obtained only experimentally.

At the VNIIG when considering different procedures of experimental research into the dynamic action of waves on slope protections it was found expedient to use the scale series method with wave parameters closely approaching those in the prototype.

The necessity of conducting large-scale model experiments is mainly caused by the desire to get rid of the scale effect and establish conditions when the results of model investigations can be recalculated for the prototype according to the scale relations implied by the law of gravitational similitude, i.e. the Froude criterion.

At the Institute an experimental set-up was built consisting of a wave flume and a wave productor capable of generating waves up to 2 meters in height (Fig.1.).

The present paper is devoted to a short description of the set-up and some research findings on wave loads on slabs with open joints.

I. Description of the Experimental Set-Up.

The operation of the pneumatic wave productor is based on inducing oscillations in the liquid in an air tank with its open side submerged in

** water. The oscillations in the air tank are created by the varying air pressure in the part of the tank which is above the water level.

The air pressure parameters, the dimensions of the tank and the characteristics of the waves generated were established by utilizing the theoretical solution to the problem of wave generation caused by periodically varying pressures.

Using Lamb's solution for wave disturbances at an infinite depth E.F.Sakhno (R.1) received a more general solution of the problem for the case of waves at a limited depth under unit surface pressure equal to unity varying in accordance with the law:

$$P(x,t) = e^{i\sigma t} \cos k (x-a) \qquad / 1 /$$

where

e	-	the base of natural logarithms.
σ	-	circular frequency of pressure variation.
T	-	wave period.
t	-	time.
λ	-	wave length.
a	-	width of the air tank.
x	-	abscissa with the origin of coordinates at the still water level at the reservoir wall.

Preliminary experiments showed that air pressure in the air tank is
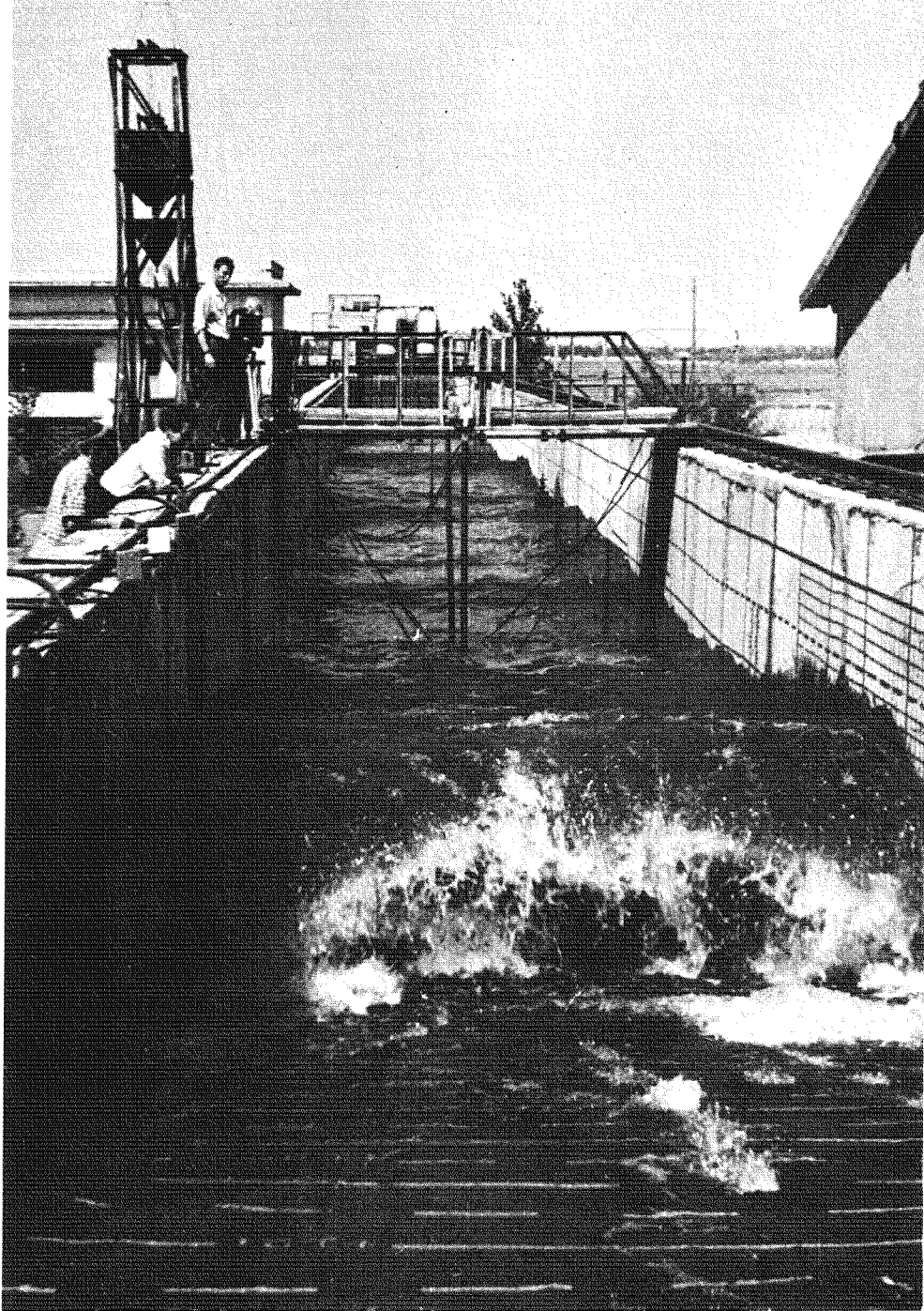
Fig.1. General view of the wave stand.

uniformly distributed across its width. In this case the equation for the waves generated by the wave producer can be written as :

$$y(x,t) = -\frac{2P_0}{\gamma} A e^{-2\pi b/\lambda} \sin 2\pi \frac{a}{\lambda} \sin\left(\sigma t - 2\pi \frac{x}{\lambda}\right) \qquad /2/$$

where
y – wave surface ordinate.
P_0 – amplitude of the air pressure in the air tank.
γ – volume weight of water.
A – factor taking account of the effect of shallows on the wave amplitude.

$$A = \frac{Sh\ 4\pi \frac{H}{\lambda}}{Sh\ 4\pi \frac{H}{\lambda} + 4\pi \frac{H}{\lambda}} \qquad /3/$$

– the depth of immersion of the tank in undisturbed water
– the depth of water in the flume.

