Westchester Lagoon Dam Analyses
December 10, 2002

Background

The purpose of this work was to assemble additional information to assist Project Management and Engineering (PM&E) in determining the correct hazard classification for the Westchester Lagoon dam. The State of Alaska Dam Safety Program assigns classification to all dams in the Alaska. Depending on the dam's physical characteristics and classification, a given dam may or may not be under the program's jurisdiction. If a given dam is within the program's regulatory jurisdiction, the hazard classification determines the required level of maintenance, monitoring, reporting, and emergency action planning. PM&E is in the process of working with the State Dam Safety Program to determine the correct hazard classification for Westchester Lagoon Dam.

Currently PM&E is working with to the State Dam Safety Engineer, Mr. Charles Cobb, P.E., to determine the hazard classification from class of the dam. PM&E is also a partner in a multi-agency effort to restore all tide access for fish migration between the Westchester Lagoon and Knik Arm. Mr. Cobb has stated that to make a decision on this request he will require additional information on dam stability issues related to both the dam and the proposed of fish passage improvements. This information includes a dam break analysis, a wave height analysis, channel stability analysis, and a dam seepage and stability analyses. These required analyses assume that the proposed fish passage improvements are constructed. Therefore PM&E has requested that HDR perform these four studies. The results of these studies are summarized below. Full reports and calculations are attached to this report.

The Municipality of Anchorage is a partner with the Corps of Engineers in a large-scale aquatic habitat restoration project on Chester Creek. Conceptual design of the habitat improvements were developed by the US Army Corps of Engineers are contained in *Chester Creek 206 Study Aquatic Restoration Project*, HDR Alaska, Inc., December, 2000. The first projects for construction from this report were the fish passage improvements at Westchester Lagoon. PM&E, through funding supplied by the US Fish and Wildlife Service, prepared a preliminary design for the proposed fish passage improvements at Westchester Lagoon. The major components of the fish passage improvements that are relevant to the dam hazard classification and analyses are:

- constructing a new spillway and outlet channel between the lagoon and Knik Arm;
- removing existing creek culverts and constructing a railroad trestle; and
- lowering the two existing sewer force mains and the two petroleum pipelines to below the proposed open channel.

When these improvements are constructed the lagoon will have an effective dam height of 18 feet. The outlet channel will flow between the toe of the railroad embankment fill and the dam. These changes will affect the magnitude of dam break floods, potential wave heights and erosion, embankment erosion and toe stability, and dam seepage and stability. Hence, to determine the correct dam classification these conditions were used as base assumption for all analyses done as part of this work.

Dam Break Analysis Summary

A dam break analysis was performed for the proposed outlet channel and dam configuration. A dam break analysis evaluates the effects of uncontrolled releases due to dam failure. To perform the analysis the National Weather Service program DAMBRK (a dam-break flood forecasting model) was used to model the lagoon and downstream channel through the propose trestle and beyond the location of the proposed utilities. This model was selected because it is an industry standard and was recommended by Mr. Cobb. The project geotechnical investigation [*Geotechnical Report Chester Creek Improvements Westchester Lagoon Fish Passage Anchorage Alaska*, Shannon & Wilson, July 2001] was used to determine the dam type for failure mode selection. Dam breach parameters were developed using Federal Energy Regulatory Commission (FERC) guidelines for earthen dams. These guidelines were selected as they represent an industry standard for such evaluations. The model assumed the following conditions:

- Full lagoon;
- Failure concurrent with 25, 100, and 500-year flood events;
- Granular and silty dam materials;
- Failure during low tide;
- Channel and trestle dimensions from the 30% design report; and
- Breach widths varying between maximum and minimum FERC recommendations.

The dam break analyses results give a maximum velocity upstream from the proposed trestle ranging from 20 to 43 feet per second with peak flows ranging between 7,000 and 18,000 cubic feet per second. The flow range variation was due to varying the selected breach width. Velocities at the trestle, and buried utilities, remained high in all cases. The differences in velocities and flows due to changes in the flood flow rate were minimal. To meet FERC criteria or flow rates and velocities, the middle to high range of breach parameters and middle to low range of failure time should be used for further analyses. The final selected failure scenario should be negotiated with Mr. Cobb and used in the final design of the outlet channel protection material.

Wave Height Analyses Summary

Dam freeboard is required to prevent overtopping of the dam during flood events or high tide. Overtopping of the dam may result in an uncontrolled dam failure.

