Appendix HMC-02
Engineering Study on Rockfill Dam Stability

SUMMARY
STABILITY ANALYSIS
FOR
ROCKFILL DAM
HALF MOON COVE TIDAL PROJECT

INTRODUCTION

The dam section analyzed is the section shown in the MAIN Report, "Half Moon Cove Tidal Project, dated November, 1980, Drawing HMC-1, General Plan. The proposed construction scheme would install a cellular cofferdam from the Perry side of the project beyond the midpoint of the channel. Inside this cofferdam, the power units, gates and Perry side embankment would be constructed. The embankment section would be the same as for the Eastport side, except in the case of Perry, the embankment would be constructed using standard compaction techniques.

The Eastport side embankment, the subject of this study, will utilize dumped rockfill shells and dumped (sluiced) core and filter zones. Our analysis looked at both static and dynamic cases (o.1g - recommended for the project area).

GEOLOGY AND SEISMICITY

Geology and Foundation Conditions

The subject project includes a dam and a powerhouse located in the same general area and alignment of the former Bar Harbor Bridge connecting Perry and Eastport. Published geologic information available for this site area includes the following:

(1) "Geologic Map and Cross Sections of the Eastport Quadrangle, Maine"; Olcott Gates, Maine Geological Survey, 1975. Text accompanying the map includes 19 pages of description and bibliography.

(2) Preliminary Geologic Map of Maine, Maine Geological Survey, 1967.

(3) Sketch of bedrock profiles at the site, by Maine State Department of Highways and the International Joint Commission on Passamaquoddy Power, 1959.

(4) "Investigation of the International Passamaquoddy Tidal Power Project"; Report by the International Passamaquoddy Engineering Board, Appendices 1, 2 and 3, October 1959.

(5) Plan and Elevation of the Bar Harbor Bridge, Maine State Department of Highways, 1923.

(6) Daily records of pile driving, Bar Harbor Bridge, Maine State Department of Highways, 1923.

Bedrock Geology

All of the rocks in the vicinity of the site were apparently derived from the aerial and sub-aerial deposition of volcanic extrusives and eruptives, including lavas and volcanic ash, and sediments resulting from their erosion. The deposition of the materials was random in space and time, in turn resulting in coupled stratigraphy and lithology. In addition, once the strata were consolidated, strong tectonic forces caused numerous faults and some folding.

On the southeast (Eastport) abutment at the proposed site, the bedrock is assigned to the Eastport formation of lower Devonian age. These layers are classified as devitrified vitrophyre and breccia with rhyolite. Considerable hydrothermal alteration has occured. At the the northwest (Perry) abutment, the bedrock is the Perry formation of upper Devonian age, composed in this area of maroon cobble and pebble alluvial conglomerate and arkosic sandstone. It includes clasts of granites and the underlying volcanic rocks.

According to the detailed quadrangle map, the contact between the Eastport and Perry formations is a pronounced unconformity, which apparently passes under the axis of the proposed dam at an obtuse angle near the northwest abutment. The major portion of the dam will be founded on rocks of the Eastport formation, as will the powerhouse, and only a minor portion of the northwest end of the embankment will be founded on strata of the Perry formation.

According to records of the Maine State Highway Department and the International Joint Commission, the bedrock surface ranges from exposure at sea level to as low as 37 to 38 feet below sea level in the middle of the channel where the powerhouse would be located. The slope from sea level to maximum channel depth is slightly steeper on the northwest abutment than is the slope on the southeast side. The

Maine State Highway Department profile is generally somewhat lower in elevation than that of the International Joint Commission, as much as 15 feet in some cases. In all probability this may be accounted for by a lateral shift in profile location and alignment, since the rock surface is obviously very irregular. The type and thickness of any overburden in the channel is not known. Records of the wooden piles driven for the Bar Harbor Bridge in 1923 give only dates driven and total length of piles. There is no record of tip and top elevations, nor of the blow counts. A few notes indicate that some of the piles reached refusal on "ledge", but most have no such descriptions.

