

SUPPLEMENT NUMBER 1

TO

PRELIMINARY SAFETY ANALYSIS REPORT

FOR

OYSTER CREEK UNIT #2

JERSEY CENTRAL POWER & LIGHT COMPANY

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SEPTEMBER 3, 1968

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SUPPLEMENTAL FLOOD STUDIES
FOR
OYSTER CREEK NUCLEAR POWER PLANT
UNITS NO. 1 and NO. 2

Report to:
Jersey Central Power & Light Company
Morristown, New Jersey

BASIS FOR REPORT

1. Under date of December 28, 1967, the undersigned submitted a report on the subject of tidal flooding to be expected under conditions of the probable maximum hurricane which could occur in this area. By letter June 25, 1968, the AEC requested the following additional information from Jersey Central Power & Light Company:

- a. All surge height elevations associated with the Probable Maximum Hurricane (PMH) should be evaluated based on the PMH parameters derived by the Hydrometeorological Branch of the U. S. Weather Bureau. These parameters, appropriate for the Oyster Creek site, are defined in "Meteorological Characteristics of the Probable Maximum Hurricane Atlantic and Gulf Coasts of the United States" Interior Report, U. S. Department of Commerce, ESSA, Weather Bureau Hydrometeorological Branch, dated May 1968.
- b. Computations involving variations of the PMH parameters, namely radius to maximum winds (R) and forward speed of translation (T) should be carried out in order to establish that the hypothetical storm selected produces the maximum surge height at this site. The results of this investigation should be fully discussed in your response.
- c. The maximum open coast surge height should be evaluated and described using methods given in either "Shore Protection, Planning and Design" Technical Report No. 4, U. S. Army Coastal Engineering Center, Third Edition, 1966, or "Estimation of Hurricane Surge Hydrographs," Journal of the Waterways and Harbors Division, ASCE, Vol. 94 No. W.W. 2, Proc. Paper 5945, May 1968.

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- d. It is not apparent from the text of the Unit 2 PSAR that an additional pile up of water within the bay was considered at the mainland shore in the vicinity of the site due to wind stress acting on the free water surface of the bay. This water pile up would give an additional setup of the surge. This effect should be considered in your evaluation using the method of computing the bay surge coupled to the open coast surge presented in "Numerical Model for Storm Surges in Galveston Bay," by Reid, R. O. and Bodine, B. R., Journal of the Waterways and Harbors Division, ASCE, Vol. 94 No. W.W. 1, Proc. Paper 3805, February, 1968.
- e. What is the additional water surface rise in the bay system due to direct rainfall and upland rainfall runoff? The quantity of rainfall and duration would be that associated with the Probable Maximum Hurricane.
- f. Describe the results of your analysis and methods used to support the extreme low tide elevation of -3.4 feet MSL as stated in the Unit 2 PSAR.
- g. Information is necessary to support the 1 foot wave runup at the site since it is not apparent that wind waves would decay to the magnitude indicated in Section 2.4.3.1e of the Unit 2 PSAR.

PRELIMINARY CONSIDERATIONS

2. The ESSA Memorandum of May 1968 in its present form is regarded as an excellent document providing guidelines but not necessarily providing exact values for specific sites. In transmitting the report to the Corps of Engineers it was stated that it "presents preliminary estimates of generalized indices for the Probable Maximum Hurricane (PMH) along the Atlantic and Gulf Coasts of the United States". "Preliminary" means that the values arrived at are not certain. All reports of that type will probably be preliminary for many years in the future for meteorology is still much more of an art than a science. "Generalized" means that the values are approximations with respect to any specific site and must be adjusted by interpolation and evaluation of local environmental anomalies. It is our mission to interpret the ESSA Memorandum in terms of its application to Oyster Creek and to determine what adjustments, if any, should be made in our prior report.

3. It is obvious that assigning the highest possible value to each of the parameters that govern storm intensity and its effect at a specific shore location would result in the Maximum Possible Hurricane rather than the Probable Maximum Hurricane. Definition of the latter becomes a matter of reasoning and judgment.

SUPPLEMENTAL STUDY

4. The attached report by my Associate, Mr. T. E. Haeussner, presents a thorough analysis of the ESSO Memorandum in application to Oyster Creek. He is not in agreement with the approach used in determining the Asymptotic Pressure and presents his reasoning in this regard. He also presents an adjustment of the critical forward speed of the storm which is based upon the effects of local topography. In my opinion his reasoning is quite sound and results do not violate the basic concept of the PMH.

