Appendix E

"Cement and Concrete Studies on the Passamaquoddy Tidal Power Project 9,

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Cement and Concrete Studies on the Passamaquoddy Tidal Power Project*

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In connection with the Passamaquoddy Tidal Power project, the design and construction of which were undertaken by the Corps of Engineers in May 1935 as a major work relief project under an allotment of funds from the Emergency Relief Appropriation of 1935, extensive cement and concrete studies were made.

DESCRIPTION OF PROJECT

Although small tidal power mills have been in operation in this country and in Europe for centuries, the Passamaquoddy project is the first large-scale tidal power project ever to have been undertaken. The Quoddy project was to harness the power in the high Fundy tides which prevail in the vicinity of Eastport, ranging from apogean neap tides of less than 9 feet to extreme perigean spring tides of almost 27 feet with mean tidal range of 18.1 feet. As may be generally known, it was the plan of Dexter P. Cooper, the original proponent of the project, to enclose Cobscook Bay and Passamaquoddy Bay, arms of the Bay of Fundy, from the latter bay by a series of dams, gate-structures and navigation locks. Between Passamaquoddy and Cobscook Bays a series of darns and a power house were to be constructed. (Fig. 1).

The plan of operation was that at and near high tide the gates to Passamaquoddy Bay would be opened and the Bay filled to near high tide levels. As the tide receded the gates would be closed. Conversely at and near low tide, the gates to Cobscook Bay would be opened and the Bay drawn down to near low tide elevation. Thus there would be a head produced between the two pools, available, through the interconnecting power house, for the production of energy.

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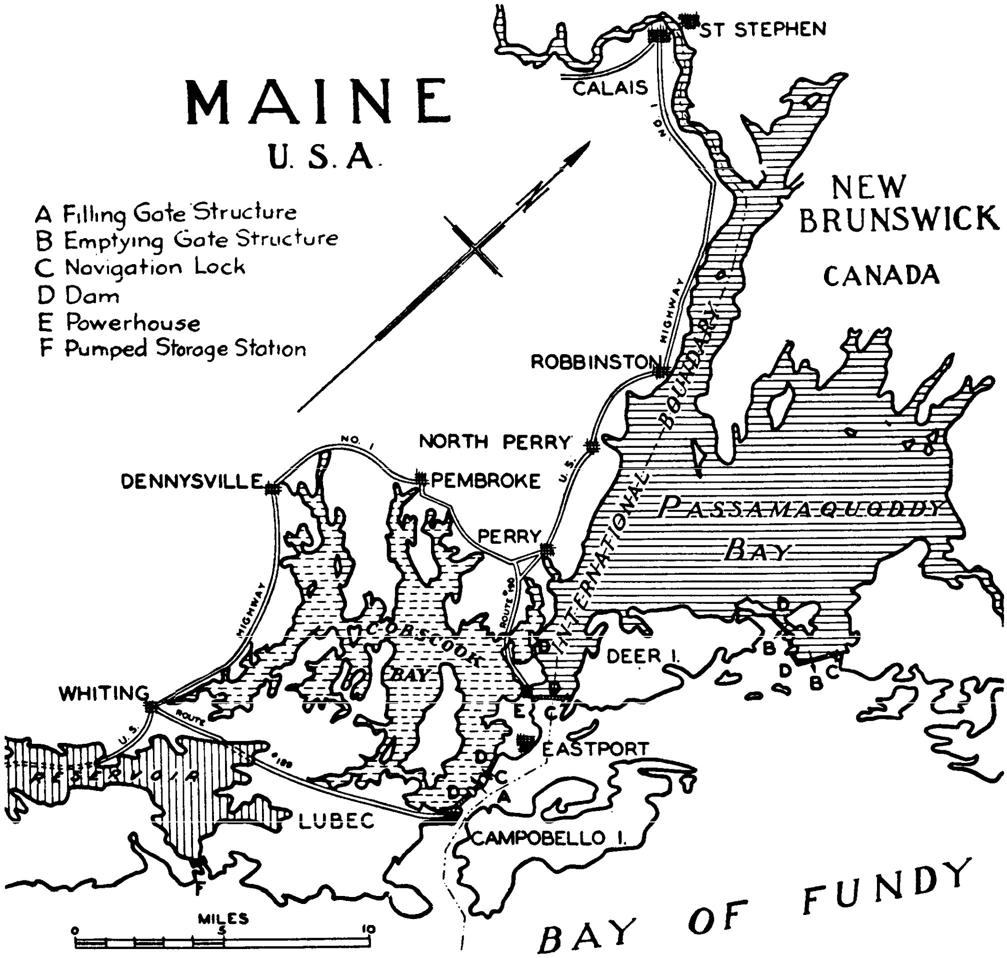

FIG. 1

The project authorized in 1935 was for the construction of only one step of the two-basin plan. It provided for the development only of Cobscook Bay, a 37 sq. mile (at high tide) estuary lying entirely in American waters. (Passamaquoddy Bay lies in Canada and its ultimate inclusion in a two-basin development will require the cooperation of Canada.) The plan of operation was to drain Cobscook Bay to low tide levels and when the outer tide rose some 5 1/2 feet higher than the interior tailwater basin level, to open the power house, generating power on the rising tide to high tide level and on the falling tide until the head was reduced again to near 5 1/2 feet. The power house would then be shut down due to insufficient head. As the tide fell further to the level of the interior basin, the gates would be opened and the "tailwater" in the basin permitted to drain to near low tide levels and the cycle resumed.

Whereas the two-basin development would provide some constant power, the single-basin development would require supplemental priming means to develop any firm power. For of the 12 hr. and 25 min. tidal cycle, power could be produced in varying amounts for periods of approximately 7 1/4 hours only. Cooper proposed a pumped storage reservoir at Haycock Harbor, about 16 miles distant, where surplus electric energy produced at and near high tide would be stored in the form of potential hydraulic energy by pumping sea water up into the artificially created high-head storage reservoir, to be drawn on for regeneration during shut-down periods at the tidal station.

CONCRETE PROBLEM

Under this program, as originally contemplated, providing 30,000 KW firm output and including a 10-unit 150,000 KW power station, with navigation lock, a commensurate gate structure and a large pumped storage development, a total of approximately 800,000 cu. yd. of concrete would have been required. This concrete would have been subject in varying degree to severe treatment from salt water action, intense freezing and extensive alternate wetting and drying. The quantities and conditions were such as to warrant an extensive concrete research program.

MODIFIED PROJECT

Our engineering investigations, including thorough foundation investigations, soils mechanics studies, model tests of turbines, scroll cases and draft tubes, complete hydraulic and power studies, and detailed designs and estimates, indicated the necessity of a material revision upward of the cost estimates of the project's initial promoter. They also disclosed that a better and cheaper project could be attained by (1) development of Cobscook Bay as a high-level rather than low-level basin with reversed operation from that originally planned, and (2) elimination of the pumped storage feature, and substitution therefor, for primary power, of power interchange with existing utilities or the provision of a thermal or hydro-electric auxiliary stand-by plant. With these changes it was also found that an equivalent energy output to that contemplated in the initial development (30,000 KW prime) could be attained by a reduction in size of the main tidal power and gate structure installation. Our total concrete requirements were thereby reduced to approximately 260,000 cu. yd. Inasmuch as the extensive concrete research program had, however, already been initiated on the basis of the larger project, and inasmuch as it was felt that these studies would prove to be of general value to the Corps of Engineers, to the engineering profession and general public, the program was continued until curtailment of activity on the project due to the action of Congress in not authorizing the project and appropriating no additional funds for its continuation. Certain phases of this investigation including maintenance and periodic inspection of 43 different concrete test columns now exposed at Eastport to salt water action at half tide elevation will, however, be continued indefinitely.

The conditions affecting Quoddy concrete are, as previously stated, most severe. Initially, this concrete is subject to salt water action, commonly recognized as injurious to normal concrete; secondly, due to the extreme tidal range existing at Eastport including a mean range slightly in excess of 18 ft. with maximum range approaching 27 ft., a large area of concrete is exposed to alternate wetting and drying twice each lunar day (24 hrs. 50 min.). Water under varying pressure is thus forced into the pores of the concrete at high tides and subsequently exposed at low tide to atmospheric effects; thirdly, due to the latitude at Eastport, concrete is subject to severe freezing temperatures.

BASIC ESSENTIALS

It must be recognized initially that there is no single panacea or cure-all for these conditions. It was not expected that some special cement or admixture or process would be found or developed which of itself would ensure a perfect concrete resistant to all these conditions.