The set-up is comprised of two principal parts: a wave flume and a pneumatic wave producer (Fig.2). The system of controlled air supply into the air tank /1/ incorporates two fans /2/, suction and delivery pipes /3,4/, an air distribution valve /5/ and butterfly throttles /6/.
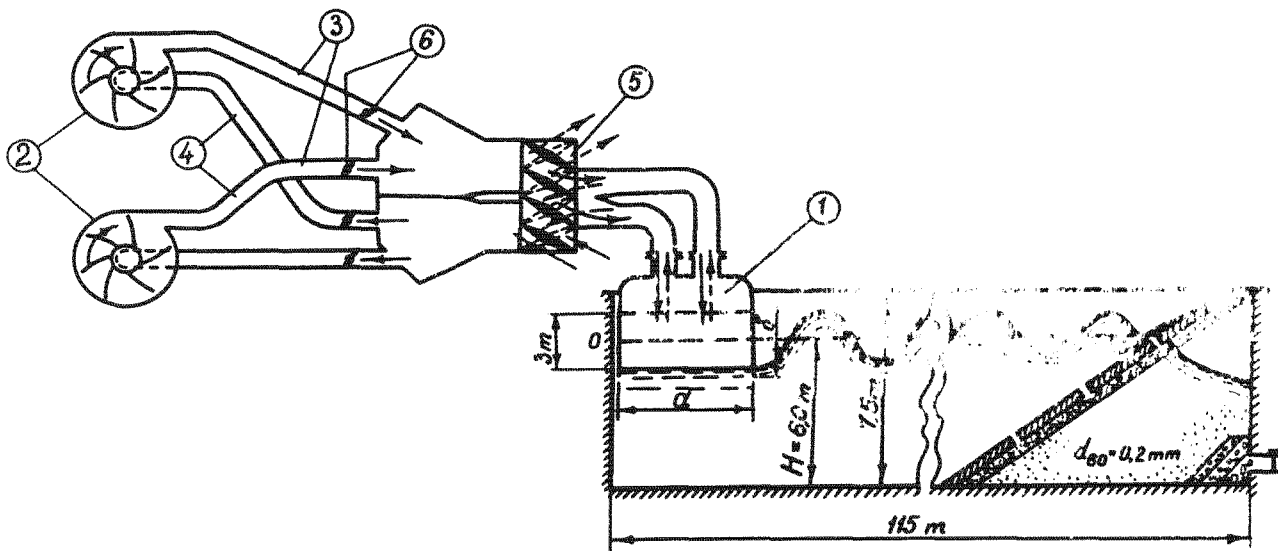

Fig.2. Layout of the wave stand with a pneumatic wave productor
1 – wave generating tank, 2 – fans, 3– and 4 – suction and delivery piping, 5 – air distributing valve, 6 – butterfly throttles.

The air distribution valve is designed to alternately connect the delivery and suction pipes to the atmosphere or to the part of the tank filled with air during the operation of the wave generator.
The wave height depends on the volume of air forced into and out of the tank. The air flow through the air pipes is regulated with butterfly throttles. The shutters of the air distribution mechanism are operated through a reducer and a crank drive by a direct-current motor, which allows to control the shut-off frequency of air pipes / wave frequency / within a wide range from 0.55 to 0.17 1/sec., which corresponds to the wave length of 5 to 40 m .
Characteristics of the experimental set-up:
1. Wave flume dimensions:
Length – 115 m, Width –– 4 m, Depth – 7.5 m.
2. Wave parameters:
Height, h = 0.5 – 2.0 m ; Length, λ = 5 – 40 m ; Period, T = 1.8–5.8 sec

3. Fan characteristics:

Overall capacity – 1400 m^3/min. Excess working pressure – 2900 mm of water column. Total output of electric motors for operation of fans – 800 kW. Air pipe diameter – 704 mm.

The wave productor tests have shown that:
- The profile of waves generated with a steepness $h/\lambda < 1/20$ is close to a sinusoid, and of those with a steepness $h/\lambda \geqslant 1/20$ is closer to a trochoid.
- The optimum depth of immersion of the air tank in undisturbed water, $b = (0.08 - 0.10)\lambda$.
- Wave generator efficiency factor is $\approx$ 40%.
- Steady -type waves are formed at a distance of 1.5λ from the wave generator.
- The maximum deviation of the height of the waves generated from the calculated value is $\pm$ 5%.

As compared to a mechanical wave productor a pneumatic one possesses a number of advantages: its higher efficiency results in reducing electric power consumption by the crank drive, the layout of the driving mechanism is more compact, wave generation takes place over a short section at the beginning of the flume and is performed with greater accuracy.

II. Distribution of Maximum and Minimum Wave Pressures over the Slope.

The term "wave pressure" is used here to denote a deviation of the pressure p at a given point on the slope protection from the hydrostatic pressure p_0 , i.e. $\Delta p = p - p_0$.

The investigation on the distribution of the extreme values of wave pressure over the slope was aimed at determination of the maximum pressure value in the breaking zone, and evaluation of the degree of wave pressure damping above and below the breaking zone.

At the first stage of the study attention was concentrated on waves of a steepness 1/8 – 1/10 as most representative for river reservoir conditions.

The results of pressure measurements on a slope 1 : 4 for a wave steepness h/λ = 1/10 are plotted in Fig .3.

In this diagram along the y -axis maximum wave pressure is plotted in terms of the water column height against the wave height ratio $\frac{\Delta p_{max}}{\gamma h}$ and $\frac{\Delta p_{min}}{\gamma h}$ while along the x -axis is plotted the relative distance from the water edge. It was established that x/λ

1. The maximum pressure at the wave impact point is $\Delta p_{max} = 1.45 \gamma h$
2. The maximum pressure point is at a distance of $x = 0.07\lambda$ from the water edge.
3. The minimum pressure is about $\Delta p_{min} = -0.25 \gamma h$ and is applied at a distance of $(0.2 - 0.4)\lambda$ from the water edge.
4. The experimental points obtained at different wave heights / h = 50, 75, 100 and 125 cm / are concentrated about the curves $\frac{\Delta p_{max}}{\gamma h}$ and $\frac{\Delta p_{min}}{\gamma h}$ which indicates the existence of a stable functional relationship.