Because of terrain and the location of the dam, wind has to come from the east to produce a wave that breaks on the lagoon dam face and must occur during the ice-free period. Maximum lagoon waves were assumed to occur during the normal ice-free period, April to October. Using standard wind wave generation formulas, the predicted wave height is 1.3 feet with a maximum wave run up on the 1h:3v dam face of 1.9 feet. These values should be used in design of the new creek spillway and in determining the required dam freeboard.

Waves from Knik Arm can occur any time of the year but are confined to a narrow predominant fetch diction. Using standard wind wave generation formulas and using maximum wind speeds that have occurred from the west, the predicted wave height is 6.1 feet with a maximum wave run up on the 1h:3v dam face of 5.4 feet. These values should be used to design wave erosion protection for embankment faces where they are exposed to Knik Arm wave erosion forces.

Channel Stability Analysis Summary

The proposed outlet channel will be constructed between the dam and the railroad embankment. These embankments must be protected to maintain their stability. To reduce the chance of the channel erosion during flood events, the channel will be lined with rip rap. While the new channel will convey all low flows to Knik Arm, higher flows will be split between the existing outlet weir and the new channel. The spillway and channel are designed to convey the majority of the flow during flood events. Riprap was sized to maintain channel stability during the 500-year, or 0.5% chance, event. Riprap should be ADOT&PF Class II or larger. Riprap lining of the channel must extend to the depth of the 500-year flood.

Seepage and Stability Analysis Summary

This seepage and stability analyses evaluated the proposed dam and constructed channel. The analysis used the geotechnical information contained in *Geotechnical Report Chester Creek Improvements Westchester Lagoon Fish Passage Anchorage Alaska*, Shannon & Wilson, July 2001. No additional field investigations were for this analysis.

Slope stability was evaluated for both static stability and seismic loading using the Makdisi and Seed (1977) procedure. Static condition limit equilibrium analyses were preformed using the modified Bishop method for calculating circular surfaces. Seismic evaluation for initial deformation and liquefaction analyses used a design event for ground motions with a return period of 500 years. Significant deformation risk analyses evaluated the return interval of the earthquake to produce a 1 to 3 foot deformation in the embankment. Groundwater levels were held to the most stringent condition found in the soil borings; i.e. groundwater levels at or above elevation 10. The results of these analyses found that the displacement during a 500-year magnitudes seismic event is about 3 inches. The risk of displacement of an amount that can be expected to cause failure (1 to 3 feet) can be related to a seismic event of greater than 2,000 years. The report concludes that the current design geometry for the channel will not pose a danger to the stability of the lagoon embankment fill.

Seepage was evaluated with a two-dimensional, steady state, numeric model (Fastseep 2D). Model input was developed from the previously referenced geotechnical report and literaure values for hydraulic conductivity. The lagoon was assumed full, water surface at elevation 16, for this analysis. Seepage from the lagoon into the proposed channel was also evaluated using a variation of Darcy's Law as described in the Corps of Engineers Engineering Manual 1110-2-1913. Seepage results indicate that under certain condition the existing face gradient will approach, or slightly exceed, the recommended Corps of

Engineers limit. Either moving the channel further from the dam or decreasing the permeability of the embankment fill can reduce the gradient. Reducing the permeability can be done by constructing a low permeability cutoff below the channel bottom where the channel is adjacent to the dam embankment. This recommendation will be given to the channel designers for incorporation into the final channel plans.