The simple elements of the problem are generally as follows: A dense and impermeable masonry mass must be formed of durable stone and sand aggregates strongly cemented together, with the mass and its constituents unaffected by the physical changes resulting from heat and cold and from wetting and drying or by the chemical attack of salt water.

This dictates first that the *physical* structure of the concrete be sound and free from honeycombing, pores or tiny fissures which would furnish incipient "paths of deterioration" from freezing or incurrence of salt water. This requires: (a) selection of sound, dense, clean and well-graded coarse and fine aggregates; (b) limitation of water content to the minimum required for proper workability, as any water over and above that necessary for the hydration of the cement will leave pores corresponding to the physical space occupied by the water prior to its evaporation; (c) rigid control of mixing and placing, including vibration, to ensure a uniform mix of dense compaction, without segregation, of the concrete; (d) choice of a cement subject to minimum physical change in expansion or contraction in the process of setting. Inasmuch as the rate and extent of expansion and contraction are functions not only of the size of pour but also of the chemical composition and fineness of the cement, which affect its heat of hydration, control must be exercised over these features to prevent temperature cracks and fissures; (e) protection of the setting concrete by proper curing, including controlled cooling and protection from severe cold, to avoid insofar as possible excessive temperature changes within the mass, particularly during the early period when tensile and compressive strengths are still low.

The next major requirement is to ensure that the dense and impermeable masonry mass previously defined is protected from *chemical* disintegration. For this purpose, (a) aggregate must be sound and durable and not subject to disintegration from the chemical attack of sea water, and (b) similarly, but of even more importance, the cement, the critical binding agent in the mass, must be immune insofar as possible from chemical disintegration caused mainly by volumetric deformation in the formation of chemical combinations of certain elements in the cement with others, such as sodium and magnesium sulphate, commonly found in sea water.

To ensure that the concrete will be protected against physical change and destruction from freezing, (a) it must first of all be made dense and impermeable initially as previously stated, and (b) its constituents should have as low and uniform a coefficient of expansion as possible in order to avoid excessive differential temperature stresses.

Above all, however, it must be stressed that rigid and unending control of" the mixing, placing and curing of the concrete on the job are of importance equal to if not even greater than that of the determination and selection of the most suitable ingredients and mixture best adapted to resist severe climatic conditions and salt water attack. Much has been written on the importance of this or that element in cements or concrete mixtures but perhaps not enough on the importance of rigid field control in the concreting operation itself.

It is therefore to be stressed that the following discussion of the experimental program of cement and concrete tests recently conducted on the Quoddy project is offered not in any sense as a final solution to the problem of evolving a concrete resistant to cold weather and salt water but merely as an indication of the effects or tendencies of certain elements or conditions on such concrete. It is offered as a complement of and not a substitute for close field control of mixing, placing and curing.

An extensive and well equipped laboratory was set up at Eastport, as a subdivision of the Engineering Division, for the work under the immediate charge of Charles E. Wuerpel, Associate Engineer, who performed a splendid task in the organization of the laboratory and in the conduct of the experimental program. The laboratory was equipped with every facility for the testing of aggregates, cements and concretes.

AGGREGATES

Since stone or gravel and sand forming as they do the great bulk of any concrete are most important elements of good concrete, it was necessary to determine sources of suitable material obtainable locally if possible, in order to avoid high freight charges on the large tonnages required. Fifty-four sources of stone, gravel and sand were located and investigated. The most favorable sources of excellent appearing aggregates available at moderate haul were subjected to the usual laboratory analysis for grading, freedom from silt, specific gravity, porosity as evidenced by per cent absorption, soundness as evidenced by the magnesium sulphate test, and resistance to freezing and thawing. Sands were also subject to the standard tensile and compressive tests by comparison of mortars made of standard Ottawa Sand with others made of the sands to be tested.

An excellent sand in quantity (125,000 cu. yd.) more than adequate for the requirements of the initial project was found in an esker at Dennysville. The extent and consistency of this formation were determined by deep test pits excavated in the formation. Much of the gravel could also be obtained there, (70,000 cu. yd. incident to procuring the sand), the remainder of the coarse aggregate to be crushed stone from diabase formations at the site of the project.

Prior to finding the required amount of suitable sand, study was made of the possibility of producing a manufactured sand of fine crushed stone. Such stone dust produced a concrete of even greater compressive strength than that with Dennysville sand (which in turn gave better results than standard Ottawa) but required for approximately equal workability about 1/2 bag of cement more per cubic yard of concrete. Because of the lack of workability and tendency to produce a more porous though stronger mortar or concrete, it is not recommended.

As will be discussed later under the cement and concrete studies, the aggregates from these sources were found to be resistant to freezing and sulphate action; and in proper gradation and with suitable cement made an excellent concrete well suited to its conditions of exposure. To ensure absolutely the provision of suitable aggregate throughout the work, it was our plan, if operations had gone ahead, to install a screening and washing plant at Dennysville and a crushing and screening plant at Treat Island and to furnish to all contractors on the project their aggregate requirements. On a very large construction project such as Quoddy, with a number of different contractors, such procedure is considered essential to excellent concrete.

CEMENT

It was not our plan in the limited time available, to develop a new cement but rather to test the many standard cements commercially available. These cements extend over a wide range of physical and chemical characteristics. Some 45 cements were received and analysed, many of them with almost identical characteristics. They may be classed, however, as portland cements, high early strength portland cements, alumina cement, portland-puzzolan, synthetic puzzolan, natural cement, and blended cements.

CEMENT CHARACTERISTICS

The cements were subject to the routine tests for composition, fineness, soundness, consistency, time of set, and tensile and compressive strength (Table 1). There was close relationship between the increase in mortar compressive strength at 28 days with fineness of grinding for a number of portland cements of otherwise comparable characteristics (Fig. 2).

SALT WATER FOR MIXING

Studies were made of the effect of the use of salt water for mixing purposes on the compressive strength of cement mortar. (Fig. 3). Although increasing concentrations of sea water accelerate setting as compared to fresh water, such action is generally attained at the expense of ultimate strength and durability and is therefore to be condemned. Furthermore, concrete formed by salt solutions is less resistant to freezing and thawing action. It seems almost axiomatic therefore that only fresh water should be employed for mixing concrete.

HEAT OF HYDRATION

The heat of hydration is an important characteristic of the various types of cement. The effects of important elements affecting the heat of hydration are well illustrated in Table 2.

TABLE 2—CONTRIBUTION OF EACH PER CENT OF COMPOUND COMPOSITION TO HEAT OF HYDRATION IN CALORIES PER GRAM

Element	Age 2 Days	Age 7 Days	Age 28 Days
C_3S	1.0	1.14	1.25
C^2S	0.0	0.21	0.42
C_3A	1.5	2.44	2.32
C_4AF	0.4	0.20	0.11
Spec. Surf.*	2.0	2.20	2.00

*For each 100 cm^2/gm. over 1200 cm^2/gm. specific surface multiply by the factors shown for contribution to heat of hydration. These factors apply to cements having a heat of hydration of about 100 calories per gram.

TABLE 1—OXIDE ANALYSIS AND COMPOUND COMPOSITION OF TEST CEMENTS (SEE TABLE 1A)