The second stage of the studies is devoted to determining the influence of wave steepness on the magnitude of wave pressure. Experiments conducted for $\varepsilon = h/\lambda$ = 1/10, 1/15, 1/20, 1/25, 1/30 showed that with a reduction in wave steepness, i.e. when wave length increases and wave height remains constant the relative pressure in the impact point increases linearly.

The relationship $\Delta p_{max}/\gamma h = f(\varepsilon)$ for the wave impact point is illustrated in Fig.4. The linear relationship between pressure and wave steepness seems to be caused by the linear dependence between wave energy and wave length when wave height remains constant.
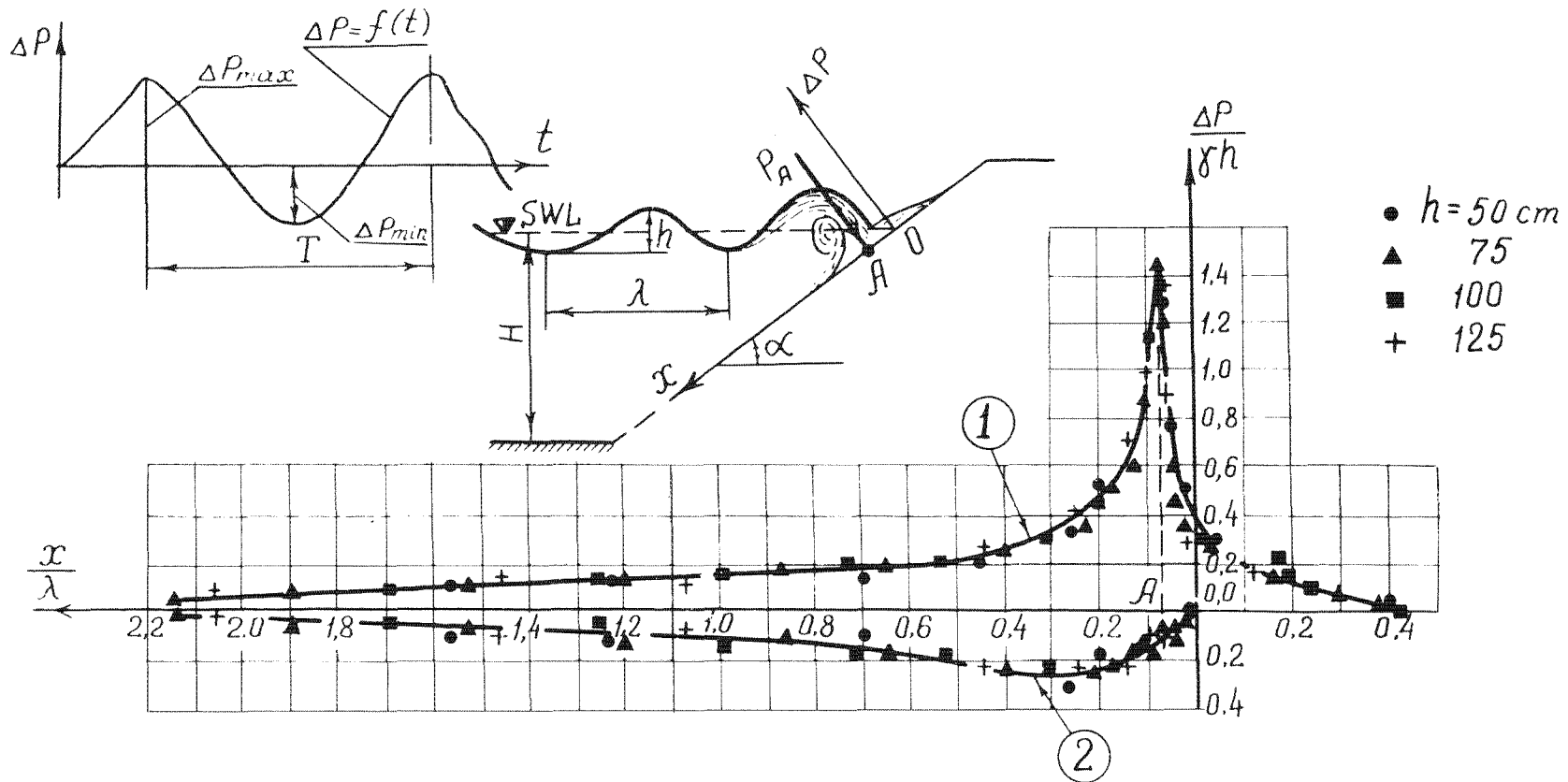
Fig.3. Distribution of extreme values of wave pressure on the upper surface
of the slab protection (data based on experimental studies on the wave stand).

① $\dfrac{\Delta P_{max}}{\gamma h}$;   ② $\dfrac{\Delta P_{min}}{\gamma h}$; $\varepsilon = \dfrac{h}{\lambda} = \dfrac{1}{10}$, $\dfrac{H}{\lambda} = 0{,}48 - 1{,}1$; $ctg\,\alpha = 4$.
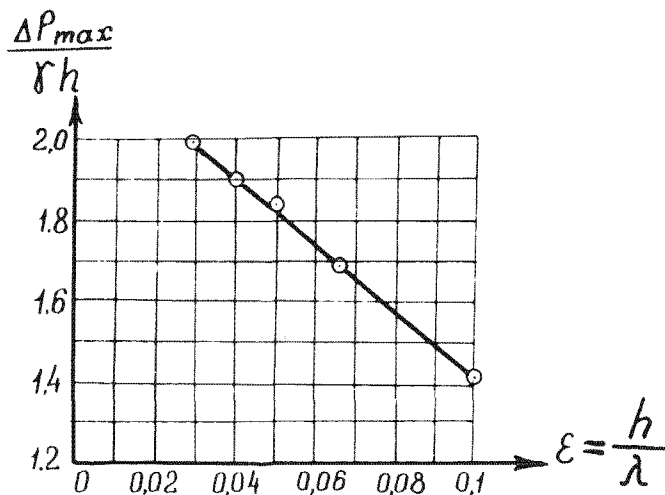
$$\frac{\Delta P_{max}}{\gamma h}$$

Fig.4. Effect of wave steepness on the
wave pressure valve at the zone.

