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# ADDENDUM REPORT
## PART C

ON

# WESTERN BEAUFORT REGION
# CONCRETE AGGREGATE STUDY

PREPARED FOR

INDIAN AND NORTHERN AFFAIRS CANADA
OTTAWA, ONTARIO

PA 2291.04

MARCH 1989

KLOHN LEONOFF
CONSULTING ENGINEERS

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OUR FILE: PA 2291.04.01

March 17, 1989

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Western Beaufort Region
Concrete Aggregate Study
Contract Serial No. A7134-8-0015/02-ST
Part C - Elaboration of Original Study

Dear Sirs:

In accordance with the terms of this contract we are pleased to enclose ten copies of our report. This finalizes this phase of our part in the project.

Yours very truly,

KLOHN LEONOFF LTD.

L.E. RODWAY, Ph.D., P.Eng.
Manager, Materials Engineering Division

LER/sh
Enclosure

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KLOHN LEONOFF

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## 1.0 INTRODUCTION

During September 1986 Klohn Leonoff Ltd. conducted a field sampling program of six designated aggregate sites along the western shores of the Mackenzie Delta and the Beaufort Sea, generally between the Aklavik area, N.W.T. and King Point, Yukon Territory. For identification through the study the specific aggregate items are referred to as follows:

1. Shingle Point, Yukon, Sources Y-78 and Y-85 at 68°56′N, 137°12′W.
2. Running River, Yukon, Sources Y-79 and Y-80.
3. King Point, Yukon, Source Y-70 at 60°06′N, 138°04′W.
4. Jacobs Ridge, Yukon, Source Y-74 at 68°59′N, 137°38′W.
5. Moose Creek, Yukon, Source Y-102 at 68°45′N, 136°30′W
6. Willow River, N.W.T., Source 467 at 68°12′N, 135°27′W.

The aggregate samples were each subsequently subjected to a series of laboratory tests for determination of their suitability or otherwise for use in concrete in the Arctic.

Those evaluative procedures consisted of thirteen different but standard basic tests on samples from each of the six sources as well as both standard and non-standard alkali-reactivity tests. The latter non-standard tests involved the immersion of cement-aggregate prisms in a brine solution containing ice. This was an attempt to simulate subsurface conditions for concrete immersed in waters of the Beaufort Sea.

The results of that study were summarized in a rather extensive 175 page report (Klohn Leonoff, 1988a) referred to as Part A, which was submitted in final form after numerous additions during April 1988.

It was deemed necessary as a result of the Part A report to proceed with certain additional and more sophisticated tests on certain of the aggregates that had initially produced ambiguous results. Concurrently, the length change measurements of both the standard and non-standard alkali-reactivity tests were continued over a longer time period. A summary of the results were submitted toward the end of April 1988 (Klohn Leonoff, 1988b). That program is designated Part B of the Western Beaufort Concrete Aggregate study.

Part C of the total study is the document now in hand which is intended to summarize the two previous reports, present the last long term alkali-reactivity length change measurements and most importantly to elaborate and discuss in more detail the results of the more sophisticated tests with a view to presenting our overall conclusions.

## 2.0 SUMMARY OF RESULTS

### 2.1 KLOHN LEONOFF, 1988A

### 2.1.1 Field Program

The sampling program noted that Willow River and Moose Creek were furthest from the sea with attendant access and transportation problems particularly in summer when development operations would occur and yet transportation to the Beaufort would have to be on winter roads. Overburden and location were other problems at these sites. As it happens the subsequent laboratory program found these two sources to be deficient in many durability attributes as well.

The conclusion, based on the foregoing comments was that further exploration for possible future development of aggregate sources be concentrated upon the remaining four sites amongst those sites which we sampled, at Shingle Point, Running River, King Point and Jacobs Ridge.

Of these Running River, King Point and Jacobs Ridge were considered most suitable for development from the point of view of terrain and the necessary available space for stockpiling of overburden as well as both unprocessed and processed aggregate in addition to that space required for a concrete batch plant, a precast concrete plant, plus a camp and equipment maintenance facilities.

The Shingle Point source was considered a special case since the area sampled, although having the advantage of being adjacent to the Sea, was located on the escarpment opposite the Shingle Point DEW Station. This proximity could lead to security restrictions as to whether the source could be used at all or if so whether the existing wharf facilities could be used.

### 2.1.2 Laboratory Program

A total of thirteen procedures including gradings, all considered to be short term basic evaluative tests, were conducted on aggregate samples from all six sources.