Serial No.	Brand	Compound Composition				Oxide Analysis							Insol. Res.	Free CaO	Sp. Grav. Cement
		C_3S	C_2S	C_3A	C_4AF	SiO_2	CaO	Fe_2O_3	Al_2O_3	MgO	SO_3	Ign. Loss			
PC- 1	C	50	22	5.6	15.2	20.7	63.1	5.0	5.3	2.2	1.8	1.6	0.3	0.8	3.16
PC- 2	B	61	16	1.0	16.7	21.7	65.8	5.5	3.9	0.9	1.5	1.0	0.2	1.0	3.18
PC- 4	A	48	29	6.5	10.3	22.9	64.9	3.4	4.6	1.6	1.8	0.9	0.3	0.7	3.15
PC- 5	B	50	28	7.5	10.4	22.6	65.0	3.4	5.1	1.3	1.6	0.7	0.7	0.4	3.16
PC- 6	B	51	26	7.4	10.4	22.6	65.7	3.4	5.0	1.5	1.7	0.8	0.2	0.5	3.17
PC- 7	C	46	26	5.8	12.2	20.8	62.9	5.0	5.4	2.4	1.8	1.9	0.3	1.1	3.15
PC- 11	B	48	29	6.6	10.4	22.8	64.8	3.4	4.7	1.5	1.6	0.8	0.3	0.4	3.15
PC- 14	A	53	24	6.1	10.8	22.3	65.3	3.6	4.6	1.5	1.7	1.1	0.3	0.8	3.15
PC- 15	B	51	27	6.4	10.4	22.8	65.2	3.4	4.6	1.3	1.6	0.7	0.2	0.3	3.18
PC- 16	A	52	24	8.2	10.4	22.0	65.2	3.4	5.3	2.6	1.3	0.9	0.2	0.5	3.16
PC- 17	C	40	33	11.7	7.6	22.2	64.4	2.5	6.0	2.0	1.7	1.3	0.2	1.3	3.09
PC- 18	D	45	26	10.2	8.4	20.8	62.6	2.8	5.6	3.1	2.4	2.0	0.3	0.9	3.08
PC- 19	D	42	30	11.4	8.4	21.4	63.4	2.8	6.1	3.1	2.1	1.3	0.2	.07	3.11
PC- 20	B	50	24	9.3	8.5	21.3	64.2	2.8	5.3	3.1	2.0	1.2	0.2	0.9	3.09
PC- 21	D	43	26	15.2	7.3	20.3	63.3	2.4	7.3	3.3	1.9	1.2	0.2	0.9	3.11
PC- 22	A	50	20	14.4	7.8	19.9	63.8	2.6	7.1	3.4	1.9	1.6	0.2	0.5	3.09
PC- 23	E	50	19	14.7	7.9	19.8	64.7	2.6	7.2	3.0	1.6	1.0	0.1	1.6	3.10
PC- 24	H		Aluminous			8.7	38.0	14.3	39.2	1.5	0.3	0.0	1.1	0.0	3.19
PC- 25	F		Portland—Puzzolan			31.2	47.4	3.3	6.2	4.0	2.0	5.4	16.2	2.8	2.93
PC- 26	E	55	13	5.8	17.7	19.0	63.6	5.9	3.9	2.7	1.9	1.0	0.2	1.5	3.17
PC- 27	B	66	8	7.2	9.7	20.2	66.1	3.2	4.8	1.4	2.4	1.7	0.3	1.3	3.10
PC- 29	B	57	12	13.8	7.5	19.0	64.4	2.5	6.8	3.4	2.5	1.2	0.2	1.3	3.09
PC- 31	E	73	1	10.0	7.0	19.6	66.6	2.3	5.2	2.6	2.4	1.6	0.2	1.0	3.07
PC- 32	I		Natural			27.1	31.7	2.3	4.6	18.7	2.4	10.4	16.1	0.1	2.89
PC- 33	G		Synthetic Puzzolana			48.1	2.3	11.2	34.9	0.6	0.9	2.4	85.2	0.2	2.54
PC- 35	B	48	29	7.9	10.4	22.5	65.5	3.4	5.2	1.4	2.0	0.7	0.2	0.6	3.15
PC- 37	D	45	28	6.5	11.1	21.6	62.6	3.7	4.8	3.6	1.8	0.7	0.2	1.0	3.16
PC- 44	J	62	10	5.7	12.6	19.9	64.3	4.2	5.8	2.0	2.3	1.5	0.2	1.1	3.08
PC- 45	J	49	27	2.3	15.6	22.2	63.5	5.1	4.2	2.0	1.7	0.9	0.2	0.4	3.08
PC-100	ABC	53	23	4.6	14.1	21.8	64.6	4.6	4.7	1.6	1.7	1.1	0.6	0.8	3.16
PC-101	ADE	50	20	14.8	7.6	21.0	63.8	2.6	6.6	3.3	1.9	1.7	0.2	1.0	3.10
PC-102	ABCI	(PC 32 21%		PC-100 79%)		22.9	57.7	8.5	4.7	5.2	1.8	3.0	3.9	0.6	3.10
PC-103	ABCG	(PC 33 15%		PC-100 85%)		25.7	55.3	5.6	9.1	1.5	1.6	1.3	13.3	0.7	3.06

Table 1a (see Table 1, opposite page)

CEMENT FINENESS AND PARTICLE SIZE DISTRIBUTION — TENSILE AND COMPRESSIVE STRENGTHS—CEMENT MORTAR

Serial No.	Fineness 200 Mesh	Fineness 325 Mesh	Fineness Surf. A. cm²/gm	Particle Size Distribution, Micron Sizes—% Passing 7.5	10	15	20	25	30	35	40	45	50	55	60	Tensil Strength p.s.i. 1 Da	3 Da	7 Da.	28 Da.	90 Da	1 Da	Compressive Strength p.s.i. 3 Da.	7 Da.	28 Da	90 Da	180 Da.
PC- 1	94.2	87.2	1790	24	29	39	47	54	60	67	73	79	82	85	87	128	272	264	449	443	45	500	2200	3915	5630	5570
PC- 2	95.4	91.5	1800	22	28	40	49	59	65	72	78	83	88	90	91	135	343	435	471	453	565	1390	2940	4590	5920	6470
PC- 4	97.6	92.8	1810	24	30	40	49	59	64	73	78	83	88	91	93	140	318	402	425	479	275	2350	2925	4275	5840	6400
PC- 5	98.6	94.2	1750	21	26	38	49	60	67	73	79	85	89	93	95	133	267	371	458	475	120	1235	1950	3525	5310	6225
PC- 6	98.8	94.4	1790	21	27	40	51	62	68	75	81	87	92	93	94	140	258	348	413	461	235	1285	2000	4400	5800	6075
PC- 7	94.8	86.9	1830	24	30	39	47	55	58	67	73	78	83	85	87	158	257	365	438	458	100	2560	2990	4375	5170	5620
PC- 11	98.6	95.7	1810	22	26	38	48	55	62	71	78	85	88	94	96	190	290	341	428	432	430	1320	2675	4325	5740	5835
PC- 14	90.0	82.9	1550	19	24	35	43	49	54	60	65	72	75	79	83	98	183	313	410	466	165	1050	2325	3405	3015	5785
PC- 15	91.0	82.4	1460	14	24	37	46	52	57	63	70	73	79	80	82	100	218	343	389	471	90	720	1390	2655	4305	4735
PC- 16	91.6	83.2	1650	21	27	36	46	50	54	62	66	71	78	81	83	107	232	395	421	441	150	875	1810	3475	4895	5475
PC- 17	94.2	86.9	1820	24	30	37	47	54	62	67	71	76	80	83	87	122	317	358	445	482	200	1260	2575	4860	5930	....
PC- 18	98.4	95.0	2710	51	60	68	76	80	88	91	95	95	95	95	95	185	373	379	484	520	525	3040	4700	6415	6895	6920
PC- 19	86.0	78.0	1670	21	25	39	46	52	55	60	66	70	73	76	78	88	283	310	447	444	140	1325	2525	4275	5515	5935
PC- 20	99.4	95.8	2020	26	33	45	57	65	72	81	87	90	94	96	96	165	328	410	505	585	140	1700	3415	6085	6925	6970
PC- 21	88.0	79.2	1380	16	20	30	38	45	52	57	61	67	71	74	79	102	277	382	433	466	65	1100	2240	3950	4800	4825
PC- 22	94.2	86.8	1670	20	26	36	47	56	63	69	73	79	81	84	87	140	310	420	411	443	175	1275	2450	4210	5400	5435
PC- 23	89.4	79.5	1420	21	28	31	40	47	53	54	64	69	74	76	80	103	298	373	413	426	150	1085	2660	4085	4465	4385
PC- 24	80.0	70.0	1390	19	22	30	36	42	46	51	54	60	63	66	70	437	432	415	415	464	7200	8170	8440	8500	7450	7710
PC- 25	86.2	85.3	2490	40	45	53	58	65	70	74	77	79	82	82	85	120	245	300	404	448	100	650	1860	3160	3985	4765
PC- 26	90.4	82.9	1310	15	21	24	33	42	49	51	63	68	77	81	83	118	282	362	430	449	125	710	1990	3600	4405	4465
PC- 27	99.0	95.7	2270	31	38	51	61	70	77	83	87	91	93	96	96	277	417	453	505	509	1435	2950	4110	5060	6410	6225
PC- 29	98.8	94.9	2030	25	35	46	56	64	72	77	85	89	93	95	95	275	338	398	515	525	690	2625	4825	6440	6650	6810
PC- 31	95.6	92.2	1950	25	32	44	54	62	69	74	80	83	88	90	92	325	408	437	484	488	1150	2565	4045	6160	6590	6625
PC- 32	90.8	86.6	2370	37	44	56	65	72	75	78	82	84	84	87	87	0	65	150	196	250	0	75	175	865	1455	1760
PC- 33	94.1	91.9	1530	14	23	40	53	63	69	76	81	84	88	90	92											
PC- 35	92.6	85.2	1715	22	27	36	45	53	59	65	70	78	80	83	85	150	255	357	384	475	475	1680	2840	4250	5140	5210
PC- 37	93.6	88.6	1980	26	31	41	50	58	64	69	76	80	85	87	89	173	317	368	453	469[1]	767	2000	3450	5467	6160	6800
PC- 44	99.6	98.2	2650	38	46	60	74	82	88	92	95	93	95	98	98	333	410	460	475	475	1460	3933	5525	6485	6750	6725[2]
PC- 45	95.0	88.2	1840	25	30	40	48	56	63	70	76	79	82	87	88	208	310	364	453	484	175	1700	2810	4310	5000	5370[2]
PC-100	95.8	90.5	1890													143	303	385	452	472	175	1425	2370	4485	5890	6150
PC-101	90.5	81.8	1490													115	263	265	419	443	75	1700	2350	4285	5265	5175
PC-102	94.7	89.7	1960													124	265	364	424	447	50	975	1960	3840	4740	5215
PC-103	95.6	90.7	1800													118	266	350	436	489	42	1215	2165	4240	5585	6940