Dam Break Analysis Technical Memorandum

WESTCHESTER LAGOON DAM BREAK ANALYSIS

Chester Creek Aquatic Habitat Restoration ***May 13, 2002***

Reviewed by: Tony Barela

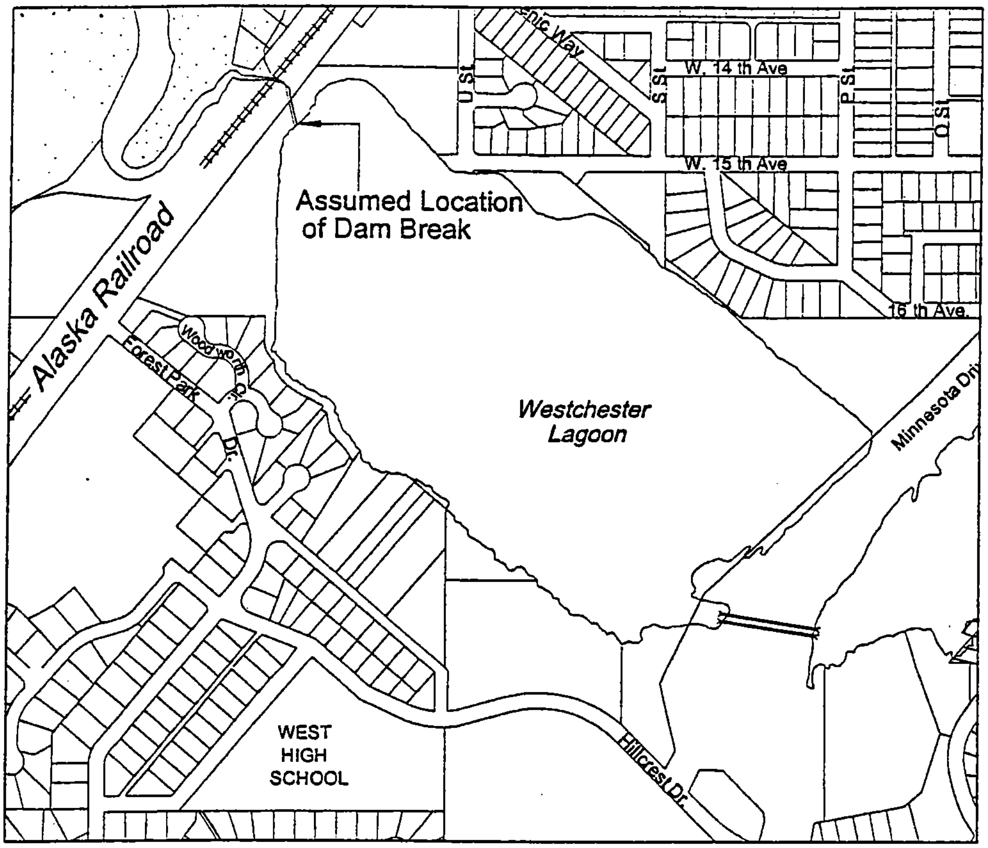
Prepared by: Sara Miller

Background/ Purpose

A Dam Break analysis was performed for the Westchester Lagoon located along Chester Creek in Anchorage, Alaska. A dam break analysis evaluated the effects of uncontrolled releases from the lagoon due to dam failure. The information from this analysis will be used to determine how to protect the proposed downstream rail trestle and utilities. The analyses evaluated break scenarios during summer low flow (25-year storm), 100-year flood event and the 500-year flood event. The dam break routing included runs, which varied the breach parameters to check the sensitivity of assumptions. The results from the modeling include the peak flow and average water velocity at the cross section located immediately upstream of the proposed railroad trestle.

Modeling Methodology and Assumptions

The dam break is assumed to occur near the location of the existing outlet weir and slide gate. Figure 1 below shows the assumed location of the dam break. The dam break is located approximately 250 feet upstream from the proposed Railroad Trestle. The dam break flow was routed through the new channel to the proposed trestle. Dimensions of the channel were taken from a typical section provided by HDR, Alaska. Three cross sections were used to model Chester Creek downstream of the dam break section. The creek outlets to the ocean located directly downstream from the trestle. All modeling assumed low tide conditions.



Lagoon Capacity

The analysis was conducted for a worst-case scenario where the lagoon is assumed to be full (at an elevation of 18 feet) and low tide conditions exist. The reservoir area-capacity curve was calculated by HDR, Alaska using historic lagoon data. The data for the lagoon is provided in Table 1 below.

Table 1. Area-capacity data for Westchester Lagoon

Elevation (feet)	Area (Acres)	Average End Area (Acres)	Volume (Acre-feet)
19	73		
		73	73
18	73		
		73	73
17	73		
		41	41
16	8		
		5	87
0	3		
		[illegible]	[illegible]
		Total Volume =	275

Chester Creek Hydrology

Flood flow rate data for Chester Creek for the recurrence intervals analyzed were provided by HDR, Alaska in their design study report and can be found in Table 2 below.

Table 2. Flood Magnitude for Chester creek at Arctic Boulevard

Recurrence Interval (years)	Discharge (Cubic feet per second)
25	265
100	380
500	559

Breach Parameters

The National Weather Service (NWS) program DAMBRK (A Dam-Break Flood Forecasting Model) was used to model the lagoon and downstream conditions. The model uses standard breach parameters to analyze the effects of a breach on the downstream reach. The breach parameters used in the program are the width of the base of the breach, the elevation of the bottom of the breach, the side slope of the breach and the time to maximum breach size. In order to determine the values of the breach parameters, the Federal Energy Regulatory Commission (FERC) suggested breach parameters were consulted for an earthen dam. Table 3 shows the range of breach parameters recommended by FERC for earthen dams.