Structural deformation of the rock indicates some tilting of strata in blocks, bounded by faults which trend north to northeast in the vicinity of the site. The nearest mapped fault strikes NNE and passes by the northwest abutment at a distance of about one quarter of a mile. Previous reports indicate that all of the faults were active during the Acadian orogeny, but may not be classified as quiescent. Recent conversations with personnel of Weston Geophysical Engineers Inc., who have been involved in detailed research of the seismicity of the region, including the establishment of a network of seismic recording stations, indicate that there is a strong possibility that the Oak Bay Fault, which may be traced from Calais, Maine to the vicinity of Eastport, and which generally follows the course of the St. Croix River, is active, and that the nest of tremor epicenters which occur northeast of the site in Pasamaquoddy Bay result from intermittent slippage along this fault. This is discussed more fully under the regional seismology section. Other structures include joints, which vary in both pattern and spacing depending on the type of rock and on the forces to which it has been subjected. Neither patterns nor spacings can be determined without field measurements.

Overburden in Channel

As far as can be ascertained, no borings have been made in the channel at the site, nor have seismic surveys been performed. Extrapolation of results of both borings and seismic profiles from investigations of the bottom in Carrying Place Cove, southeast of Bar Harbor Bridge, revealed that the material overlying rock was generally granular with minor silt, sand and clay. Since the area where these borings were made is not subjected to the strong tidal scour which occurs daily in the Bar Harbor channel, a correlation with conditions in Bar Harbor channel is in considerable doubt.

On the other hand, the 1923 pile driving records indicate that the piles were driven into some unconsolidated material in most instances. The nature of the material is not known, but it is possible that some compacted till or ground moraine overlies the rock in the channel, at least in some areas.

The plan and elevation of the Bar Harbor Bridge show that as much as 15 feet of overburden was penetrated by some of the piles before refusal by either bedrock or friction. It is doubtful if the same profiles could be obtained today along the same alignment, considering the 60 years, since 1923, of strong currents which alternately remove and deposit sediments.

While details of the foundation rock quality and the effects of the previously mentioned hydrothermal alteration cannot be determined without field investigations with specific reference to this site, it is expected that the bedrock in the channel and on the abutments would provide a stable and safe foundation for any of the project facilities. Due to the inherent vagaries of eruptive and intrusive volcanic deposition, the quality of the rock can vary significantly, both vertically and horizontally, within a very short distance. Prior to expenditure of any engineering effort, it is recommended that a thorough field reconnaissance of the site area be made by an experienced geologist.

Regional Seismicity

A number of sources were consulted to ascertain the earthquake history of the region. Included were the following publications:

1) "Earthquake History of the United States", Publ. 41-1, with supplement 1971-1980, reprinted 1982, U.S. Department of Commerce and U.S. Department of the Interior, U.S. Geological Survey.

2) "Compilation of Earthquakes, Northeastern U.S. and Eastern Canada, 1534 to 1964", Weston Geophysical Research Inc., Westboro, Mass.

3) "Catalogue of Significant Earthquakes, 2,000 B. C.-1979", World Data Center for Solid Earth Geophysics, 1981.

4) "Global Tectonics and Earthquake Risk", Cinna Lomnitz, Volume 5 Developments in Geophysics, Elsevier Scientific Publishing Co., New York, 1974.

5) "Maximum Acceleration in Rock in the Contiguous U.S.", Algermisen and Perkins, Open FIle report 74-416, U.S. Geological Survey, 1976.

6) "World Atlas of Seismic Zones and Nuclear Power Plants", Wyle laboratories, 1979.

7) Earthquakes in Maine, Oct. 1975-Dec. 1982. A map, published by the Maine Geological Survey, showing dates, location and magnitude of tremors, compiled by LePage and Johnston, Maine Geological Survey, January 1983.

In addition, Weston Geophysical Engineers Inc. of Westboro, Mass. kindly furnished information on the activity of the Oak Bay fault which passes under or very near to the Village of Eastport. For the purposes of this report, the most detailed and reliable information on tremors is obtained from the cited Maine map, since both epicenters and magnitudes are based on instrumental recordings. Three recording stations are located within 25 miles or less of the proposed project site: one at Cooper Hill, about 20 miles NNW; another at East Ridge, about 15 miles due west; and another at East Machias, approximately 26 miles SSW. While only 7 years of record is available from this map, the epicentral locations and the magnitudes afford a good general picture of the seismic activity which is not altered significantly by other sources which have a much longer period of record.