III. Resultant Wave Pressure on Slabs.

The strength of slabs is designed according to maximum wave pressures acting on their upper surfaces. The thickness of the slab ensuring its stability on the slope is determined by the maximum uplift value. Wave pressure on the upper surface of the slab depends on wave parameters, slope steepness, water depth above the structure. Besides, wave pressure is also affected by the perviousness factor of the protection (in the case under investigation by the number and dimensions of joints between slabs), the location of the wave impact point relative to the joint, and the seepage characteristics of the foundation. The modelling of hydrodynamic processes under the slab involves great difficulties. Therefore the establishment of regularities in the distribution of wave pressures under the slab required a series of scaled experiments, with wave heights approaching those in the prototype. Wave pressure and uplift studies were conducted on an earth dam model 7.5 m high, with a crest width of 5 m and the upstream batter 1 : 4. The dam was constructed of fine sand. The upstream slope of the dam was protected by concrete slabs 4 x 1.98 m, 15 cm thick, with the joints between the slabs 2 cm wide. The slabs were placed on a continuous inverted filter 25 cm thick. The ✱✱largest particle diameter in the filler material was 70 mm.

Wave pressure and uplift were recorded by inductance meters with a resolution of up to 500 cps.

In the breaking zone the meters were spaced at 10 cm intervals; above and below the breaking zone the spacing varied between 50 and 100 cm. Two pickups were placed at every metering station, permitting to record wave pressure and uplift simultaneously. In the experiments wave heights varied between 50 and 1.25 cm, and wave steepness between 1/10 and 1/35. Synchronous recording of dynamic wave action on the slope for different wave phases with an interval 0.1 T led to obtaining curves of resultant pressure on slabs in the form of

$$\frac{\Delta P}{\gamma h} = \frac{\Delta P_1 + \Delta P_2}{\gamma h} = f\left(\frac{y}{\lambda}\right)$$

- 7 -

where ΔP_1 and ΔP_2 - wave pressure and uplift, respectively.
h and λ - height and length of wave.

$\quad\quad\quad\quad x$ - distance of the point on the slope from the water edge.

$\quad\quad\quad\quad T$ - wave period.

The most unfavourable combinations of loads on the slab were found to occur during two wave phases:

1. At the moment of breaking on the slope, which corresponds to maximum pressure on the upper surface of the slab.

2. At the moment preceding the breaking of the wave ($0.20 - 0.25$)T before the impact, when maximum uplift pressure occurs under the slab. The indicated moments and corresponding curves are accepted as design ones in calculating strength and stability of the slab protection under study.

Curves of resultant wave pressure for the indicated wave phases are presented in Figs 5-I and 5-II for the case when the wave impact zone lies in the central part of the slab. The location of the joint between slabs relative to the wave impact zone materially affects the magnitude and the distribution of the resultant wave pressure. In Fig.5-III the pressure curve is given for the case when the wave impact falls on the joint between the slabs. In this case due to instantaneous transfer of external pressure to the zone under the slab, the uplift pressure under the slabs above and below the impact zone increases and the resultant downward pressure value in the impact zone decreases (up to 40%).

The change in maximum ordinates of pressure and uplift depending on the location of the wave impact zone in relation to the joint is shown in

Fig. 5-IV.

Experiments conducted led to the following conclusions:

1. Coincidence of pressure variation phases both for the upper and lower surfaces of the slab protection is achieved by making pervious joints between slabs placed on a continuous inverted filter.

2. Coincidence of pressure phases leads to a reduction (up to 40%) in the total wave load on the upper surface of the slabs and to a material increase in the uplift pressure under the slab protection with the joints open as compared to a continuous protection.

3. Periodical pressure variations under the slabs exert an adverse influence on the performance of the inverted filter, therefore its stability against piping and its percolation characteristics must be higher than those for a filter underlying slab protections with closed joints.

4. The problem of the permeability factor of the protection should be solved on the basis of an economic comparison between different design versions for the protection.

IV. Effect of Wave Dimensions on the Relative Pressure Value in the Wave Impact Zone.

In the course of experimental studies the researcher is often confronted with the problem of establishing the minimum model wave dimensions which allow to scale up experimental findings to the prototype on the basis of the Froude criterion. For dynamic wave action on a slope the automodelling region is not yet strictly defined. A series of experiments was conducted with wave heights ranging between 3 and 120 cm, with a constant steepness of 1/10 aimed at defining the scale effect of the absolute wave dimensions on the value of relative wave pressures in the impact zone, the run with 120 cm wave height being provisionally accepted as the prototype.

The other runs with wave heights $h_M \leqslant$ 120 cm were considered as

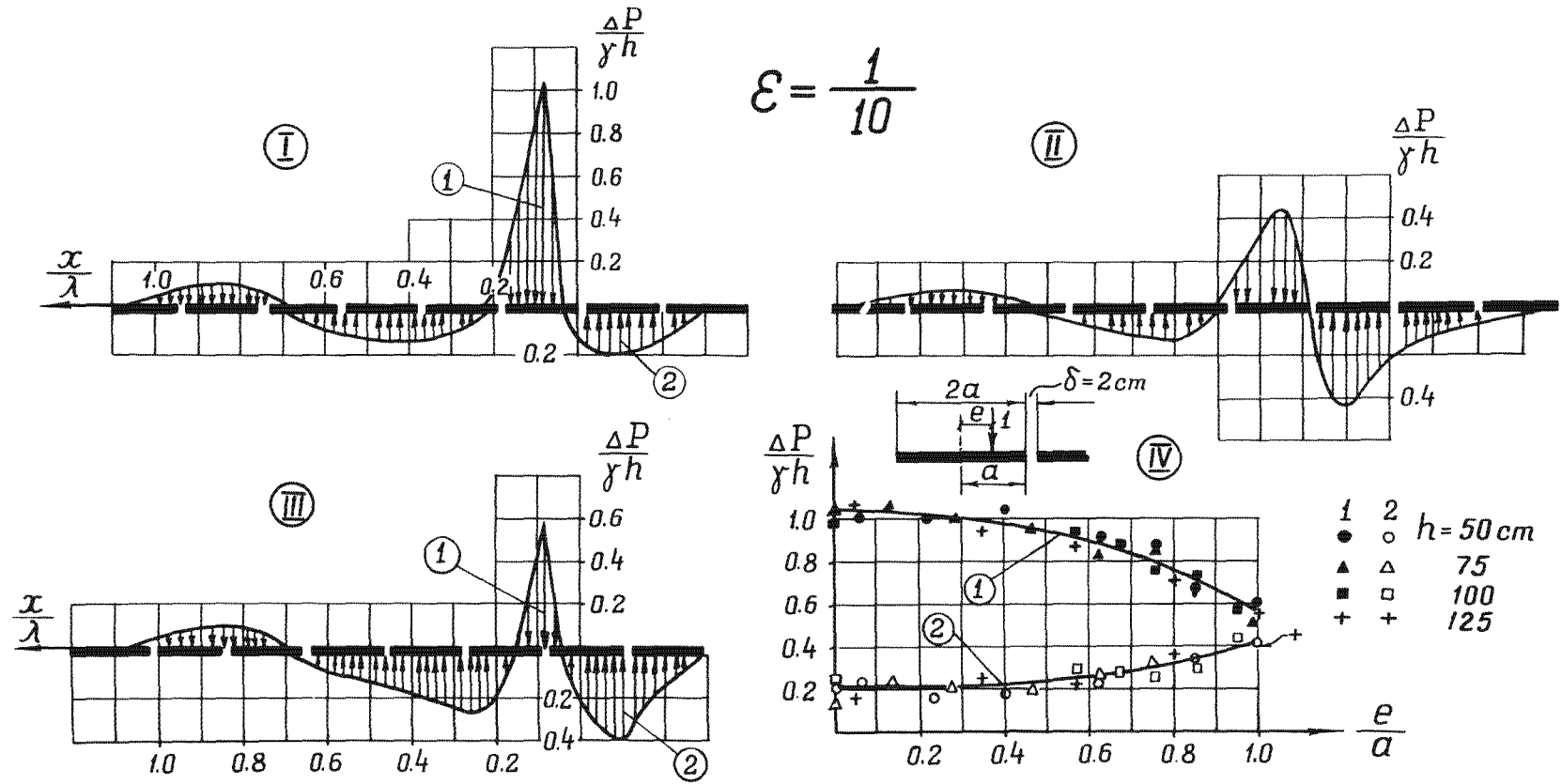** models of the prototype. The results of the scaled series of experiments are