Table 3. Suggested Breach Parameters (FERC)

Breach Parameter	Value
Average width of Breach (BR)	2 x Height of the Dam ≤ BR ≤ 4 x Height of the Dam 36 feet ≤ BR ≤ 72 feet
Horizontal Component of side slope of Breach (z)	0.25 ≤ z ≤ 1
Time to failure in hours (TFH)	0.1 ≤ TFH ≤ 0.5

According to FERC for a worst-case scenario, the average breach width should be in the upper portion of the recommended range and the time to failure should be in the lower portion of the recommended range. The average width of the breach was also calculated using an equation developed by Froelich based on the height of the dam and the volume of the reservoir. This method is recommended in the DAMBRK model documentation. The equation is presented below.

$$\overline{BR} = 9.5 \times k_o \times (V_r \times h_d)^{0.25}$$

Where:

$\overline{BR}$ = average breach width

k_o = 1.0 for overtopping

V_r = volume of the reservoir (acre – feet)

h_d = height of the dam

The average width calculated using the above equation is approximately 80 feet.

Model Results

Three trials were run for the 25-year, 100-year and 500-year flood flows. For each of the trials the breach parameters were varied to check the sensitivity. Tables 4, 5 and 6 show the results from the model runs along with the breach parameters used.

Table 4. Results from model runs using 25-year flood flow rate

Ave BR (ft)	z	TFH	Flood Flow rate (cfs)	Max Flow just u/s of Railroad trestle (cfs)	Max Velocity just u/s of Railroad trestle (fps)
80	1	0.5	265	10,633	25.43
54	1	0.5	265	9,468	21.65
36	1	0.5	265	6,760	19.19
80	0.25	0.5	265	10,844	25.76
54	0.25	0.5	265	9,926	23.30
36	0.25	0.5	265	7,422	19.85
80	0.5	0.5	265	10,791	25.69
54	0.5	0.5	265	9,805	23.30
36	0.5	0.5	265	7,200	19.85
80	1	0.3	265	14,767	34.79
54	1	0.3	265	10,627	25.43
36	1	0.3	265	7,037	19.46
80	1	0.1	265	16,999	41.50
54	1	0.1	265	11,455	27.73
36	1	0.1	265	7,399	19.87
80	0.25	0.3	265	15,286	35.95

Ave BR (ft)	z	TFH	Flood Flow rate (cfs)	Max Flow just u/s of Railroad trestle (cfs)	Max Velocity just u/s of Railroad trestle (fps)
54	0.25	0.3	265	11,296	27.22
36	0.25	0.3	265	7,710	20.12
80	0.25	0.1	265	17,650	43.22
54	0.25	0.1	265	12,126	29.72
36	0.25	0.1	265	8,076	20.50
80	0.5	0.3	265	15,121	35.58
54	0.5	0.3	265	11,075	26.22
36	0.5	0.3	265	7,485	19.91
80	0.5	0.1	265	17,433	42.65
54	0.5	0.1	265	11,902	29.15
36	0.5	0.1	265	7,849	20.29

Table 5. Results from model runs using 100-year flood flow rate

Ave BR (ft)	z	TFH	Flood Flow rate (cfs)	Max Flow just u/s of Railroad trestle (cfs)	Max Velocity just u/s of Railroad trestle (fps)
80	1	0.5	380	10,883	25.91
54	1	0.5	380	9,684	23.13
36	1	0.5	380	6,810	19.24
80	0.25	0.5	380	11,189	26.42
54	0.25	0.5	380	10,164	23.64
36	0.25	0.5	380	7,470	19.90
80	0.5	0.5	380	11,095	26.28
54	0.5	0.5	380	10,011	23.54
36	0.5	0.5	380	7,249	19.69
80	1	0.3	380	14,894	35.08
54	1	0.3	380	10,684	25.53
36	1	0.3	380	7,072	19.50
80	1	0.1	380	17,050	41.64
54	1	0.1	380	11,493	27.81