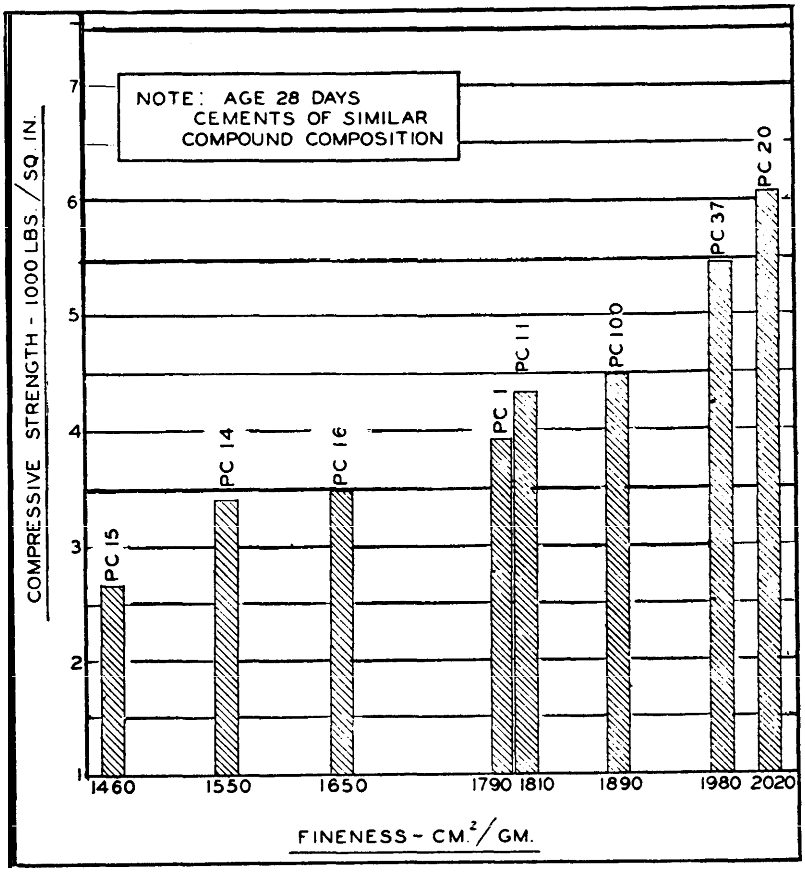

FIG. 2—EFFECT OF CEMENT FINENESS ON COMPRESSIVE STRENGTH OF MORTAR

The great effects on heat of hydration in the important initial period of setting of (a) fineness of grinding, (b) the tricalcium silicates (C^3S) and (c) the tricalcium-aluminates (C^3A) are readily apparent. These indicate the need of a balancing of these factors in the selection of the cement, as well as a close control of heat dissipation in mass and semi-mass concretes if excessive stresses, occuring during a period when the strength of the concrete has not been fully developed, with resultant cracking and incipient failure through disintegration, are to be avoided.

MAGNESIUM SULPHATE

The cement must be resistant to the chemical action of sea water. It is not possible in a short period to test the resistance of cements and concrete to sea water itself although such tests are underway with concrete columns now exposed at Eastport and will be continued). It is possible, however, to determine analogously the relative resistance of cements to sea water attack by the standard accelerated magnesium

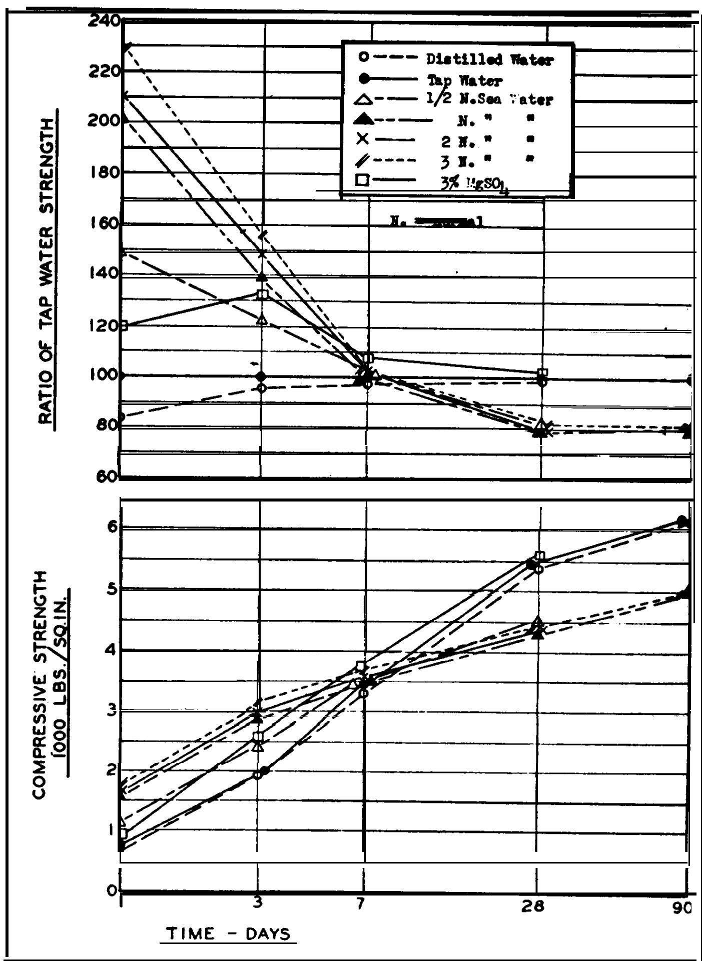

FIG. 3—EFFECT OF MIXING WATER UPON COMPRESSIVE STRENGTH OF CEMENT MORTAR (PC-37)

sulphate test by immersion of neat cement slabs in 10 per cent magnesium sulphate solutions for 280 days. Our tests (Table 3) in general confirmed the findings of other investigators that those cements high in C^3A had slight resistance to the magnesium sulphate attack and hence are not adapted to salt water use. However, the aluminous cement, which attains such high strengths in relatively short time, shows itself absolutely resistant to the magnesium sulphate action.

Our magnesium sulphate tests indicated very definitely the desirability of limiting the C^3A compound in portland cements to less than 8 per cent for proper resistance to salt water action.

BLEEDING

A concrete, including of course its cement binding paste, must be resistant to "bleeding," the expression used to designate the tendency of particles in the paste to settle so that only a film of water remains