Between the year 1977 and the end of 1982, 21 tremors occurred within a radius of 25 miles of the site, 5 between the 25 and the 50 mile radius, and 30 between the 50 and the 100 mile radius.

Of the 21 within the 25 miles radius, which includes 6 in New Brunswick, only 5 were of magnitude of 2.0 to 2.9, while none were of greater magnitude. Equivalent Mercalli Intensities would be from less than I to less than III. An intensity of I is described as "Not felt except by a very few at rest under especially favourable circumstances", while an intensity of II is "Felt only by a few people at rest, especially on upper floors of buildings. Delicately suspended objects may swing." Horizontal ground acceleration in "g's" is less than 0.005.

In the 25 to 50 mile radius zone, two of the 5 have been assigned magnitudes of 2.9 or less, while the other three are less than 2.5 or no magnitude is given.

Since 1638 many earthquakes with epicenters far removed from the Eatsport area were probably felt there, although since the area was not settled until the 18th century, no records exist. A listing of some Northeastern Region earthquakes follows by date.

	Date	Location	Intensity	
1.	June 11, 1638	Probably St. Lawrence Valley	IX	Major
2.	April 14, 1658	New England	VII	"
3.	Feb. 5, 1663	St. Lawrence Region	X	"
4.	Sept. 16, 1732	St. Lawrence Region	IX	"
5.	Nov. 18, 1755	East of Cape Ann	VIII	"
6.	Oct. 22, 1869	Bay of Fundy	VIII	"
7.	Feb. 28, 1925	St. Lawrence Region	M-7.0	"

The following are classified as intermediate to minor earthquakes:

8.	May 22, 1817	Central Maine (widely felt)	VI
9.	Feb. 8, 1855	Canada (felt in Maine)	VI
10.	Oct. 17, 1860	Canada (strong at Saco, ME)	VIII- IX
11.	Feb. 27, 1874	S.E. Maine (severe shock @ Eastport)	V
12.	Dec. 31, 1882	Boundary of Maine & New Brunswick (felt at Eastport)	V
13.	March 22, 1896	Calais, Maine	IV-V
14.	March 21, 1904	S.E. Maine (strongest @ Calais & Eastport)	VII
15.	Dec. 11, 1912	Maine & New Brunswick	V-VI
16.	Jan. 13, 1914	Maine & New Brunswick (felt @ Calais)	V
17.	Nov. 18, 1929	Grand Banks (felt throughout N.E.) Atlantic cables ruptured, tsunami	X

18.	Jan. 7, 1931	Canada	(felt in Maine)	IV
19.	Dec. 20, 1940	Lake Ossippee, N.H.		VII
20.	Jan. 14, 1943	Dover-Foxcroft, ME		V
21.	Apr. 26, 1957	Near Coast of ME	(Portland area)	VI
22.	July 23, 1966	Jonesport, ME		V

The above listing is probably not complete for the tremors which may have affected the Eastport area. On the other hand, it is highly probable that a good proportion may not have been felt there.

Although no records are available to indicate that earthquake damage in the Eastport area has ever exceeded the plaster cracking, window breaking, or chimney felling stage, which correlates with Mercalli Intensities of VI to VII, the potential for a damaging earthquake exists in the nearby Oak Bay fault. Contrary to the normal regional northeast trend of the geologic structure, this particular fault may be traced from north of Calais south to Eastport on a NNW-SSE alignment. It apparently intersects with several faults aligned in the normal trend. Due to the fact that it follows the valley of the St. Croix River and Passamaquoddy Bay, few if any traces may be located at the surface. The records obtained between 1977 and 1982 would suggest that adjustment along the Oak Bay and intersecting faults to accommodate stress accumulation is a continuing process and will continue during the life of the proposed Half Moon Cove Project.

These and other data have been taken into consideration by Algermissen and Perkins, who classify the Eastport area as "having a 90% probability of not having an horizontal acceleration (in rock) of more than 0.06 to 0.08 g, within a 50 year period". While earthquakes generating such tremors may damage ordinary structures, especially those founded on overburden, they are expected to have no effect on well designed structures founded on bedrock.