Fig.5. The resultant of the wave pressure on the slabs.
I – with t = 0.0 (moment of impact) } wave impact on the
II – with t = 0.2 T (before the impact) } central zone of the slab.
III – with t = 0.0 wave impact at the joint between the slabs.
IV – Variation of the resultant wave pressure depending on the location
    of the wave impact relative to the joint between the slabs.

presented in Fig.6 , where $\quad K_p = \dfrac{\overline{\Delta P_M}}{\overline{\Delta P_N}} \; ; \quad \alpha = \dfrac{h_M}{h_N} \; ;$

$$\overline{\Delta P_M} = \dfrac{\Delta P_M}{\gamma h_M} \; ; \quad \overline{\Delta P_N} = \dfrac{\Delta P_N}{\gamma h}$$
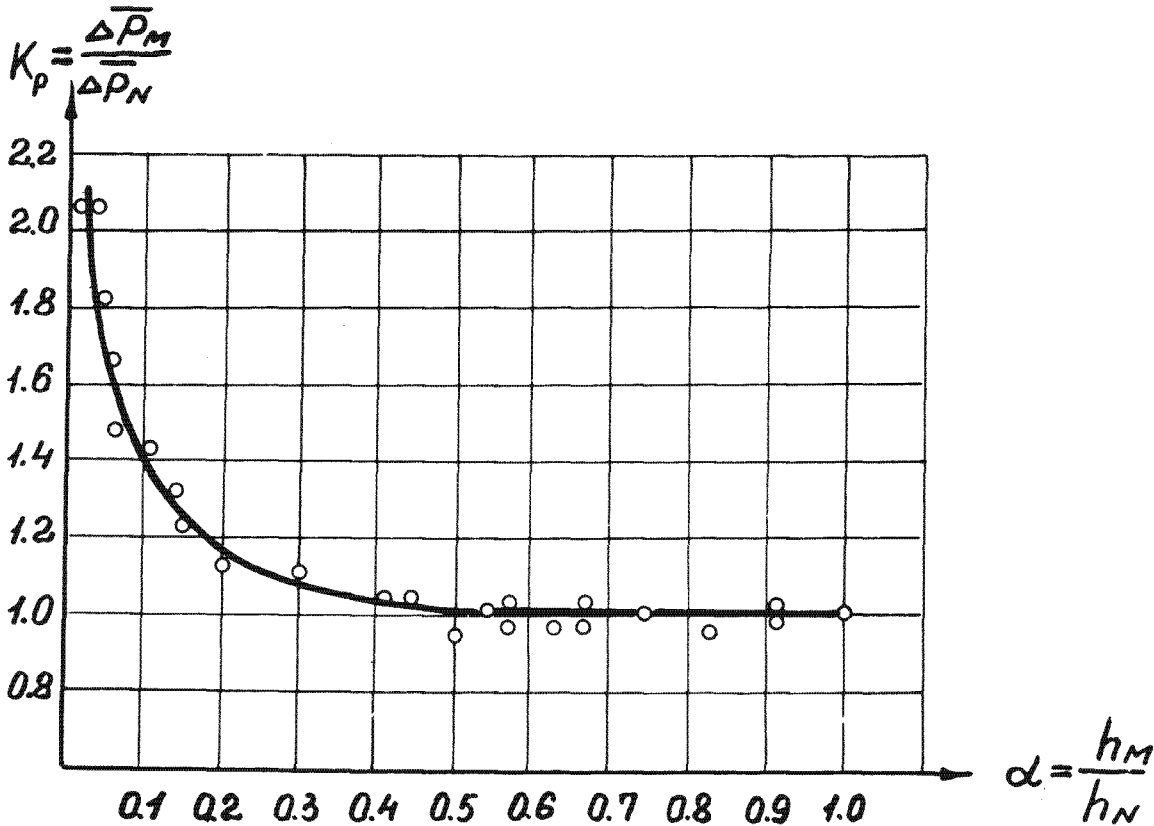
Notation for the subscripts : M - model, N - prototype.

Fig.6. Effect of wave height on the relative wave pressure value in the wave impact zone.

As can be seen from the graph the value of relative wave pressure is practically independent from the absolute wave height only when $\alpha \geqslant 0{,}4$, i.e. at wave heights $h \geqslant 50$ cm.

The data obtained from the scaled experimental series indicate, that on the basis of the Froude criterion it is not permissible to directly scale up to the prototype values of wave pressures measured on small scale models. One of the principal reasons of the great difference in pressures recorded at the impact zone seems to lie in the fact that with greater wave heights (h 50 cm) the jet falling from the wave crest is highly aerated and sprayed, the water cushion which transmits the impact pressure on the slab is also air saturated and the interaction between the wave and the slope materially differs from that of small .height waves when aeration is negligible or does not occur at all.