Additionally, longer term tests aimed at evaluation of aggregate stability due to alkaline effects were begun.

The results were summarized in eighteen pages (from pages 20 to 37 inclusive) of the Part A report. A further synopsis overview and relative ranking of all six aggregate sources based on all test results at that time were presented in the two following tables, again included here for ease of reference.

In terms of the so-called short term basic tests it was concluded firstly that aggregate from both Willow Creek and Moose Creek would require washing and secondly that chert was noted in the petrographic evaluation. Finally, the high colour values also raised a flag in terms of suitability of the aggregate for use in concrete. These latter two points - the presence of chert and the high organic test colors- required by specification that further investigation was required into the implications in terms of concrete durability of those items.

TABLE 1
SUMMARY OF TEST RESULTS

| Description of Test | Test I.D. | SOURCE IDENTIFICATION | | | | | |
|---|---|---|---|---|---|---|---|
| | | 1 Shingle Point | 2 Running River | 3 King Point | 4 Jacobs Ridge | 5 Moose Creek | 6 Willow River |
| Soundness | A | P | P | P | P-B | P | P-W |
| L.A. Abrasion | B | P | P | P | P-B | P | P-W |
| Petrographic | C | F | F | F | F | P-B | F-W |
| Density | D1 | P | P | P-B | P-B | P | P-W |
| Absorption/Coarse | D2 | P | P | P | P | P-B | P-W |
| Absorption/Fine | D3 | P | P | P-B | P-B | P | P-W |
| Durability Absorption | E1 | P | P | P | P-B | P | P-W |
| Durability Index | E2 | P | P | P | P-B | P | P-W |
| Organic | E3 | P | P | P | F-W | P | F-W |
| Cleanness Coarse | F1 | P | P | P | P-B | F-W | F-W |
| Cleanness Fine | F2 | P | P | P | F | F-W | F |
| 3 mos. Expansion Fine | G1 | P | P | P | P-B | F-W | P |
| 6 mos. Expansion Fine | G2 | P | P-B | P | P | P-W | P |
| 3 mos. Expansion Coarse | H | P | F | P-B | F-W | P | P |
| Expansion Brine<br>3 mos.<br>6 mos. | I | F-W<br>0.03<br>0.04 | P-B<br>0.007<br>0.015 | F<br>0.025<br>0.030 | P<br>0.019<br>0.024 | F<br>0.027<br>0.036 | P<br>0.019<br>0.026 |

NOTE: P denotes pass
P-B denotes pass - best performance
P-W denotes pass - worst performance

F denotes fail
F-W denotes fail - worst performance

TABLE 2
COMBINED FIELD AND LABORATORY RELATIVE EVALUATION

| Component | Shingle Point | Running River | King Point | Jacobs Ridge | Moose Creek | Willow River |
|---|---|---|---|---|---|---|
| A. Field Evaluation | | | | | | |
| 1. Access | VI | III | V | IV | II | I |
| 2. Site Conditions | I | III | IV | VI | II | V |
| 3. Deposit Characteristics | V | II | IV | VI | I | III |
| B. Laboratory Evaluation | | | | | | |
| 1. Soundness | III | II | IV | VI | V | I |
| 2. L.A. Abrasion | III | II | IV | VI | V | I |
| 3. Petrographic | IV | V | II | III | VI | I |
| 4. Density | III | IV | V | VI | II | I |
| 5. Absorption/Coarse | V | IV | III | II | VI | I |
| 6. Absorption/Fine | IV | III | V | VI | II | I |
| 7. Durability Absorption Ratio | IV | V | II | VI | III | I |
| 8. Durability Index | III | V | IV | VI | II | I |
| 9. Organic | II | III | IV | V | VI | I |
| 10. Cleanness Coarse | V | IV | III | VI | II | I |
| 11. Cleanness Fine | VI | IV | V | III | I | II |
| 12. 3 mos. Expansion Fine | II | IV | V | VI | I | III |
| 13. 3 mos. Expansion Coarse | V | I | VI | II | IV | III |
| 14. Expansion Brine | I | VI | III | V | II | IV |

NOTE: I indicates relatively poorest of the six
VI indicates relatively best of the six

Results of the longer term alkali-activity test measurements were presented after exposure to the respective standard and non-standard curing regimes for just over one year.

Generally it was noted in this standard procedure that the fine aggregates were considered to have satisfactorily met the test criteria in terms of expansion. On the other hand the coarse aggregates from

Shingle Point, Running River and Jacobs Ridge were all close to the allowable expansion limit at three months followed thereafter by relative stability. Further investigation of this was considered advisable.