5. He has presented an analysis applicable to each of the questions raised by AEC. He has evaluated the effect of wind set-up in Barnegat Bay which could raise the storm tide level at the plant site by as much as 1 foot and proposes raising the previously recommended design high water stage (17.06 ft. MSL) to 18.06 ft. MSL. He finds no other adjustments to be justified.

CONCLUSIONS

6. I concur in Mr. Haeussner's findings and recommend that the design storm tide for the MPH be 18.1 ft. MSL with 1 foot added for wave runup resulting in extreme flood stage of 19.1 ft. MSL.

July 23, 1967

Richard O. Eaton
Richard O. Eaton, P.E.
Consulting Engineer

Encl. Report by T.E.Haeussner,
dated July 16, 1968

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ADDITIONAL INFORMATION ON P.M.H.
FOR OYSTER CREEK, UNITS 1 AND 2

The following information is submitted in response to a request from the Atomic Energy Commission to Jersey Central Power & Light Company, dated June 25, 1968. Information requested was contained in Attachment A, as enclosure to that letter, consisting of 7 questions on Hydrology. Replies to those questions are enumerated 1 through 7 in the following paragraphs.

1. Evaluation of P.M.H. parameters. A comparison was made of the P.M.H. parameters contained in Unit 2 PSAR and those that would be derived from Memorandum HUR 7-97, "Interim Report - Meteorological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coasts of the United States". The latter report, dated May 7, 1968, contains preliminary estimates of the generalized indices for the P.M.H. and, as such, is an interim report with the final report to be published at some later date. For ease in reference, each parameter will be compared in a separate sub-paragraph below.

a. C.F.I. (p₀). Table 1 of ref. Memo. HUR 7-97 presents a listing of CPI values considered reasonably applicable at various latitudes along the Atlantic and Gulf coasts. The latitude of the plant site is approximately 39° 49' N.; that at the point of entry (landfall) selected for the PMH in Unit 2 PSAR is 39° 10' N. CPI values for latitudes 39° and 40° N. from Table 1 of HUR 7-97 are listed below together

with interpolated values for the above latitudes.

Table 1 HUR 7-97

Unit 2 PSAR

Lat. °N. CPI

CPI

39	27.09	inches	
40	27.17	"	
39 10'	27.103	"	*
39 49'	27.157	"	*

27.10 inches

*Interpolated, $\Delta p = .0013$ inch/min.

It can be seen from the above comparison that the Unit 2 PSAR CPI value used is within 0.003 inch of that specified in HUR 7-97.

- b. R - radius of maximum winds. Table 1 of ref. Memo. HUR 7-97 lists three possible radii for each C.I., RS, RM, AND RL. For latitude 39° N. the values given range from 7 to 39 nautical miles. The value of R used in Unit 2 PSAR is 30 nautical miles. The moderately large value of R, adopted in T.M. No. 120 for a PMH in the New York area (Ref. 7 to Unit 2 PSAR) is therefore considered reasonable for use in the Oyster Creek PMH tide analysis.
- c. Asymptotic pressure (p_n). In Figure 6 of ref. Memo. HUR 7-97 a relationship is given purportedly relating the highest possible peripheral pressure in a hurricane to latitude. It is based on an extreme envelope of p_n vs. latitude values observed in record hurricanes. A "curve of best fit" is also given representing average intensity hurricanes, which curve is about 1.3 inches less than the envelope curve. The p_n value used in Unit 2 PSAR (ref. 7 to that report) is 30.08 inches; the envelope value from

Figure 6 would indicate that a p_n value of 31.24 inches (1057.6 millibars) would apply. Using Figure 7 of Memo. HUR 7-97 a pressure value of 28.401 inches at R would be computed; the pressure value at R used in Unit 2 PSAR to compute pressure effect is 28.00 inches. however, the use of the envelope curve shown on Figure 6 is not considered reasonable from a meteorological standpoint. The most severe hurricane of record observed thus far in the south Atlantic area was that of September 1935 which had a CPI of 26.35 inches and an asymptotic pressure of 29.92 inches. The envelope curve in Figure 6 envelopes only 2 or 3 p_n values which, in themselves, are not associated with severe hurricanes of low CPI. In view of the fact that no meteorological evidence has been presented to prove conclusively that a severe hurricane of extremely low CPI must occur with an extremely high p_n value, the p_n value used in Unit 2 PSAR is considered reasonably probable of occurrence and applicable to the analysis made and results obtained in that report.