The hazard from a destructive earthquake which could breach the dam is unusually low. If it occurred during a slack tide, water would either run into or out of the Half Moon Cove basin, either filling or emptying the reservoir. Since there would be no structures "upstream or downstream" of the tidal dam, the only damage other than to the structures could be to boats if any were present in the cove or the bay at the time. Damage to the dam itself could be more severe after a breaching, due to the head differentials between high and low tides, and strong scouring action through a breach.

The final earthquake design criteria selected for the site structures must depend on the estimate of probability of a damaging earthquake, and the economic consequences. The most reliable estimate of probability available at this time is that of Algermisen and Perkins, who suggest that for any 50 year period there is a 90% probability that an horizontal acceleration in rock of more than 0.06 g to 0.08 g is not to be exceeded.

It is recommended therefore, since structures and most of the dam will be founded on rock, that a "g" factor of 0.1 be used for horizontal acceleration, and two-thirds of that, or 0.066 g, for vertical acceleration.

SECTION PROPERTIES

We have no site specific geologic data for this project. Our review of published geologic information as well as our experience in the general area, suggests that a major portion of the dam and powerhouse will rest on rocks of the Eastport formation and or the Perry formation, or on overburden material varying from clays to sand. For our analysis, we have assumed a till with a relatively low (conservative) friction angle. The material properties of the section analyzed are shown on Drawing 3571-2-1 Sketch-1. Because this dam will be constructed of dumped materials, we have selected low (conservative) strength parameters. Since the Perry side embankment, will be a compacted dam of similar materials, it will have a computed safety factor significantly higher than those calculated for the Eastport embankment.

METHODS OF ANALYSIS

a. General: A variety of computational procedures are available for estimating the stability of an embankment fill. In general, most methods currently used give approximately the same evaluation of the overall Factor of Safety against Sliding (FS_s). The choice of procedure therefore depends on the method the designer is most familiar with. For the analysis reported herein, two procedures were adopted, namely, the Simplified Bishop circular arc procedure and a wedge method proposed by Seed and Sultan. The following paragraphs describe the general procedures of these methods of analysis.

b. Simplified Bishop Procedure: The Bishop procedure analizes circular arc failure surfaces by the method of slices. The simplified Bishop procedure neglects consideration of inter-slice forces (thus simplified) based on extensive computational experience that inclusion of the inter-slice forces will not significantly improve the estimate of FS_s for practical problems. This is generally a conservative assumption. The procedure adopted by MAIN is as outlined in "The Use of the Slip Circle in the Stability Analysis of Slopes," A.W. Bishop, Geotechnique, Vol. 5, 1955, pp 7-17. It is further discussed in "Use of Computers for Slope Stability Analysis," R.V. Whitman and A.M. Bailey, Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, July 1967.

The simplified Bishop method of analysis has been used to develop MAIN's in-house computer program SLOPE which was used for the circular arc portion of the analyses reported herein. The program handles any section that can be defined by up to fifty lines and up to 10 concentrated loads. An initial trial failure arc is specified by center coordinates and radius to begin the solution. The program then automatically searches for the critical failure surface by changing the initial center coordinates and radius by 5-foot increments (other increments can be specified) converging toward the center and radius defining the critical failure surface. The program stops when the least FS_s value is determined within a specified tolerance (generally 0.01) and then produces specified output results.

The SLØPE program output includes data on each trial failure surface checked including its FS_s value; the critical surface center coordinates, radius and FS_s as well as a graphical presentation of the input section with critical surface overlaid. The graphical presentation includes the locus of least FS_s center coordinates checked from the initial input value to the most critical center found.

Two methods are provided in the program to define pore water pressure conditions within the section. The most common method used is to input the phreatic and free water surfaces as part of the straight lines defining the section analyzed. The program then computes pore water uplift pressures as the vertical distance from the phreatic surface to the failure arc times the unit weight of water. This method is correct for horizontal water surfaces but highly conservative for steeply dipping phreatic surfaces.