Hence it follows that if the maximum wave height in reservoirs usually varies between 3.0 and 4.0 m, the minimum model scale, which enables to obtain reliable data on wave loads on protection, should not be less than 1/7 - 1/8 of the prototype dimensions.

List of Notations.

h - wave height.
λ - wave length.
T - wave period.
ε - wave steepness.
t - time.
e - base of natural logarithms.
y - wave surface ordinate.
p_o - amplitude of pressure variation in air tank.
p - pressure on slabs with waves generated in the flume.
p_o - hydrostatic pressure on slabs (pressure with still water level in flume).
Δp - wave pressure.
γ - volume weight of water.
P - resultant wave pressure.

Reference

1. Sakhno, V.F., "Theory and Design of Pneumatic Wave-Productors", Trudy TsNII im. Acad. A.N.Krylov, 1961, vyp.12, Sudpromgiz.

PAPER 7

Studies of Wave Loads on Concrete Slope
Protections of Earth Dams,

by SKLADNEV and POPOV

Due to illness of the authors, there has been no discussion on Paper 7.
One of the participants to the Symposium, Mr. Bakker, was so kind to make
available a copy of his post symposium correspondence with the authors.
The Organizing Committee considers this correspondence of such importance
that it is reproduced here.

BAKKER: With much interest I read the paper of Mr. Popov and you.

In order to compute the needed thickness of asphalt revetments
Rijkswaterstaat intends to use a computer program, developed by the Royal
Dutch Shell Company, which enables to find the tension in the asphalt
revetments, if the load, elasticity coefficients of asphalt and basement
and other material constants are known.

Your paper can give a contribution about the knowledge of the loads
by wave impact. However, not only the maximum wave pressure is of
importance in this case, but also the distance along the slope over which
the impact takes place.

According to your Figure 3, a reasonable, unfavourable approximation
seems to be a uniform load of $1.4 \gamma h$ over a distance from $\frac{x}{\lambda} = .05$ till $.10$,
if the slope is 1:4 and the wave steepness ε is $\frac{1}{10}$ (Fig.1).

However, although Fig. 4 gives the influence of smaller wave
steepness on the wave pressures, I did not find the influence of the wave
steepness on the distance along the slope, over which the impact takes
place.

Although the horizontal axis in your Fig. 3 gives $\frac{x}{\lambda}$, it seems
dangerous to extrapolate this result for other wave steepness. For in the
case of Fig. 3, the impact takes place roughly over a distance $0.05\lambda =$
$= 0.05 . 10 h = 0.5 h$, which sounds reasonable. However, for a wave
steepness 0.02 this would be $0.05 . 50 h = 2.5 h$, which is more difficult
to imagine.

Summarizing, I would ask the following questions:
1. Do you think the scheme of Fig. 1, below, is reasonable in order to
   compute the maximum bending moments of an asphalt slab, in the case of

the conditions mentioned in your Fig. 3 ?

2. Can you send me similar Figures, as Fig. 3, for other wave steepness ?

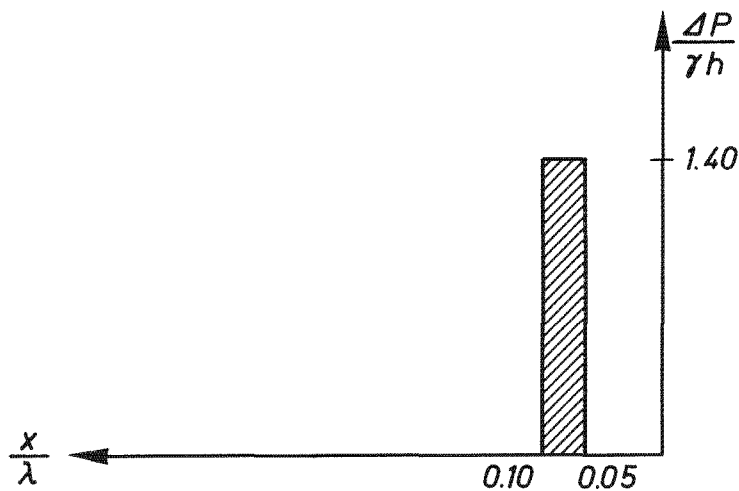3. Do you think the permeability of the slab you used effected the maximum wave pressure very much ?

$$\frac{\Delta P}{\gamma h}$$

1.40

$$\frac{x}{\lambda}$$

0.10   0.05

FIG. 1 SCHEMATISED WAVE PRESSURE

SKLADNEV: Unfortunately a prolonged illness prevented me from answering at the proper time your letter of March 28, 1969. Having resumed my duties at the Institute only at the end of May, I was unable to prepare earlier the materials for our joint reply with I.Ya. Popov to the questions you are interested in.

1. The diagram in Fig. 1 at the end of your question may be used as an approximate representation of wave impact pressure in the breaking zone with wave steepness 1/10. It must be borne in mind that the diagram $\frac{\Delta p}{\gamma h} = f \left(\frac{x}{\lambda} \right)$ shown in Fig. 3 of our report illustrates the distribution of extreme values of wave pressures over the surface of the slope. It goes without saying that extreme pressures do not occur simultaneously, but with a phase shift in time. However, taking into account the comparatively short length on which the impact pressure is applied, in our opinion, the phase shift can be neglected and, therefore, the scheme adopted by you may be considered acceptable.

2. According to our findings the length of the impact zone $\frac{x}{\lambda}$ changes only slightly with decreasing wave steepness. The change in the length of the impact zone with a varying $\frac{h}{\lambda}$ can be observed in the diagrams $\frac{\Delta p}{\gamma h} = f \left(\frac{x}{\lambda} \right)$ which correspond to $\frac{h}{\lambda} = \frac{1}{15}$, $\frac{1}{20}$, $\frac{1}{25}$ and $\frac{1}{35}$ (Figures B, C, D, and E). The above diagrams as well as the diagram in Fig 3 of our report are plotted without taking into account uplift pressure and are

valid only for _impermeable continuous slightly_ deformable slab protections.