d. V_x - maximum wind speed. A maximum 10-minute average 30 ft.-overwater wind speed of 133 mph is given in Table 1 of ref. Memo. HUR 7-97 corresponding to a CPI of 27.09 inches at latitude 39° N., a p_n value of 31.24 inches (noted in sub-paragraph c above), and a forward speed of 30 knots. If a forward speed of 20 knots had been used the value of V_x would have been 6 mph less, or 127 mph. The value of V_x given in Unit 2 PSAR is 120 mph and was based on a p_n value of 30.08 inches and a forward speed of 20 knots. In view of the arguments given above, disagreeing with the use of an

envelope value for p_n , the value of $V_x = 120$ mph is considered reasonable for the PMH at Oyster Creek. The maximum difference in wind speed between the two values is about 6% (127 mph vs. 120 mph).

e. U - forward speed of the storm. Table 1 of ref. Memo. HUR 7-97 lists 3 forward speeds possible of use, ST, MT, and HT, in conjunction with PMH analysis. Those values are 11, 27, and 49 knots, respectively. In Unit 2 PSAR a forward speed of 20 knots was selected based on consideration of offshore shelf slope, and water depths along the Continental Shelf. As will be discussed in more detail below, the forward speed of the storm affects the shape and duration of the resulting surge hydrograph at the coast. For faster moving storms a slightly higher peak surge will result at the coast, but for a much briefer duration of time at lesser-than-peak height values.

In summary, the various parameters defining the PMH for the Oyster Creek tide analysis, as given in Unit 2 PSAR, are considered as having a reasonable probability of occurrence. The use of an envelope p_n value, having a high improbability of occurrence, in combination with an extreme CPI for the PMH...is not considered reasonable.

2. Variation of PMH parameters. Attachment A requests computations be made involving variations in the PMH parameters to establish that the hypothetical storm parameters selected produce the maximum surge height at the site. Excluding the value of p_n (discussed above) the only PMH parameter possible of variation in the study which might affect

the maximum surge height at the site is forward speed. The radius of maximum winds controls only the value of horizontal extent of the surge along the coast, not the value of the surge height itself. This fact is pointed out quite emphatically on page 743 of Reference 1 below. However, there does exist a critical forward speed which, with all other parameters held fixed, will give the highest surge at the coast, but not necessarily the highest surge, or tide elevation, at the plant site. In the Unit 2 PSAR a peak surge height of 11.25 ft. (exclusive of pressure and wave effects and the astronomical tide component) was computed. Based on a storm speed of 20 knots the surge hydrograph shown on Figure 2 of that report was determined. The effect of storm speed on peak coastal surge height can be related to the response factor, S, in the surge prediction equation, which depends on the ratio of forward speed of the storm to the speed of the free wave, \bar{C} . In Unit 2 PSAR the speed of the free wave was determined to be 62 ft/sec., or 42 mph, based on offshore topography. For a storm speed of 20 knots (23 mph) the ratio of $V/\bar{C} = 0.55$; for a storm of 30 knots (34.5 mph) that ratio = 0.82. The value of the response factor, S, for a speed of 20 knots is 1.03; for 30 knots it is 1.08. Using the latter value in the equation

$$N_m = \left\{ K \left(\frac{T}{C_1} \right) \left(\frac{H_m}{H_0} \right)^{\frac{1}{2}} \right\} W_m^2 S \quad (\text{from Ref. 8 of Unit 2 PSAR})$$

$$\text{gives } N_m = \left\{ 3.0 \times 10^{-6} \left(\frac{7,000}{79} \right) \left(\frac{19.5}{50} \right)^{\frac{1}{2}} \right\} 175^2 \cdot 1.08 = \underline{11.3 \text{ ft.}}$$