36	1	0.1	380	7,426	19.89
80	0.25	0.3	380	15,467	36.32
54	0.25	0.3	380	11,354	27.34
36	0.25	0.3	380	7,745	20.15
80	0.25	0.1	380	17,698	43.35
54	0.25	0.1	380	12,162	29.81
36	0.25	0.1	380	8,100	20.52
80	0.5	0.3	380	15,278	35.91
54	0.5	0.3	380	11,130	26.36
36	0.5	0.3	380	7,526	19.94
80	0.5	0.1	380	17,482	42.79
54	0.5	0.1	380	11,938	29.24
36	0.5	0.1	380	7,874	20.32

Table 6. Results from model runs using 500-year flood flow rate

Ave BR (ft)	z	TFH	Flood Flow rate (cfs)	Max Flow just u/s of Railroad trestle (cfs)	Max Velocity just u/s of Railroad trestle (fps)
80	1	0.5	559	11,277	26.67
54	1	0.5	559	10,068	23.80
36	1	0.5	559	6,890	19.32
80	0.25	0.5	559	11,593	27.19
54	0.25	0.5	559	10,477	25.15
36	0.25	0.5	559	7,549	19.98
80	0.5	0.5	559	11,470	26.99
54	0.5	0.5	559	10,346	24.15
36	0.5	0.5	559	7,328	19.76
80	1	0.3	559	15,080	35.80
54	1	0.3	559	10,778	25.75
36	1	0.3	559	7,189	19.57
80	1	0.1	559	17,133	41.89
54	1	0.1	559	11,555	27.95

Ave BR (ft)	z	TFH	Flood Flow rate (cfs)	Max Flow just u/s of Railroad trestle (cfs)	Max Velocity just u/s of Railroad trestle (fps)
36	1	0.1	559	7,471	19.93
80	0.25	0.3	559	15,672	36.97
54	0.25	0.3	559	11,448	27.53
36	0.25	0.3	559	7,804	20.21
80	0.25	0.1	559	17,777	43.57
64	0.25	0.1	559	12,219	29.96
36	0.25	0.1	559	8,140	20.56
80	0.5	0.3	559	15,475	36.59
54	0.5	0.3	559	11,226	26.59
36	0.5	0.3	559	7,581	20.00
80	0.5	0.1	559	17,563	43.01
54	0.5	0.1	559	11,997	29.41
36	0.5	0.1	559	7,916	20.35

Conclusions

The results from the dam break analysis give a maximum velocity just upstream from the proposed railroad trestle ranging from 20 to 43 feet per second (fps) with flows ranging from 7,000 to 18,000 cubic feet per second (cfs). While varying the breach parameters did affect the results, the velocities tended to remain high for all of the trials. The difference in the velocities and flows due to the change in the flood flow rate were minimal. The force of the volume of water stored in the reservoir had a greater effect on the dam break results than the flood flows. To meet FERC criteria flow rates and velocities in the middle to high range should be used for any further analysis.

Channel Stability Technical Memorandum

To:	Memo to File
From:	Jason Kent
Date:	July 18, 2002
Subject:	Chester Creek Riprap Design

HDR

Memorandum

This memo documents the methodology used to check the riprap sizing performed by Inter-Fluve, Inc. for Chester Creek downstream of the Westchester Lagoon Dam.

Inter-Fluve calculated riprap size for the 500-year flow event during low tide. While water surfaces are lowest during low tide, this situation maximizes the stream velocity. Inter-Fluve designed riprap with "D84 equal to 15-inches for rounded rock placed along the bed and D84 equal to 33-inches for subangular rock placed along a 2H:1V stream bank."

HDR checked the riprap selection by Inter-Fluve using calculations and methods prescribed in HEC-11, Design of Riprap Revetment. Appropriate calculation sheets and nomographs accompany the hard copy of this memorandum.

Riprap size was calculated using equations 6 and 7 in HEC-11. Angle of repose (ϕ) was estimated at 36° for rounded rock and 39° for subangular rock. Main channel velocity (10.10 ft/s), average depth (3.95 ft), and bank angle (2H:1V) were taken from the HEC-RAS model submitted by Inter-Fluve with their technical memorandum. Uncorrected D50 for both rock types were calculated at 0.85 ft for the subangular rock and 0.95 ft for the rounded rock. The subangular rock D50 was multiplied by a stability factor (SF) of 1.8 to account for ice buildup in the winter. The corrected D50 for the subangular rock on the channel banks is 1.53 ft. The SF was 1.0 for the channel bottom, so the D50 for the rounded rock remains 0.95 ft.