TABLE 3—QUALITATIVE COMPARISON OF RESISTANCE OF CEMENTS TO ATTACK BY MAGNESIUM SULPHATE

Cement No.	Compounds				Fineness cm^2/gm	Date Immersed	Age Days	Degree of Attack	Age Days	Degree of Attack
	C_3S	C_2S	C_3A	C_4AF						
PC- 1	50	22	5.6	15.2	1790	10-31-35	104	Slight	280	Slight
PC- 2	61	16	1.0	16.7	1800	11- 8-35	96	Slight	280	Slight
PC- 4	48	29	6.5	10.3	1810	10-25-35	110	Slight	280	Slight
PC- 5	50	28	7.5	10.4	1750	10-25-35	110	Slight	280	Slight
PC- 6	51	26	7.4	10.4	1790	10-29-35	106	Slight	280	Moderate
PC- 7	46	26	5.8	15.2	1830	10-29-35	106	Slight	280	Moderate
PC-11	48	29	6.6	10.4	1810	10-25-35	110	Slight	280	Slight
PC-14	53	24	6.1	10.8	1350	10-25-35	110	Slight	280	Slight
PC-15	51	27	6.4	10.4	1460	10-25-35	110	Slight	280	Slight
PC-16	52	24	8.2	10.4	1650	10-29-35	106	Slight	280	Slight
PC-17	40	33	11.7	7.6	1820	10-31-35	104	Moderate	280	Moderate
PC-18	45	26	10.2	8.4	2710	10-31-35	104	Moderate	280	100% Disint.
PC-19	42	30	11.4	8.4	1670	10-31-35	104	100% Disint.*	280	100% Disint.*
PC-20	50	24	9.3	8.5	2020	11-14-35	90	Slight	280	Slight
PC-21	43	26	15.2	7.3	1380	11- 8-35	96	Serious	280	100% Disint.*
PC-22	50	20	14.4	7.8	1670	10-25-35	110	100% Disint.*	280	100% Disint.*
PC-23	50	19	14.7	7.9	1420	10-25-35	110	Slight	280	Moderate
PC-24		Aluminous			1390	10-25-35	110	None	280	None
PC-25		Portland Puzzolan			2490	10-25-35	110	Slight	280	Slight
PC-26	55	13	5.8	17.7	1310	10-25-35	110	Slight	280	Slight
PC-27	66	8	7.2	9.7	2270	10-29-35	106	Slight	280	Slight
PC-29	57	12	13.8	7.5	2030	10-31-35	104	Moderate	280	100% Disint.*
PC-31	73	1	10.0	7.0	1950	10-25-35	110	Slight	280	Slight
PC-37	45	28	6.5	11.1	1980	4-14-36	118	None	...	....
PC-44	62	10	5.7	12.6	2650	4-14-36	118	Very Slight	...	....
PC-45	49	27	2.3	15.6	1840	4-16-36	116	Very Slight	...	....

*Completely disintegrated.

at the surface to bind the underside of overlying aggregate particles. As this water film evaporates, a tiny void area on the under surface of the aggregate particles remains, resulting not only in a weaker concrete but one permitting the access of water with resultant danger from freezing, etc. (Reference is made to the photomicrograph illustration, Fig. 4, of certain concrete slabs which were divided by the concrete saw employed in the laboratory and then polished.)

Tests for bleeding were made of 18 of the cements with pastes of various water-cement ratios ranging from 0.7 to 1.0 (with the apparatus shown in Fig. 5). Our tests indicated generally that high specific surface area or fineness, particularly in particles finer than the 7.5 micron size tended to reduce the bleeding effect.

Here again, however, it is desired to stress that the use of a cement of reduced bleeding characteristics, resistance to magnesium sulphate attack, etc., will not eliminate in any manner the need for close field control in the matter of low water content, excellent grading of aggregates and mix, rate of placing, and manner of placing and compaction. The cement factor is complementary to but no substitute for close field control.

TEST RESULTS ON TYPICAL CEMENTS

It is of course impossible within the limitations of this paper to list all or even a major part of the test results. The following tabulation summarizes the tests made on some of the cements which may be considered typical of the following classes:

Cement		C_3S	C_3A	Fineness
Portland	A (PC-100) (PC-37)	normal ∓ 50%	med. to low (8 to 1%)	med. high ∓ 1900 $\frac{cm^2}{g}$
	B (PC-101)	normal ∓ 50%	high (10 to 15%)	med. ∓ 1500 $\frac{cm^2}{g}$
High-Early Strength	C (PC-27) (PC-31)	normal to high (45 to 73%)	med. to high 5.7 to 13.8%)	high (1950 to 2710 $\frac{cm^2}{g}$)
Natural	(PC-32)			high 2370 $\frac{cm^2}{g}$
Aluminous	(PC-24)	(not calculable)		med.-low 1390 $\frac{cm^2}{g}$
Portland-Puzzolan	(PC-25) (PC-103)			high 2490 1800 $\frac{cm^2}{g}$
Portland-natural blend	(PC-102)	(79% PC-100 + 21% PC-32)		1960 $\frac{cm^2}{g}$

The tabulation of test data* shows the various cement classes, the chemical analysis and compound composition, sieve analysis, fineness,

*The author's Table 4 will be of considerable interest to some readers. It is in 18 typewritten pages. Copies will be supplied to members of the Institute at the cost of reproduction by whatever process is suited to the evident demand.—EDITOR

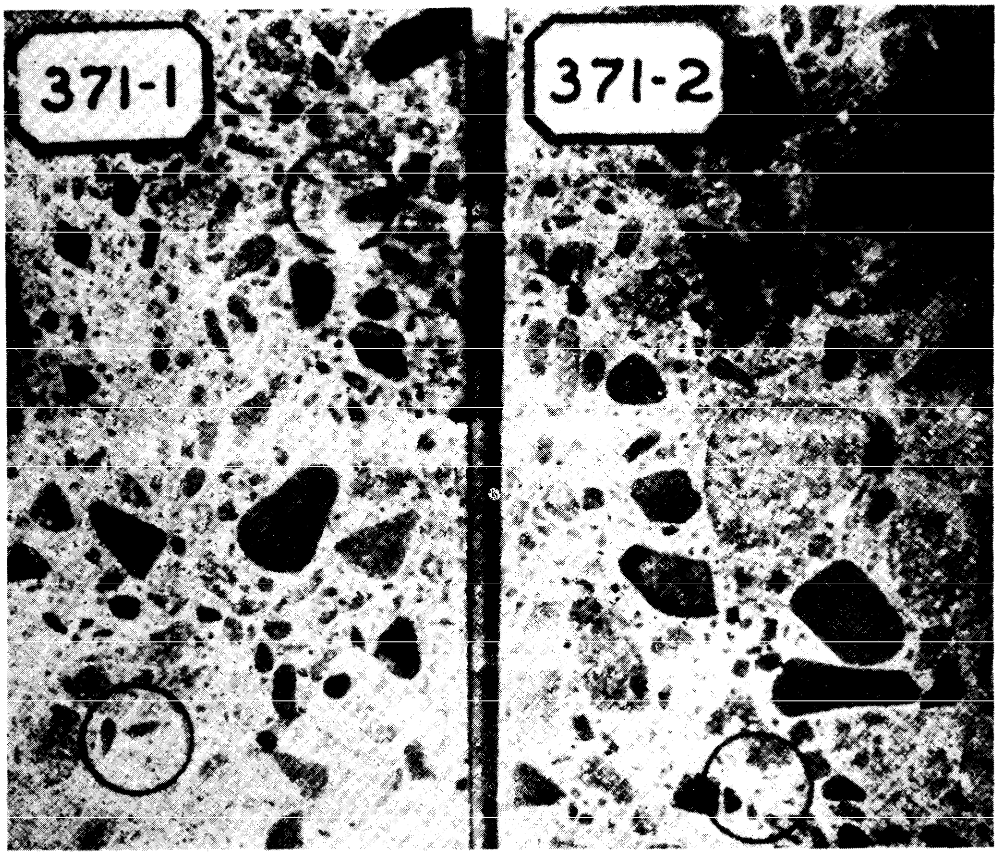

FIG. 4—CONCRETE SLABS MARKED AND READY FOR DRILLING CORES (SEE FIG. 4A)

resistance to magnesium sulphate action, volumetric change of plastic mortar during curing, under alternate wetting and drying and under alternating freezing and thawing, thermal coefficient of expansion, compressive strength under varying curing conditions, tensile strength of standard mortar, and, for concrete made with this cement, volumetric changes under different curing conditions and under alternating freezing and thawing, the coefficient of thermal expansion and compressive strength.

In general, the tabulation of test results shows that the portland cements of the A type (medium high fineness and low C^3A) attain excellent strengths in reasonably short periods and are resistant to magnesium sulphate attack and to material volume change under both alternate wetting and drying and freezing and thawing.

The portland cement Type B (medium fineness and high C^3A), though showing satisfactory strengths, could not withstand the sulphate attack nor the volume change resulting from alternate freezing and thawing conditions.