As shown above, the faster moving storm would result in a peak coastal surge 0.55 ft. higher. However, its resulting surge hydrograph must necessarily be of a shorter duration. This, in turn, must logically affect the peak surge height at the plant site by reducing the duration of overflow of the coastal beach strip and consequently the volume of overflow and inflow to the bay. The net result would be a lower water level in Barnegat Bay. In Unit 2 PSAR tidal overflow of the beach dune was determined to begin at $T_{.3\frac{1}{2}}$ hours and to extend to $T_{.2\frac{1}{2}}$ hours, at about which time the ocean surge begins to recede below the peak elevation reached in the bay (16.2 ft. MLW). Significant overflow of the beach dune thus occurs over a 4+ hour period for a storm moving at a forward speed of 20 knots. For a storm moving twice as fast across the Continental Shelf the duration of overflow would be proportionately less. Perhaps the most critical factor affecting and, to a large extent, controlling the predicted peak PMH surge height at the Oyster Creek site is the condition of dune erosion and consequent tidal overflow. The assumptions made regarding the rate of erosion, resulting beach length eroded, and the total depth of erosion, are considered "reasonable" for an event of this nature and severity. However, those assumptions could be quite easily affected and radically altered; for example, by future construction of a major highway along the beach which might not erode to the extent assumed and which could prevent all but minor overflow to the bay during the PMH. The total overflow volume to the bay was computed to be

662,500 acre feet. If that volume was to be reduced only 30% as a result of a faster moving hurricane, or because of a lesser rate and depth of erosion along the beachfront, the resulting water level in Barnegat Bay would be more than 3 feet lower than that given in Unit 2 PSAR, or at about elevation 13 ft. MLW. While there may be some difference of opinion as to the exact CFI, radius of maximum winds, p_n value and other FMH parameters to be used in the tidal analysis, the most important factor in the entire analysis is the degree and extent of beach erosion that will occur during a FMH event.

3. Evaluate maximum open coast surge height. Several methods are referred to in Technical Report No. 4, "Shore Protection Planning and Design" U. S. Army CERC, Third Edition 1966 for use in computing the maximum open coast surge height. Among those is ref. 171 which is the same as Reference 8 to Unit 2 PSAR....that of Dr. R. D. Reid. The formula and procedures recommended in Reference 8 were those used in Unit 2 PSAR to arrive at the maximum open coast surge height of 19.32 ft. MLW. An evaluation of the various parameter and components used or derived in that analysis is given in this report.

4. Additional wind setup in Barnegat Bay. During the period of FMH surge occurrence along the oceanfront water levels in Barnegat Bay will have gradually risen from the effect of rainfall, tidal overflow of the beach dune, and tidal inflow through Barnegat Inlet.

At the time that the average water level in the bay reaches its maximum of 16.2 ft. M.L.W. ($T+\frac{1}{2}$ hours) peak hurricane winds are directed across the east-west axis of the bay, a distance of some 3 to 4 miles. An additional pile up of water along the mainland shore will occur within the bay across this relatively short fetch distance over and above the mean water level in the bay during the critical hour $T-\frac{1}{2}$ to $T+\frac{1}{2}$. Numerous studies have been made of simultaneous inflow, overflow, and wind setup within such bay: barrier reef: inlet systems; among the more complex are those presented in References 2, 3, and 4. The basic trial-and-error method and procedures used in those reports are relatively the same -- evaluation is made of the presence and effect of submerged barrier reefs by means of submerged weir-type flow relations; of simultaneous tidal inlet flow by use of an orifice formula; of the causative effects of wind stress on both bay and ocean levels; and lastly, of the additive effects of atmospheric pressure, normal tides and wave action on the final elevation reached at shore. Of those effects the additional pile up of water in Barnegat Bay due to wind stress was not determined in Unit 2 PSAR. An estimate of that effect is evaluated as follows.

The Barnegat Bay hydrograph, shown on Figure 2 of Unit 2 PSAR, represents the mean bay level resulting from tidal overflow and inlet inflow. The effect of 0.25 ft. of direct rainfall on the bay (see paragraph 5 below) must be added to the peak mean value of 16.2 ft. M.L.W., giving a value of 16.45 ft. M.L.W. at $T+\frac{1}{2}$ hours. In estimating

the magnitude of local wind effects across the bay from $T-\frac{1}{2}$ to $T+\frac{1}{2}$ hours a 2-mile wide bay section was used, as shown on Exhibit 1, attached. That section fronts the plant site and encompasses both discharge and intake channels, and is considered to be of sufficient width to be representative of average bottom elevations, wind speeds, wind setup, and wave action in the bay area fronting the plant site. An average ground and bay bottom profile was obtained for that section from available U.S.G.S. and C&G.S. topographic maps. That profile is shown on exhibit 2, attached. Ocean surge elevations and mean water surface elevations in the bay at $T-\frac{1}{2}$, T_0 , $T+\frac{1}{2}$ hours are also shown. Peak FM wind speeds across the section in that period would be on the order of 120 mph, with wind directions as shown by arrows on Exhibit 1. The problem of determining wind setup across Barnegat Bay is unique in that during the hour $T-\frac{1}{2}$ to $T+\frac{1}{2}$ the mean water level in the bay rises over 6 feet -- from 10.2 ft. to 16.4 ft. MLW, during which time wind stress on the bay surface is attempting to establish a steady-state setup condition. The "normal" boundary conditions relative to setup-setdown and surface-bottom flow relations, while still present, do not achieve stability in accordance with the classical concepts. Because of the rapidly changing water depths within the critical hour, from overflow and inflow to the bay, it is highly improbable that a true "steady state condition" can be expected to occur, or be attained in the bay. Therefore, in an effort to arrive at a "reasonable" estimate of the range in magnitude of additional pile up at shore due to wind stress across the bay in the period $T-\frac{1}{2}$ to $T+\frac{1}{2}$, computations were made using an "average" water