3. In the case you refer to for more accurate calculations can be applied the diagram of instantaneous wave pressure at wave impact with $\frac{h}{\lambda} = \frac{1}{35}$ (Fig. A). The "peak" value of $\frac{\Delta P_{max}}{\gamma h}$ for wave steepness $\frac{1}{50}$ can be obtained by extrapolation from Fig. 4 of our report.
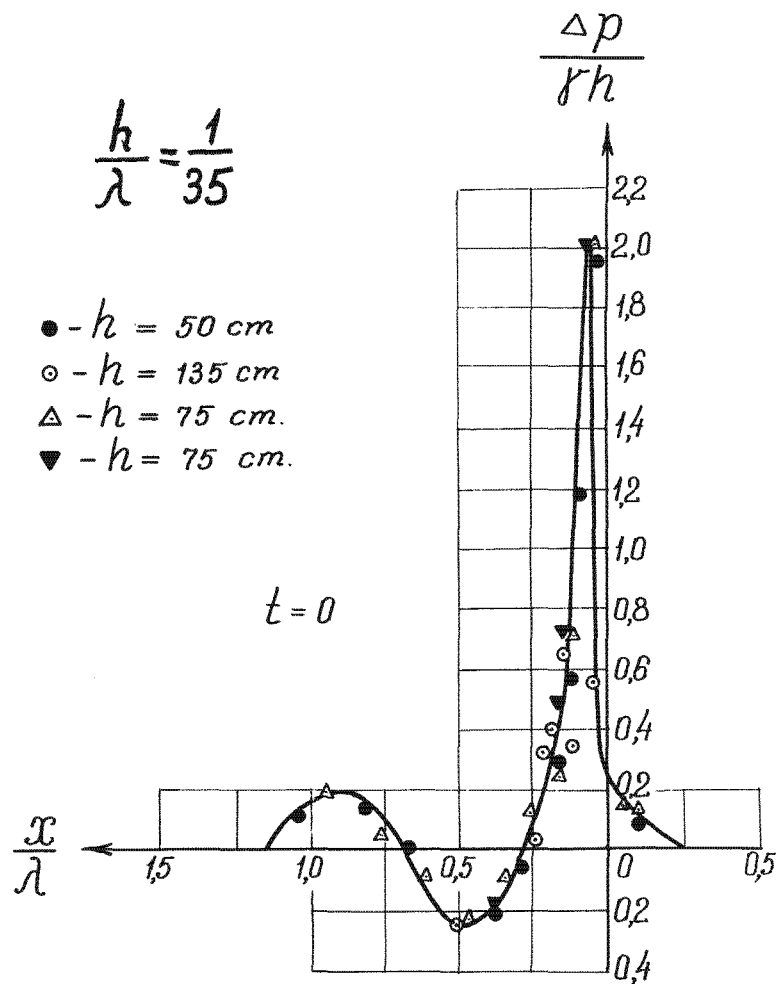
$$\frac{h}{\lambda} = \frac{1}{35}$$

$$\frac{\triangle p}{\gamma h}$$

● $- h = 50\ cm$
⊙ $- h = 135\ cm$
△ $- h = 75\ cm.$
▼ $- h = 75\ cm.$

$t = 0$

$\frac{x}{\lambda}$

2,2
2,0
1,8
1,6
1,4
1,2
1,0
0,8
0,6
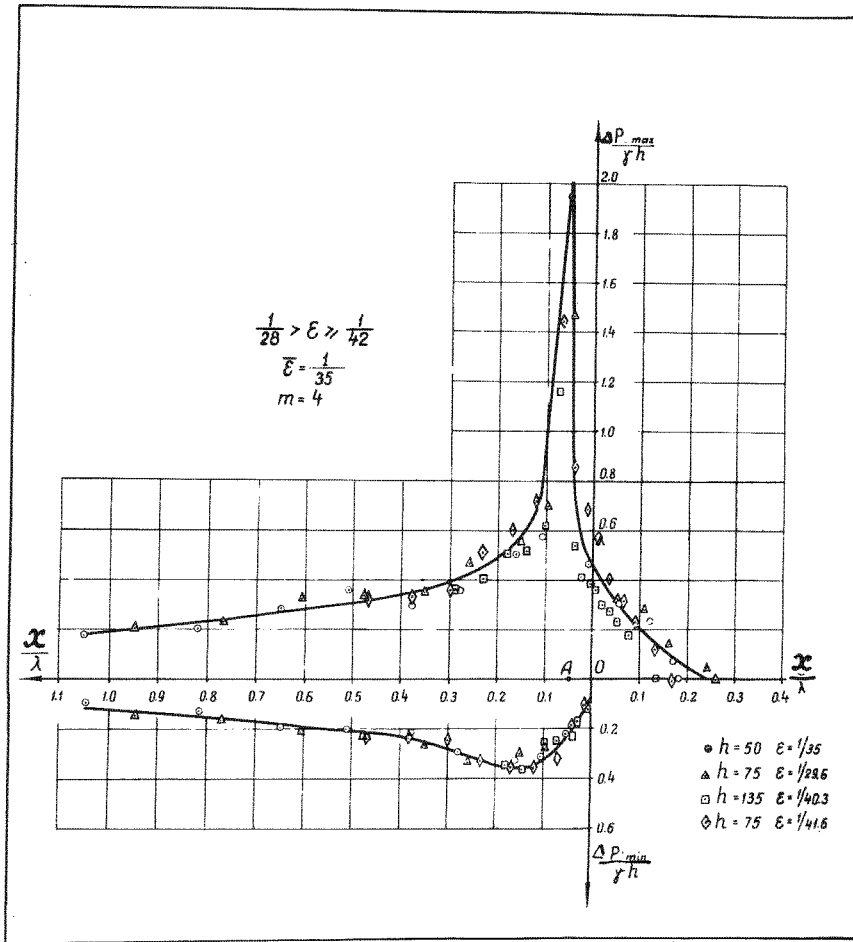0,4
0,2