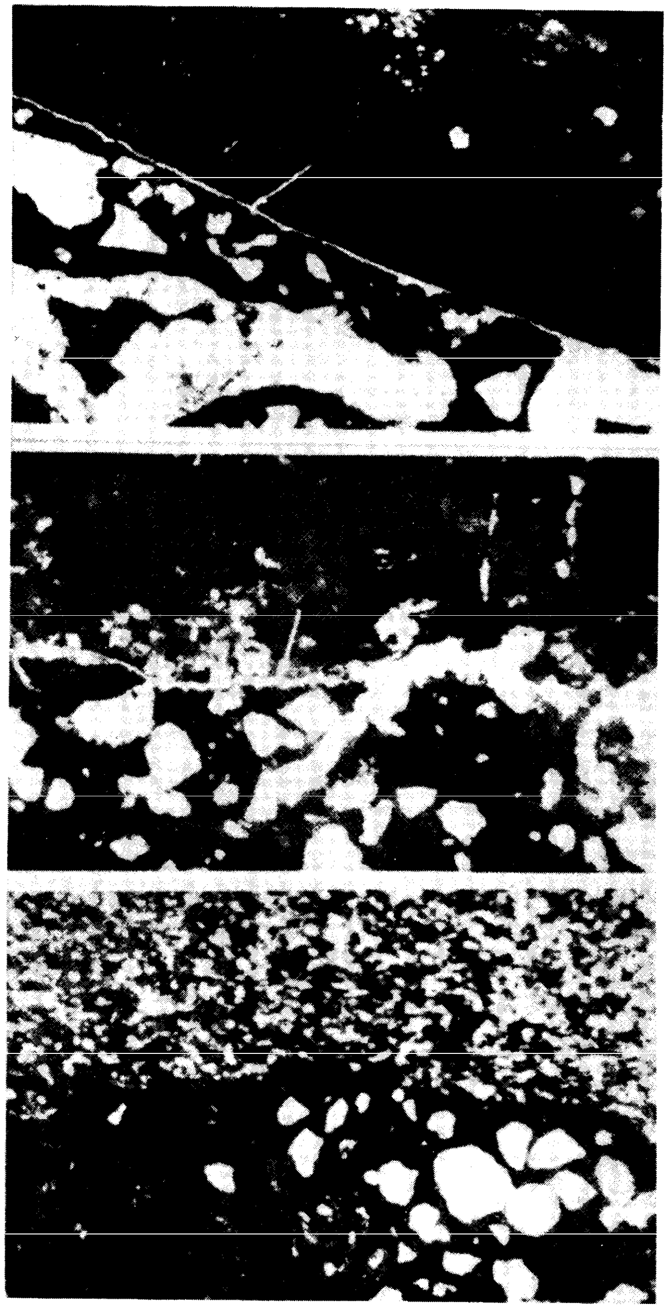

FIG. 4A—PHOTOMICROGRAPHS OF UNDER SIDES OF AGGREGATE PARTICLES SHOWING FILMS OF WATER-GAIN DUE TO BLEEDING OF CEMENTS

Top.—Normal cement; water-cement ratio, 9.90
Center.—Normal cement; water-cement ratio, 0.80
Bottom.—Non-bleeding cement; water-cement ratio, 0.90

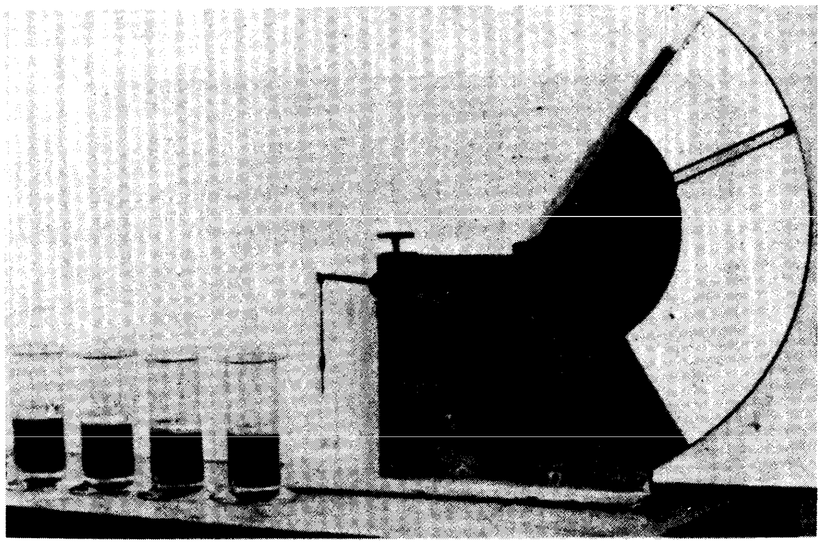

FIG. 5—BLEEDING TEST APPARATUS

The high early strength cements gave, of course, excellent strength characteristics, showed themselves but slightly affected by the magnesium sulphate attack, and resisted well the alternating wetting and drying and freezing and thawing. In mass concrete, however, there would be a problem in the dissipation of the heat of hydration.

The natural cement showed its great weakness in its slow rate of setting and in low strengths attained.

The aluminous cement gave an excellent performance throughout. It showed maximum strengths at very early stages and was absolutely resistant to sulphate attack, as well as giving an excellent performance in resistance to volumetric change from alternate wetting and drying and freezing and thawing. It showed also a low thermal coefficient of expansion. In mass concrete, there would of course be a problem in the dissipation of heat in the setting.

The portland-puzzolan cement showed reasonable strengths, gave high resistance to magnesium sulphate attack, but performed poorly under alternate freezing and thawing.

The blend of portland and natural cements showed moderate strengths, but showed poor resistance to alternating conditions of freezing and thawing.

OPTIMUM MIXTURE

The concrete studies were extended to determine for the various available aggregates the optimum mixture. This determination is most important for any major concrete structure if a dense impermeable and long-lived concrete is to be attained. It should be carried out continuously not only prior to but during the entire concreting operation in order to make such changes as are currently indicated with change in aggregate, etc. Our studies were based on concrete to be placed by internal vibration at a rate of +3800 r.p.m., requiring a slump of about 1 1/2 in. Such determinations do not of course represent the optimum for other methods of placement.

The average grading of fine aggregate employed (closely controlled) was as follows:

Sieve No.	4	8	16	28	48	100
% Passing	100	71.7	50.5	29.2	12.8	4.9

A coarse aggregate also carefully graded with maximum size of 2 in., (because of the extensive amount of closely spaced reinforcement) was also employed.

Tabulations were made of various combinations of sizes of the major coarse aggregates considered showing unit weights and voids, together with the optimum mixtures as obtained from over 150 trial batches (Tables 5 and 6).

To test density obtained—discs (3/4 in. through) were sawed from the top, bottom and center of various 6 x 6 x 12-in. specimens of optimum mixes and tested for absorption, specific gravity and voids (Table 7).

When one considers that 7 1/2 per cent of the voids occur from the excess water required for proper workability, but which fails to enter into combination with the cement, it is seen that air voids in the mix are reduced to approximately one per cent. And only by such close control can a durable concrete be obtained.

VOLUMETRIC CHANGE

Inasmuch as data on the volumetric change of concrete in setting and under extreme ranges of heat ‘transfer including freezing, are of great importance in concrete design, an extensive series of tests relating to volumetric changes was performed on concrete beams of varying aggregates and cements. Corollary thereto and incident to the conversion of one to the other, tests were also run to determine, in addition to the compressive strength, the modulus of elasticity and of rupture, plastic flow, thermal coefficient of expansion, diffusion constant, thermal conductivity and specific heat. Tests were run also to determine the volumetric changes caused by changed moisture conditions.

A description of these tests and discussion of their results should form the basis of a separate paper and are too extensive to be included in the scope of this article. However, some of the principal results which are of general interest are summarized.

TABLE 5—UNIT WEIGHTS AND VOIDS—COARSE AGGREGATE (COMBINED SIZES)