The second method of defining internal pore water pressures for an embankment section is to zone the section with additional lines defining contours of equal r_u values. r_u is a parameter equal to the ratio of pore water pressure to total vertical stress within the fill. This second procedure requires an estimate of the embankment flow net and/or construction pore pressure responses to permit proper zoning of the structure.

c. Wedge Method: The wedge method of analysis adopted herein is based on a procedure proposed in "Stability of Sloping Core Earth Dams" and "Stability Analysis for a Sloping Core Embankment", companion papers by H.B. Seed and H.A. Sultan, Journal of the Soil Mechanics and

Foundations Division, American Society of Civil Engineers, Volume 93, July 1967. The method is used on failure surfaces defined by a series of straight lines and vertical interblock faces.

Solutions for individual failure surfaces are developed by a trial and error, graphical solution of cascading force polygons. The FS_s value of the surface is assumed, orientation of all external forces for each block are defined and the cascading polygon constructed. If the polygon does not close, then a new FS_s is assumed, new force orientations defined and a new polygon constructed. By trial and error, FS_s values are adopted until an exceptable value can be interpolated from the results. Additional surfaces are analyzed by the same procedure and the critical surface defined by interpolation of all solutions.

The most significant assumption in the wedge method procedure relates to the orientation of interblock and resultant resistant forces. There are two types of resistant forces acting to stabilize a soil mass. Cohesive soils under undrained conditions such as clay, exhibit shearing resistances equal to the Mohr Coulomb cohesion intercept (C'). Materials that are granular and exhibit a drained behavior such as sands derive shear strength from intergranular friction. In terms of Mohr Coulomb failure theory the frictional resistance is defined by the friction angle (Ø'). It is assumed that forces tending to drive the blocks are resisted either by cohesion or friction.

Materials that derive their strength from frictional resistance, exhibit forces that are oriented toward their corresponding block faces by an angle defined as the mobilized friction angle Ø' m. This angle is equal to:

$$\phi m' = \mathrm{Tan}^{-1} \frac{(\mathrm{Tan}\ \phi')}{FS_s}$$

where Ø' is the available effective friction angle of the material.

Materials which derive their strength from cohesion exhibit resisting forces parallel to the failure surface of the block. The resisting force is termed mobilized cohesion (C'_m) and defined as:

$$C'_m = \frac{C' \times L}{FS_s}$$

where (C') is the available undrained shear strength of the soil. Thus, for each new trial FS_s adopted during the solution, frictional resisting forces change orientation and cohesive resistant forces change magnitude. By adopting various FS_s values, the cascading force polygons can be closed by trial and error.

The options for pore water pressure assumptions for the wedge method of analysis are the same as those available for the simplified Bishop procedure.

RESULTS OF ANALYSIS

Dike stability results are presented on Plate 1. Three loading cases were evaluated:

Case 1 - Maximum Headwater Elevation (+13.2) equal to Maximum Tailwater Elevation

Case 2 - Minimum Headwater Elevation (-13.0) equal to Minimum Tailwater Elevation

Case 3 - Maximum Headwater Elevation (+13.2) and Minimum Tailwater Elevation (-13.0)