The non-standard alkali-activity tests were conducted with the coarse aggregate exposed to a curing regime of brine approximating Beaufort Sea salinity which was maintained near the freezing point. It was found that the expected expansive process was not slowed by the low temperature relative to the standard test but rather had accelerated after three month exposure by an order of magnitude.

Further length change readings over a longer period were recommended.

## 2.2 KLOHN LEONOFF, 1988B

As a result of some of the anomalies and flags raised with the initial Part A report, certain additional investigative efforts were expanded on some of the aggregate samples.

Specifically, aggregate from two of the sites were subjected to further alkali-activity testing in the form of exposure to the curing requirements and test methods of ASTM C289 - Potential Reactivity of Aggregates (Chemical Method). The result of these procedures was that each of the two aggregates was considered innocuous in terms of potential alkali-reactivity. This despite the fact that the two aggregates chosen for test were for the one case from samples obtained at Running River which produced the largest length change in the standard alkali-activity test and in the other from Shingle Point being the aggregate which produced the longest expansion after exposure to the non-standard 0°C brine.

Those aggregates which had produced very high indices in the organic impurities test, which by definition is designed to detect the presence of organics were retested after organic materials had been neutralized. The result of that procedure was that up to the age of 28-days at least, the inherent organic material had no adverse effect on compressive strength as measured by means of mortar cubes.

In terms of the longer term tests, results of the standard alkali-reactivity measurements were presented after fourteen months exposure.

All the fine aggregate samples from the six sources when tested according to the procedures of ASTM C227 expanded much less than the allowable maximum limit.

The coarse aggregate samples, again from all six sources, when tested according to the procedures of CSA A23.2-14A also expanded much less than the maximum allowable at one year, even after fourteen months.

The non-standard alkali-reactivity tests with specimens immersed in seawater brine at 0°C had stabilized after six to eight months. Equipment failure after that time frame forced termination of further length change measurements.

Testing was continued by others using specialized equipment not available to usual commercial laboratories. More exactly, Dr. J.E. Gillott, P.Geol., a professor on the Civil Engineering Faculty of the University of Calgary, but operating in this case under M and S Research and Consulting Ltd., examined some of the aggregates utilizing equipment owned by the University of Calgary.

Using special equipment the procedures of ASTM C666 - Resistance of Concrete to Rapid Freezing and Thawing, were conducted on three concrete prisms which had been immersed in the 0°C brine previously referred to in the non-standard tests, for eight months plus four months in standard concrete moist room at 23°C and 100% relative humidity.

All three concrete specimens quickly deteriorated and Dr. Gillott's opinion was that concrete of that sort had poor frost resistance.

Secondly Dr. Gillott conducted detailed thin-section petrographic analysis of the three concrete prisms as just referred to, plus one fine aggregate mortar bar. Finally three individual chert particles were examined by scanning electron microscope.

His conclusion as a result of these activities was that "if used as aggregate in portland cement concrete there is every likelihood that durability problems will develop due to deleterious alkali-aggregate reactions".

### 2.3 THIS REPORT: KLOHN LEONOFF, 1989C

This concluding part of at least this phase of the three part study began in September 1986, presents as new data, further results of the long term standard alkali-reactivity tests and updated results of those fine aggregates which had produced high calorimetric values.

Possibly of equal or greater importance is further detailed discussion of data to date leading to our assessment of the aggregates and our conclusion as to their suitability for use in concrete under Arctic conditions.

## 3.0 ADDITIONAL TEST DATA

### 3.1 ORGANIC IMPURITIES TEST RESULTS

One portion of Part B, the first addendum report of this study (Klohn Leonoff, 1988b), dealt with the data available up to the date of that report, April 19, 1988 of the three fine aggregate sources which had initially produced high values when processed according to the procedures of ASTM C40 - Organic Impurities in Fine Aggregates for Concrete which is identical to CSA A23.2-7A Organic Impurities in Sands for Concrete.

Those three sands from Jacobs Ridge, King Point and Willow River were processed according to the requirements of ASTM C87 - Effect of Organic Impurities on Fine Aggregate on Strength of Mortar. The procedure and rationale for same were discussed in detail in both Parts A and B and hence will not be repeated here. Compressive cube strength results up to 28-days were presented in Part B and are included below for ease of reference along with the additional 91-day compressive strengths.