Comparing the Inter-Fluve riprap design to the calculations summarized above, the riprap sizing recommended by Inter-Fluve appears to be appropriate based on the ADOT gradation given in Standard Specifications for Highway Construction (1998, metric), Section 611. The rounded, channel bottom rock is classified as Class II gradation and the subangular, stream bank rock is classified as Class III gradation.

Recommended follow-up items include verification of the input variables – main channel velocity, average depth, and bank angle. It should be noted that wave action was not taken into consideration during these calculations.

Wave Height Technical Memorandum

Job No. 08438-020-249 | No.

HDR Computation

Project:	Westchester Lagoon Outlet Rehab	Computed:	JJK	Date:	6/27/2002
Subject:	Wave runup analysis	Checked:	DB	Date:	6/26/2002
Task:	Linsley et al. 1992	Sheet:	1	Of:	1

Assumptions

1. Assumed average depth - 5 ft
2. Maximum 2-minute sustained wind = 40 mph, from 30°
3. Wind direction is such that fetch is maximized.
4. Assumed bank slope - 1:3

Assume Depth = 5

Wind Setup

$Z_s = V_w^2 F / F_1 d$

	ft	m
Z_s =	0.40	0.04

Wave height

$Z_w = F_2 V_w^{1.06} F^{0.47}$

	ft	m
Z_w =	1.28	0.39

Wave Runup

$\lambda = F_3 t_w^2$

$t_w = F_4 V_w^{0.44} F^{0.28}$

	ft	m
tw (sec) =	1.94	1.90
λ	19.24	5.63
Zw/l	0.07	0.07
Embankment slope	1:3	

From Figure 7.16, enter z_r/z_w: 1.5

	ft	m
Runup height	1.91	0.58

Wind Velocity conversion

V_{water}/V_{land}	1.08	
V_{max}	40	Maximum 2-minute wind velocity for Apr-Oct period, recorded Oct 1966, from 30°

Variable	Definition	Value (US)	Value (SI)	
V_w	(1)	43.20	69.52	Maximum 2-minute wind velocity for Apr-Oct period, recorded Oct 1966
F	(2)	0.46	0.74	Maximum length of lagoon, scaled from USGS Quad Anchorage A-8 NW
F_1	(3)	1400	63200	
F_2	(3)	0.034	0.005	
F_3	(3)	5.12	1.56	
F_4	(3)	0.46	0.32	

(1) Wind velocity 25 feet above water surface (NCDC Climatological records), corrected to over water. (mph, kmh)
(2) Fetch (mi, km)
(3) Coefficients

HDR Computation

Job No. | No.

HDR

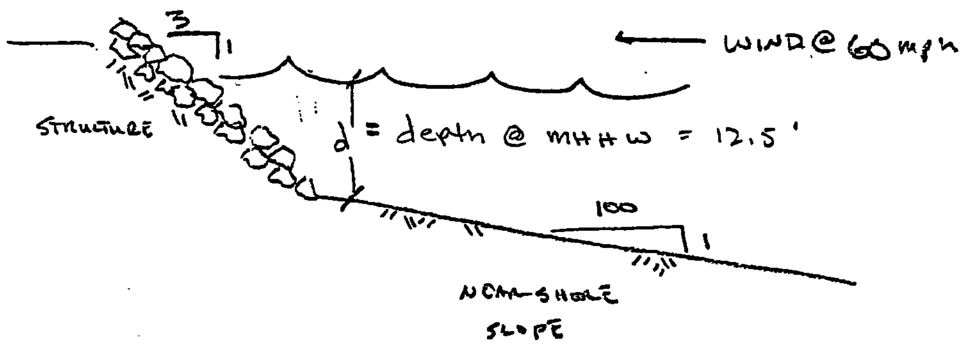
Project: COOK INLET WAVE CALCULATION | Computed: TBB | Date: 12/6/92

Subject: WAVE HEIGHT ; RUNUP | Checked | Date

Task: VIA ACES | Sheet | Of

ASSUMPTIONS:

- MAX WIND IS MAX WIND RECORDED IN JAN 1971 OF 60 MPH
- ASSUME THIS IS IN DIRECTION OF LONGEST FETCH.
- ASSUME FETCH OF 15 MILES
- ASSUME



FROM ACES:

H_{M0} = WAVE HEIGHT = 6.1 FEET (DEEP WATER = 6.75')

T_p = PERIOD = 4.76 SEC

R = WAVE RUN UP = 5.4 FEET