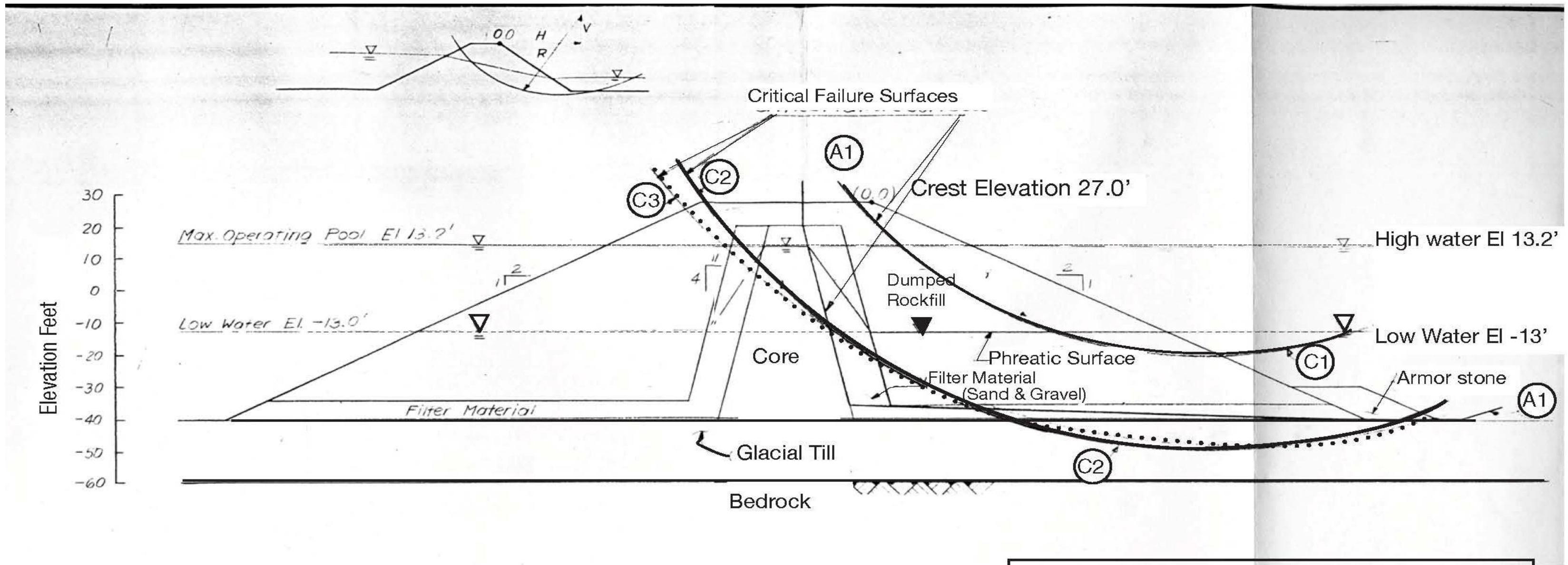
These loading cases represent the maximum and minimum operating conditions that would effect the dike. Dike stability was evaluated under static and design earthquake (a_h = 0.1g) conditions. MAIN selected lower bound limits for soil strength properties, as shown on Plate 1, leading to a lower bound estimate of the Factor of Safety (FS) against sliding.

Case 1 - The minimum FS for surface C-1 under static conditions is 1.74 and under the design earthquake conditions is 1.07. This loading condition is the most extreme in terms of calculated FS.

Case 2 - Failure surface C-2 illustrates a static FS equal to 1.71 and the seismic FS equals 1.13 for the minimum head and tailwater level condition with water perched in the dike core material.

Case 3 - The FS of surface C-3 equals 1.70 for static conditions and 1.12 for seismic loading. Wedge block failure surface A-1 exhibits a static FS of 1.65 and an earthquake FS equal to 1.45.

The table in the bottom left hand corner of Plate 1 lists the critical failure arcs and FS associated with the three loading cases. All reported static FS are greater than the U.S. Corps of Engineers Standard of 1.5 for normal operating conditions. In addition, FS under design earthquake conditions are above the Corps standard of unity.



stab. condition	surface#	FS	H	V	R	Loading Case
Static	C1	1.74	96	85	129	loading case1
Earthquake	C1	1.07	96	85	129	Case 1 $a_h = 0.10g$
Static	C2	1.71	96	95	174	loading case 2
Earthquake	C2	1.13	96	95	174	Case 2 $a_h = 0.10g$
Static	C3	1.70	106	130	209	loading case 3
Earthquake	C3	1.12	106	130	209	Case 3 $a_h = 0.10g$
Static	A1*	1.65	-	-	-	loading case 3
Earthquake	A1*	1.45	-	-	-	Case 3 $a_h = 0.10g$

* Wedge block analysis

material	γ_{wet} pcf	γ_{sat} pcf	c psf	ϕ
core	115	115	0	15°
rockfill	120	140	0	40°
?	110	130	0	30°
?	120	140	0	40°
glacial till	140	140	0	33°
bedrock	167	167	10000	63°

30' 0 30' 60'

HALF MOON COVE
TIDAL POWER PROJECT
DIKE STABILITY

PASSAMAQUODDY TRIBAL COUNCIL

MAIN 1893

DATE MAY 1983

CLIENT	JOB	PLATE
3571	2	1