Material	Coa e Aggregate Ratio			Dry Weights, lbs. per cu. ft.						Per cent Voids (by Sp. Gr.)		
	No. 1	No. 2	No. 3	Loose		Rodded		Vibrated		Loose	Rodded	Vibrated
	¼" — ⅜"	⅜"— 1"	1" — 2"	% Sand	Weight	% Sand	Weight	% Sand	Weight	Same Sand Conte . as in Previous Three C umns		
Dennysville Gravel	33.33	66.67	..	....	97.50	....	111.00	....	117.00	41.2	33.1	29.5
"	41.67	58.33	..	....	97.75	....	111.25	....	114.25	41.1	33.0	31.2
"	20	40	40	....	108.75	....	114.00	....	118.25	34.5	31.3	28.8
"	25	35	40	....	109.50	....	116.25	....	119.50	34.0	30.0	28.0
" +Denn. Sand	20	40	40	....	.....	27.1	128.50	27.1	137.50	....	22.4	17.1
" "	25	35	40	....	.....	28.5	131.25	28.5	140.25	....	20.8	15.5
Shackford Diabase	33.33	66.67	..	....	92.50	....	103.75	....	111.00	48.1	41.9	37.8
"	41.67	58.33	..	....	92.00	....	103.50	....	110.50	48.5	42.0	38.1
"	20	40	40	....	95.50	....	109.25	....	116.00	46.5	38.8	35.0
"	25	35	40	....	96.50	....	112.00	....	116.25	46.0	37.2	34.9
" +Denn. Sand	20	40	40	25.2	114.50	25.2	131.50	28.6	138.00	34.6	25.0	21.0
" +Denn. Sand	25	35	40	26.1	113.00	23.4	126.25	27.4	133.75	35.4	28.0	23.4
Black H. Diabase	33.33	66.67	..	....	89.75	....	101.75	....	109.50	50.4	43.8	39.5
"	41.67	58.33	..	....	90.25	....	102.50	....	109.62	50.2	43.4	39.4
"	20	40	40	....	94.75	....	108.00	....	115.75	47.6	40.3	36.0
"	25	35	40	....	95.25	....	109.00	....	114.25	47.4	39.8	36.9
" +Denn. Sand	20	40	40	27.2	117.75	24.3	133.00	27.0	137.25	33.3	25.0	22.2
" +Denn. Sand	25	35	40	25.6	114.50	25.6	127.00	27.3	134.50	35.1	28.2	23.8
Devil's H. Granite	33.33	66.67	..	....	83.25	....	92.73	....	99.25	48.0	42.0	37.9
"	41.67	58.33	..	....	81.50	....	92.50	....	100.00	49.0	42.1	37.4
"	20	40	40	....	88.25	....	98.62	....	104.00	44.8	38.3	34.9
"	25	35	40	....	89.50	....	99.25	....	104.75	44.0	37.9	34.5
" +Denn. Sand	20	40	40	30.4	108.50	26.9	123.00	30.3	127.00	32.8	23.8	20.9
" +Denn. Sand	25	35	40	30.5	104.75	28.2	118.73	31.2	124.25	35.2	26.4	23.1

Dennysville Gravel = Natural Dennysville Gravel
Denn. Sand = Natural Dennysville Sand
Shackford Diabase = Crushed Shackford Head Diabase
Black H. Diabase = Crushed Black Head Diabase
Devil's H. Granite = Crushed Devil's Head Granite.

TABLE 6—OPTIMUM MIXTURES OF CONCRETE

Cement Factor Sx/c.y.	Coarse Aggregate		Max. Sixe	Sand No.	Cement No.	Proportions by Weight	Grad.* of C. A.	W/C gal./sx.	Sand Agg. Ratio	Slump In.	Remolding Effect Seconds to		
	Serial No.										1.0″	0.5″	0″
5.00	G-4	Dennysville Gravel	2″	S.26	PC-37	1-2.30-4.95	A	6.00	32%	1½	7	8	10
5.25	R-3	Shackford	2″	S-26	PC-37	1-2.31-4.84	B	6.00	34	1½	7	12	20
5.25	R-16	Black Head	2″	S-26	PC-37	1-2.31-4.91	B	6.00	34	1	10	18	30
5.25	R-23	Devil's Head	2″	S-26	PC-35B	1-2.31-4.34	B	6.00	34	1¼	9	19	31
6.00	G-4	Dennysville Gravel	1″	S-26	PC-37	1-2.02-3.77	H	5.50	35	3	7	9	11
6.60	R-3	Shackford	1″	S-26	PC-37	1-1.81-3.49	H	5.50	36	1¼	14	18	24

*Grading	Size No. 1 ¼″—⅜″	Size No. 2 ¾″—1″	Size No. 3 1″—2″
A	20%	40%	40%
B	25	35	40
H	35	65	

TABLE 7—ABSORPTION. SPECIFIC GRAVITY AND VOIDS IN CONCRETE*

Coarse Aggregate	Group	App. Spec. Grav.	Abs. Sepc. Grav.	Absorption	Voids
Shackford	V-1-B	2.488	2.702	3.18%	7.92%
"	V-1-C	2.481	2.709	3.38%	8.41%
"	V-2-A	2.439	2.685	3.76%	9.16%
"	V-2-B	2.438	2.681	3.71%	9.06%
"	V-2-C	2.439	2.680	3.68%	9.00%
Dennysville	V-3-A	2.414	2.636	3.495%	8.42%
"	V-3-B	2.380	2.608	3.675%	8.74%
"	V-3-C	2.377	2.613	3.79%	9.03%
"	V-4-A	2.404	2.607	3.24%	7.78%
"	V-4-B	2.385	2.597	3.42%	8.16%
"	V-4-C	2.370	2.586	3.51%	8.35%
Average: Shackford	V-1 & V-2	2.461	2.693	3.500%	8.617%
Average: Dennysville	V-3 & V-4	2.388	2.608	3.520%	8.415%

*Optimum mixture for each aggregate—PC-37 cement.

Modulus of Elasticity

For similar concrete mixes with the portland cement type A (PC-37) and w/c = 0.8, the modulus of elasticity varied at 28 days from 3,650,000 p.s.i. for Devil's Head (granite) coarse aggregate to 5,550,000 p.s.i. with Schackford Head (diabase) aggregate with Dennysville gravel showing 4,800,000 p.s.i. These factors increased with the age of the concrete (about 25 per cent at 180 days).

Plastic Flow

Plastic flow coefficients ranged for otherwise similar optimum concrete mixes w/c = .8, under a constant load of 400 p.s.i. for 130 days from .000185 for Devil's Head granite as aggregate to .000305 for Dennysville gravel. Flow had not, however, ceased at this time when the tests had to be terminated.

Thermal Coefficient of Expansion

For similar mixes these factors ranged per degree Fahrenheit from .000004 for concrete with Schackford Head diabase to .0000045 for concrete with Devil's Head granite, just above that with Dennysville gravel at .00000445. (The coefficient for steel averages about 50 per cent higher and shows how excessive stresses may be set up between the concrete and reinforcement under wide temperature ranges.) With similar coarse aggregates (Dennysville gravel) and different cements the coefficient ranged from .00000445 (with PC-100) to .0000051 (with PC-103.) Furthermore, it was observed that those concretes with high coefficients of expansion generally showed earlier signs of failure under successive cycles of freezing and thawing.

Freezing and Thawing

Consistent with the foregoing, it was observed that the volume change, as measured by the ratio of length expansion to total length, of concrete beams under 200 and more freezing and thawing cycles varied to some degree with the same cement (PC-100) and different coarse aggregates from .00001 in. per in. to .00015 in. per in. With similar aggregate (Dennysville gravel) these coefficients ranged from .00015 with PC-100 cement to a range of .00044 to .00051 in. per in. for other cements (PC-25, 102 and 103) and to failure with one cement (PC-101). (Fig. 6).

Volume Change by Moisture Changes

With the same cement (PC-100) and different coarse aggregates the changes in length under alternate wetting and drying were generally similar whereas slightly greater changes occurred with similar coarse aggregates but different cements.

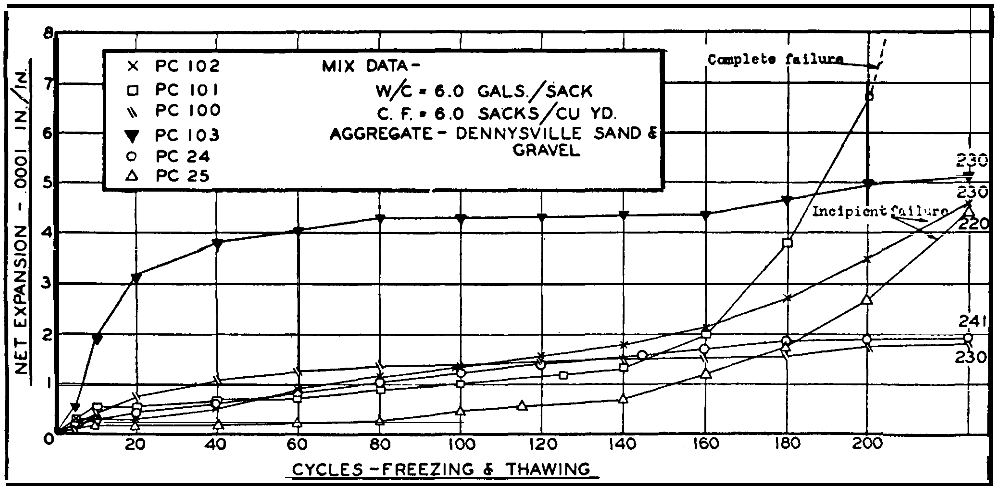

FIG. 6—FREEZING AND THAWING EFFECT OF CEMENT TYPE UPON VOLUME MOVEMENT OF 3 x 3 x 12 IN. CONCRETE BEAMS

For concrete prisms 12 x 12 x 36 in. to 48 in hermetically sealed except at one or both ends, average shrinkage ranged at 140 to 180 days from about .00027 in. per in. at 1/2 in. depth from the exposed end, to about .00008 in. per in. at 60 days and thereafter at 2.5 in. depth, to negligible shrinkage at 12 in. depth or greater. Comparative tests to determine the effect of steel reinforcement on the drying shrinkage of concrete were initiated but could not be brought to completion. However, these tests, extended over 80 days, indicated but little change at equal depths between reinforced and non-reinforced concrete. These tests do exemplify, however, in addition to the differences in shrinkage resulting from the use of various types of aggregates and cements, the great shrinkage which occurs during setting in the outer rim of any concrete pour, particularly if curing is neglected. Excessive tensile stresses are thereby set up during the early setting period, when the concrete is weak, in this critical outer section. In place of being the most important outer armor of protection, it thus becomes the Achilles heel or vulnerable zone for possible future destruction or disintegration of the entire mass through its tiny fissures and cracks, permitting the entry of water, salt, frost and other destructive agents.