level in the bay for that period (10.10 ft. + 12.15 ft. + 2 = 13.27 ft. (M.S.L.)) with an average wind speed of 120 mph. The formula and procedures used are those derived in Ref. 5 and applied in Ref. 2 and 3. Trial and error estimates established the node line at approximately mile 1.3 East (see Exhibit 2). The computed setup elevation at the mainland shore, exclusive of wave effect, was determined to be 17.36 ft. M.L.W. (17.06 ft. M.S.L.). During the hour $T-\frac{1}{2}$ to $T+\frac{1}{2}$ the depth of water over the eroding beach dune would reach a maximum of about 3-9 ft. thus allowing waves of up to 6 feet height to pass over the dune. Some attenuation of wave length could result due to the overflow velocities but those waves reaching the bay would reform and be increased in wave height in moving across the bay. In general, it is estimated that an additional 1 foot of water due to the effect of waves breaking landward of the mainland shoreline should be added to the peak setup elevation, making a total PMW bay tide elevation of $\frac{17.36 \text{ ft.}}{\text{MLW}}$ (18.06 ft. MSL).

5. Hurricane rainfall. As noted on page 7 of USWB Technical Paper No. 48 ... "hurricanes may dump as much as 12 inches of rainfall in 24 hours over large areas and even more over areas of a few square miles". In general, the amount of rain resulting from any given storm is a function of several factors ... the moisture content of the storm and influence of surrounding air masses, its path, i.e., whether over relatively flat terrain or mountainous, where the effect of orographic lifting can result in torrential and wide-spread down-

poors, and other meteorologic conditions. There have been accounts of "dry" hurricanes in which rainfall has been extremely light. Others, such as hurricane Diane of August 15-20, 1955, have dumped as much as 10 to 12 inches over an area of 10,000 square miles within several days. Casual examination of rainfall records associated with hurricane passage for the northeastern seaboard indicates that rainfall distribution is usually light along the coast, becoming heavier with rise in land elevation, and heaviest in the upper foothills and mountain areas due to orographic lifting. It is also true that the period of heaviest rainfall in a storm occurs in the vicinity of the storm path and slightly ahead of the center...this is the area of maximum moisture inflow and convergence, and that most affected by orographic lifting. For the P'K a total pre-peak tide rainfall is postulated, ranging from 3 inches along the coast over Barnegat Bay, to a maximum of 12 inches some 25 to 30 miles inland. The net effect of direct rainfall over Barnegat Bay (0.25 ft.) would be its contribution to the depth of water in the bay at the time of peak ocean tide occurrence, and its resultant effect on wind setup in the bay proper. In terms of added depth the contribution due to direct rainfall on the bay would mean about a 1% increase in average bay depth at the time the bay elevation reached its maximum level due to inflow and overflow (0.25 ft./21 feet, estimated average bay depth). No contribution to the bay from upland runoff from such streams as Toms River, Cedar Creek, Forked River, Oyster Creek, Canning River, or Mannanewkin Creek is expected at the time of peak tide occurrence because of the normal 3 to 6 hour lag between

rainfall occurrence and time of concentration in peak runoff from those watersheds. In this regard it has been noted that the effect of high water levels in bays (from tide) can and has delayed or caused a further lag in the time of occurrence of peak runoff owing to reversals in slope upstream. As discussed in paragraph above, the effect of direct rainfall on water levels in Barnegat Bay was added to the mean bay elevation in determining the additional pileup in the bay along the mainland shore due to wind effect across the bay.