1,5   1,0   0,5   0   0,5

0,2
0,4

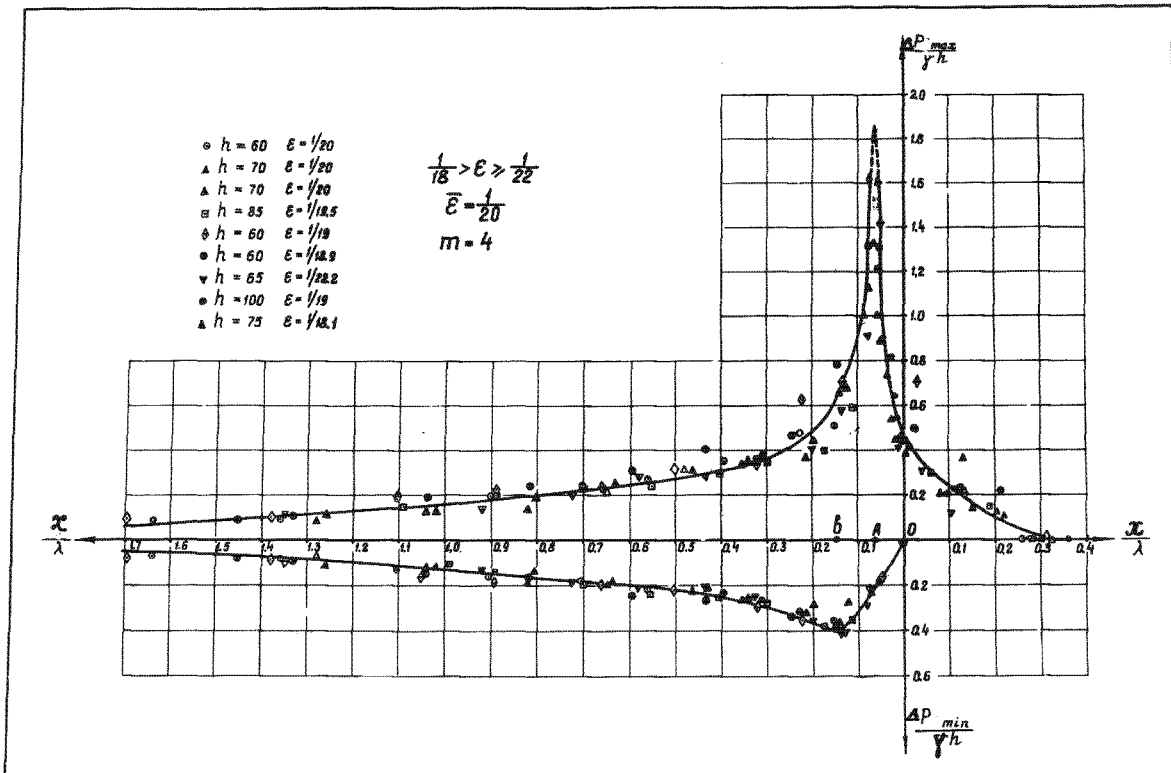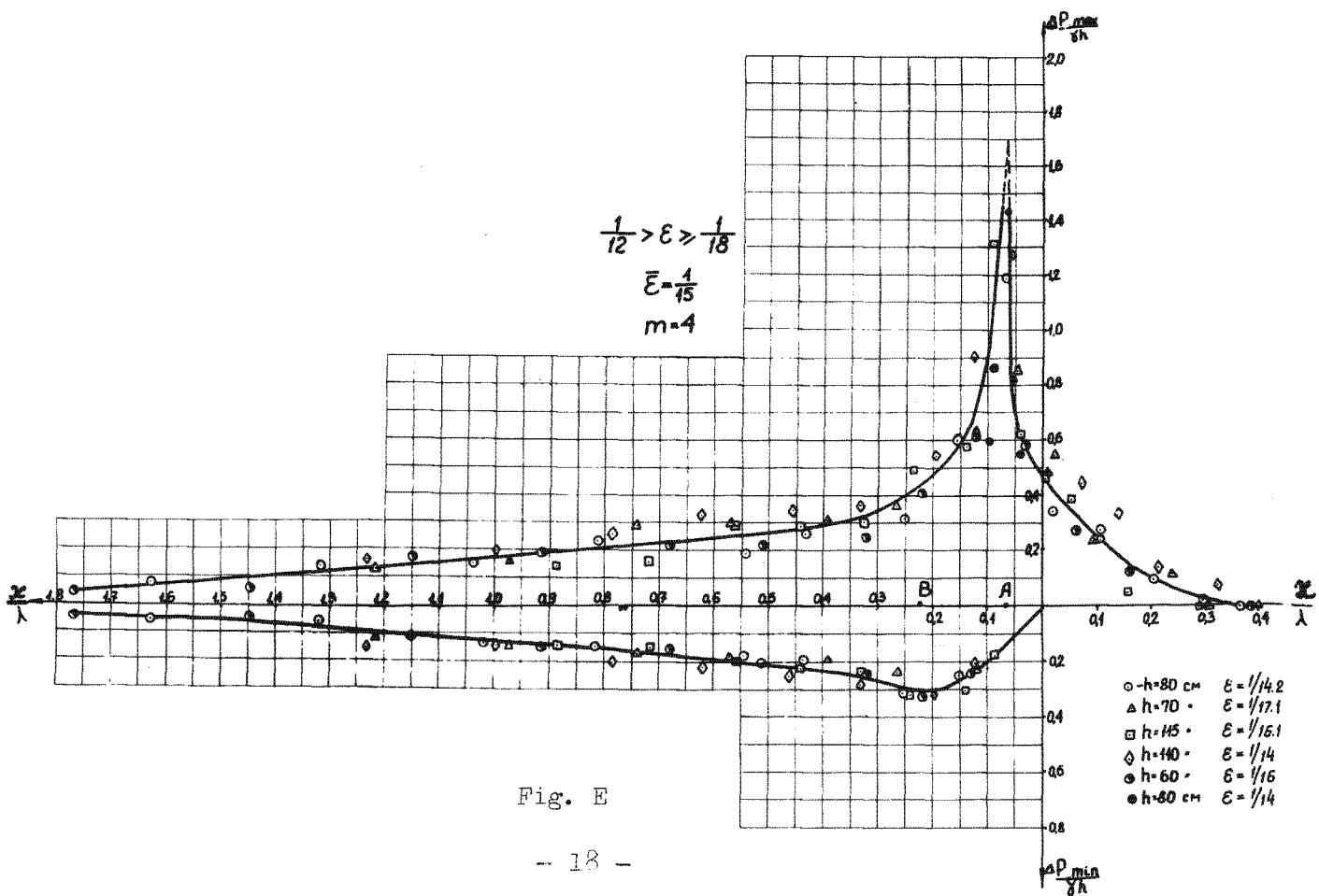Fig.A   Diagram of wave pressure at the wave impact.

Fig. B

Fig. C

Fig. D

Fig. E