Flexural Strenglh

For similar mixes flexural strength ranged as follows for corresponding coarse aggregate: For Devil's Head granite 417 p.s.i., Blackhead diabase 580 p.s.i., Dennysville gravel, 605, and Schackford Head diabase 625 p.s.i.

Thermal Flow

An extensive study of thermal flow in concrete was also undertaken by E. A. Wilder, Junior Engineer, Assistant in the Concrete Laboratory. This included a study of average temperatures to be encountered annually, to determine the number of freezing cycles together with a study of tidal cycles to determine the varying periods to which various elevations of the concrete would be subject to alternate wetting, drying and freezing. Thus with data as to air and water temperatures and convection factors and data on conductivity, specific heat, density, uncombined moisture and coefficient of surface cooling of the concrete, it is possible to determine the areas subject to alternate freezing and thawing.

Mr. Wilder evolved, in addition to other formulae on heat flow, the following formulae on the assumption that no heat is gained by conduction from the interior and that the heat liberated by freezing of uncombined moisture is equal to the net amount of heat lost by conduction in the same time interval:

For freezing:

$$t = \frac{144\, \mu\, \rho}{(28.3 - b)\, ak} \left(\frac{ax^2}{2} + kx\right)$$

For thawing:

$$t = \frac{144\, \mu\, \rho}{(b - 28.3)\, ak} \left(\frac{ax^2}{2} + kx\right)$$

t = time in hours

b = air or water temperature to which exposed (For example, air = 28° F. water = 40° F.)

a = convection factor air = 2 BTU/□c/hr./ F.
water = 260 BTU/□c/hr./ F.

Freezing point of sea water = 28.3° F.

k = conductivity factor concrete (example = 1.07 BTU/ft./hr./°F.)

ρ = density concrete (example 158.9 lb./cu. ft.)

μ = uncombined moisture (example 4%)

x = depth of freezing in feet

For concrete subject to tidal action, the maximum freezing under assumed constant temperature conditions occurs where the concrete frozen in t hrs. is thawed in 12.4—t hrs. (the 12.4 being the hours in a tidal cycle).

Substituting t and 12.4—t in the equations and solving show for the data assumed in this tide cycle a depth of freezing of 2.1 inch occurring at a point exposed 11.23 hrs. and covered 1.17 hrs. Formulae were also evolved to determine the rate and extent of heat flow in concrete.

Thermal Properties

The thermal properties of concrete made with different coarse aggregate were determined (Table 8).

TABLE 8—THERMAL PROPERTIES OF CONCRETE AT 70° F.

Coarse Aggregate	Conductivity "k" BTU/ft./hr./°F.	Specific Heat "C" BTU/lb./°F.	Density "ρ" BTU/lb./°F.	Coefficient of Diffusion "h^2" sq. ft./hr.	Coefficient of Surface Cooling "S"
Dennysville Gravel	1.30	0.23	152.8	0.037	0.148
Devils Head Granite	1.24	0.23	149.4	0.036	0.153
Black Head Diabase Treat Island Diabase	1.06	0.23	158.9	0.029	0.161
Shackford Head Diabase	1.01	0.23	157.5	0.028	0.165

Notes: Specific heat, density, and coefficient of diffusion were determined for Class B concrete (Water-Cement Ratio = 6.0 gal. per sack. Maximum size of coarse aggregate = 2-inches) The conductivity and the coefficient of surface cooling were computed from the other thermal properties. $\left(h^2 = \frac{k}{C\rho}\right)$

CURING

In connection with the actual field placing of the concrete too much stress cannot be placed on the subject of proper curing. Irrespective of the care taken in selection of aggregates, method of mixing and placing, etc., incipient failure and limited life to the concrete may result from faulty curing.

SALT WATER CURING

Due to the expense and possible shortage of fresh water and the availability of sea water at Eastport, study was made of the use of salt water for curing purposes. No damaging effects were observed. On the contrary a comparison of 46 tests based on 28-day cylinders showed an average of 4340 p.s.i. compressive strength for cylinders cured 28 days in salt water at 70° F. as compared to 4169 p.s.i. for concrete cured in the most room 14 days at 70° F. It is most important, however, that the film of salt deposited on horizontal construction joints during curing be thoroughly cleaned off prior to concreting the next lift as otherwise a plane of weakness and future disintegration will occur.

CURING PROCEDURE FOR MASS AND SEMI-MASS CONCRETE

For the prevention or reduction of surface cracking from rapid cooling and drying, the following procedure for mass or semi-mass concrete is warranted. Initially, the cement should not have an excessive heat of hydration incident to too high fineness or specific surface area and high C^3S and C^3A content (against these must be balanced the advantages of early strength gain, a function of these factors, and bleeding, an inverse function of fineness); secondly, the size of pour and of monoliths should be limited; thirdly, ingredients entering the concrete should be kept at a reasonably low temperature (50° F.); fourthly, to avoid excessive tensile stresses at periods when the concrete has not developed strength adequate to withstand them, forms of adequate insulating value should be kept on sufficiently long and in freezing weather such supplemental protection afforded as to assure a gradual and fairly uniform heat reduction; fifthly, maintenance of the concrete at near 50° F. for a period of about 3 days after removal of forms with a gradual reduction therefrom in winter; sixthly, to ensure against cracking from early drying of the surface concrete, the concrete should be kept saturated not only while the forms are in place but also after their removal for a total period of 14 days if possible.

For salt water concrete even greater than normal protection in the amount of concrete' coverage outside of the steel reinforcement is warranted (minimum of 6 in. if possible) to ensure that the steel rein. forcement is not subject to salt water attack which might occur through the incursion of salt water through tiny surface pores or checks.

TYPE OF CEMENT PROPOSED

As a result of the various tests performed at Quoddy and review of literature on the subject, it was decided to employ a portland cement generally as specified under A. S. T. M. designation C77-32, modified as follows:

Chemical composition:

Upper limits	Loss on ignition	3.0 per cent
	Insoluble residue	0.65
	Sulphuric Anhydride (SO^3)	2.00
	Magnesia (MgO)	5.00

Ratio: Iron to alumina $\frac{Fe^2O^3}{Al^2O^3}$ = not more than 1.56
not less than 0.50

Silica (S^1O^2) not less than 21 per cent

Compound composition (R. H. Bogue method) Tricalcium silicate (C^3S) not more than 55 per cent
not less than 40 per cent
Tricalcium aluminate (C^3A) not more than 8 per cent

Fineness: not less than 1800 sq. cm. per gram
not more than 2300 sq. cm. per gram
(Wagner turbidimeter, ASTM, C115-34T)

The cement as above specified (conforming generally to the PC-100 or PC-37 as used in the tests) is considered well suited for the semi-mass concrete exposed to the conditions obtaining on the Quoddy project. It can be developed at moderate cost at every standard portland cement mill in the country. It has no radical innovations or cure-alls. It has a good service record. It makes a concrete resistant to sulphate action, to material volumetric changes occurring from setting or moisture or heat changes, and to alternate freezing and thawing; it attains a reasonably early strength without excessive heat of hydration.

But above all, it is not expected that its use will permit the reduction by one iota of the other equally important ingredients of good durable concrete; namely, closely controlled grading, mixing, placing and curing.

The writer was chief engineer of the engineering division of the project. Charles E. Wuerpel, Associate Engineer, was in immediate charge of the concrete laboratory and tests. Lt. Col. Philip B. Fleming, Corps of Engineers, was District Engineer in local charge of the project. Brig. General George R. Spalding served as Division Engineer, North Atlantic Division, in supervisory charge. Major General E. M. Markham, Chief of Engineers, is the responsible head of this and all other river and harbor and flood control projects under the Army Engineers throughout the country.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by April 1, 1937. For such discussion as may develop readers are referred to the JOURNAL *for May-June, 1957.*