6. Extreme Low Tide Analysis. The logic and basic assumptions made relative to determination of the Extreme Low Tide elevation of -3.4 ft. M.S.L. in the bay at the plant site are contained in Unit 2 FSAE. The formula used to compute the setback in the bay within the plant intake and discharge channels is that described in Ref. 5 and applied in Ref. 5, 2, and 3 to shallow lakes and bays. That formula is:

$$S = \frac{L \lambda T_s N}{\gamma D}$$

Where:

- S = total setup over the respective fetch, in feet.
- L = fetch distance, in feet.
- λT_s = tangential wind shear stress (lbs./ft.²).
- γ = specific weight of water (62.4 lbs./ft.³).
- D = average depth of water over fetch L, in feet.
- N = ratio of setup to depth (after Keulegan).

An average bay bottom profile (west to east) was constructed, as shown on Exhibit 3, attached. A 2-mile wide bay section (see Exhibit 1) was used to obtain average bay bottom elevations and for

estimating water volumes. As noted in Unit 2 PSAR wind speeds of 90-95 mph were used. The nodal point in the bay was estimated initially and subsequently finalized by a volumetric check of setup and setback volumes. Outflow from the bay through Barnegat Bay Inlet was considered negligible, assuming the condition of low normal tide in the bay (-0.1 ft. MLW) plus wind setup would be acting against a rising normal ocean tide. Basic data and computed values are given in Tabular form on Exhibit 3.

7. Wave runoff at the plant site. In Unit 2 PSAR it was estimated that a wave runoff value of 1 foot would occur in the vicinity of the plant site. This value is believed to be the maximum runoff that could occur under the following conditions. Although wave heights in the bay during the critical hour of P.M. tide would be on the order 8 to 10 feet, waves moving overland toward the plant site would break as water depths become progressively less. Topographic data indicates ground elevations east of New Jersey state highway route 9 range from 20 feet or higher for at least half-a-mile distance gradually sloping downward toward Oyster Creek and Forked River to about a 6 foot elevation. The presence of trees and native vegetation would have a considerable effect in dissipating wave energy and reducing wave height. Also, any waves reaching the highway must pass under, through, or over the two bridges in the 100 ft. to 140 ft. wide intake and discharge canals. A further energy dissipating factor is the 90° curvature of the canals in the plant site area. Any waves that might possibly reach that curvature would undergo final dissipation.

Wave action generated in either canal along their north-south alignment would be finite because of a 400-500 foot maximum generating fetch distance. The 1-foot wave runup at the plant site is therefore considered to be a conservative value.

Respectfully Submitted,



Theodore S. Haeussner
hydraulic engineer Consultant
Jacksonville, Florida
July 15, 1968

REFERENCES - BIBLIOGRAPHY

1. Jelesnianski, C. P., "Numerical Computations of Storm Surges with Bottom Stress", Monthly Weather Review, Vol. 95, No. 11, November 1967.
2. U. S. Army Corps of Engineers, South Atlantic Division, "Survey Report on Hurricane Protective Measures for Hillsboro Bay, Fla.".
3. U. S. Army Corps of Engineers, South Atlantic Division, "Survey Report on Hurricane Protective Measures for Biscayne Bay, Fla.", Jacksonville District, April 12, 1963.
4. Technical Memorandum No. 83, Reid R. D., "Approximate Response of Water Level on a Sloping Shelf to a Wind Fetch which moves toward shore", EES, June, 1956.
5. U. S. Army Corps of Engineers, South Atlantic Division, Civil Works Investigation Project CW-167, "Waves and Wind Tides in Shallow Lakes and Reservoirs", Summary report, Jacksonville District, June, 1955.

- EXHIBITS:

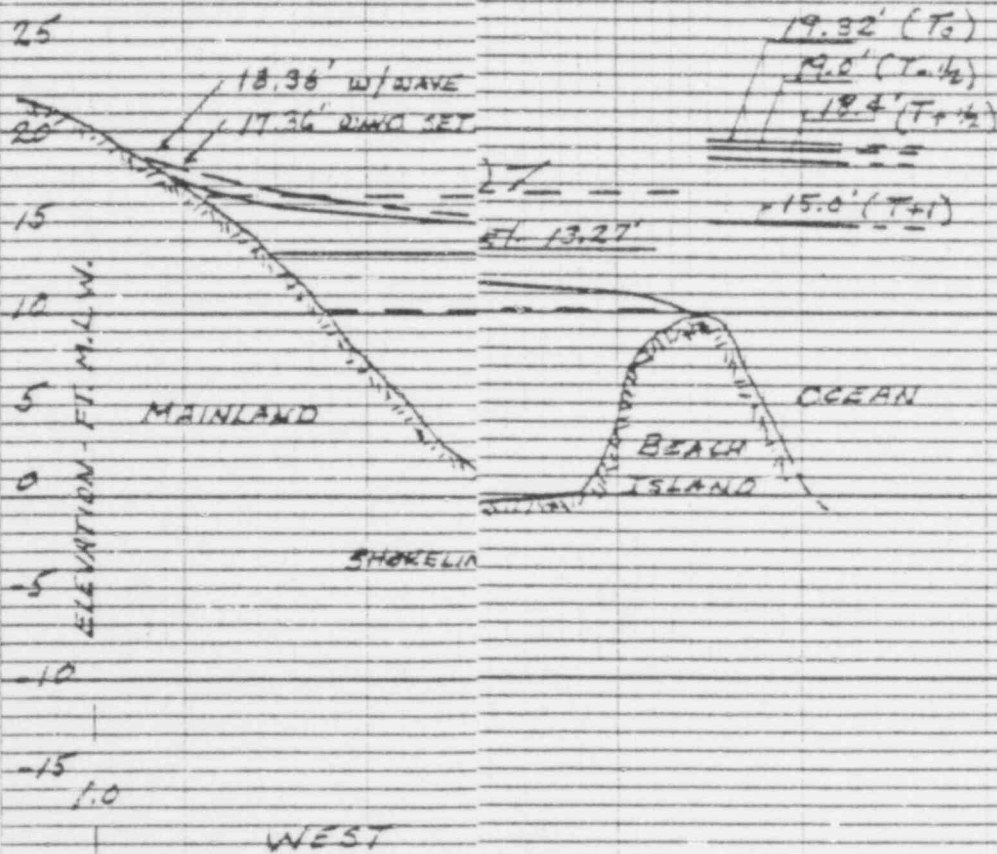
1. Wind setup section - Barnegat Bay.
2. P.M.H. tide profile - Barnegat Bay.
3. Extreme Low Tide profile - Barnegat Bay.
4. Back-up computation sheet for P.M.H. Surge



WIND SETUP SECTION
BARNEGAT BAY

EXHIBIT 1.
86 101

K-E 10 X 10 TO THE INCH 47 0703
U.S. & 15 (11/11/12)
MADE IN U.S.A.
NEUFERS & GARDNER CO.



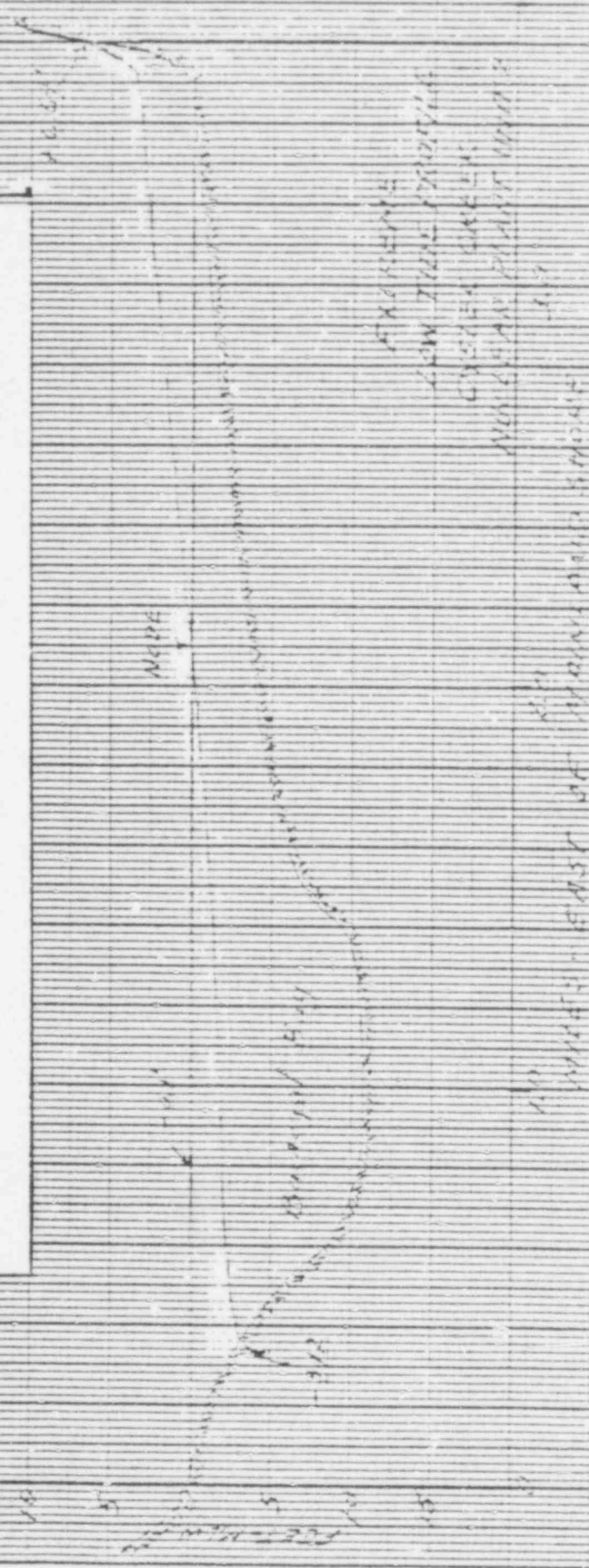
M.H. TIDE PROFILE
VESAT BAY-OYSTER EK.

EXHIBIT 86 102

Mile Section	Fetch (ft.)	V _{avg} (mph)	$\frac{\lambda T_s}{\lambda T_s}$	$\frac{D_{max}}{(ft.)}$	S (ft.)	ΣS (ft.)	Elev. (ft. M/LW) -0.01 (Node True)
2.1 to 2.6	2,640	93.5	0.0995	3.4	1.18	1.18	+1.17
2.6 to 3.2	3,158	94*	0.10037	3.3	1.45	2.63	+2.62
3.2 to 3.5	1,984	95	0.1014	3.3	0.71	3.34	+3.33
3.5 to 3.6*	540	95	0.1014	2.3	3.31	6.65	+6.64 Max.
<u>SETUP PORTION</u>							
1.6 to 2.1	2,640	92.5	0.0988	3.6	1.11	1.11	-1.12
1.4 to 1.6	1,955	92	0.0982	5.7	0.29	1.40	-1.41
0.6 to 1.4	4,224	91.5	0.0978	8.3	0.75	2.15	-2.19
0.33 " 0.6	1,430	90.5	0.0963	2.3	0.93	3.11	-3.12 Min.
<u>SETBACK PORTION</u>							

Handwritten note: *Handwritten*

Handwritten note: *V = 93 mph*



BY T.L.H. DATE 7/13/61 SUBJECT R.M.H. SURGE HEIGHT SHEET NO. _____ OF _____
 CHKD. BY _____ DATE _____ BARNETT BAY - CYSTER CREEK MUD FLAUT JOB NO. _____

TIME: T_{-2} to T_{+2} M.S. ELEV. = $\frac{16.45 + 10.10}{2} = 13.27$ Ft. M.L.W.

(Node Line at Mile 1.6 East)

Mile Section	Fetch Ft.	V_{avg} mph	λT_s	τ_{avg} Ft.	N	S Ft.	ΣS Ft.	Elev. Ft. M.L.W.
<u>SETUP PORTION</u>								<u>13.27 Ft.</u>
1.6-1.4 E	1,050	120	0.2240	19.5	0.99	0.19	0.19	13.46
1.4-0.6 E	4,200	"	"	24.0	1.00	0.63	0.82	14.09
0.6-0.2 E	2,112	"	"	18.5	0.99	0.61	1.23	14.50
0.2E-0.2W	2,112	"	"	14.8	0.98	0.50	1.73	15.00
0.2W-0.6W	2,112	"	"	8.5	0.94	0.84	2.57	15.34
0.6 -0.85W	1,320	"	"	2.5	0.80	1.52	4.09	<u>17.36</u>
<u>SETDOWN PORTION</u>								<u>13.27 Ft.</u>
1.6- 2.6 E	5,280	"	"	25.5	0.99	0.75	0.75	12.52
2.6 - 3.2	3,160	"	"	14.0	0.97	0.78	1.53	11.74
3.2 - 3.6	2,112	"	"	11.3	0.97	0.65	2.13	11.09
3.6 - 3.8	1,056	"	"	2.5	0.82	1.25	3.43	9.34

FORMULA :

$$S = \frac{L \lambda T_s N}{\gamma D}$$

Tide Elevation 17.36 Ft. + 1.0 Ft. Wave Effect =
18.36 Ft. M.L.W. (18.06 Ft. M.S.L.)

EXHIBIT -

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