TABLE 1
SUMMARY OF RESULTS OF ASTM C87

| Test No. | Sample Description | Condition | Compressive Strength, MPa (Average of 3 cubes in Each case) | | |
|---|---|---|---|---|---|
| | | | 7-day | 28-day | 91-day |
| 1 | Jacob's Ridge | Untreated | 23.3 | 33.3 | 35.4 |
| | | Treated | 21.0 | 30.7 | 31.3 |
| 2 | King Point | Untreated | 21.4 | 30.9 | 33.5 |
| | | Treated | 14.5 | 21.2 | 23.9 |
| 3 | Willow River | Untreated | 24.9 | 31.3 | 31.7 |
| | | Treated | 21.7 | 29.6 | 32.8 |

## 3.2 LONG TERM STANDARD ALKALI-REACTIVITY TEST RESULTS

These test procedures were begun and reported upon in Part A of the study, continued as an update in the Addendum Part B and finally we include herewith as Part C, the results up to the age of two years shown of course as 24 months on the accompanying graphs.

To reiterate earlier statements, the sands or fine aggregate were tested by mortar bar according to the procedures of ASTM C227 - Potential Alkali Reactivity of Cement - Aggregate Combinations (Mortar-Bar Method).

On the other hand the gravels or coarse aggregates were tested by concrete prisms according to the procedures of CSA A23.2-14A Alkali-Aggregate Reaction.

The selection of these two test methods was because the former, the ASTM method, is for direct application to fine aggregate proposed for use in concrete while the latter, the CSA method, is the only standard test available on coarse aggregate as is, proposed for use in concrete.

The data updated to 24 months follows as Figures 1 to 6 inclusive.

SAMPLE No. 1 - SHINGLE POINT

EXPANSION CURVES for MORTAR BARS and CONCRETE PRISMS

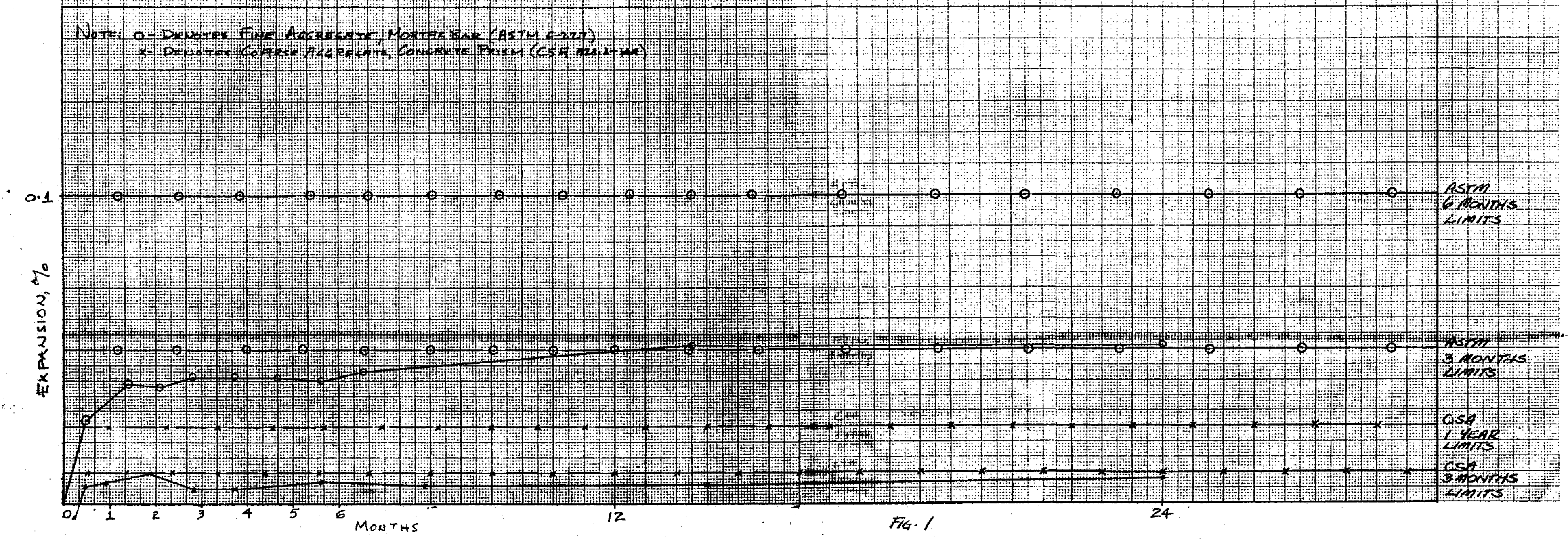


FIG. 1

SAMPLE NO. 2 - RUNNING RIVER
EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

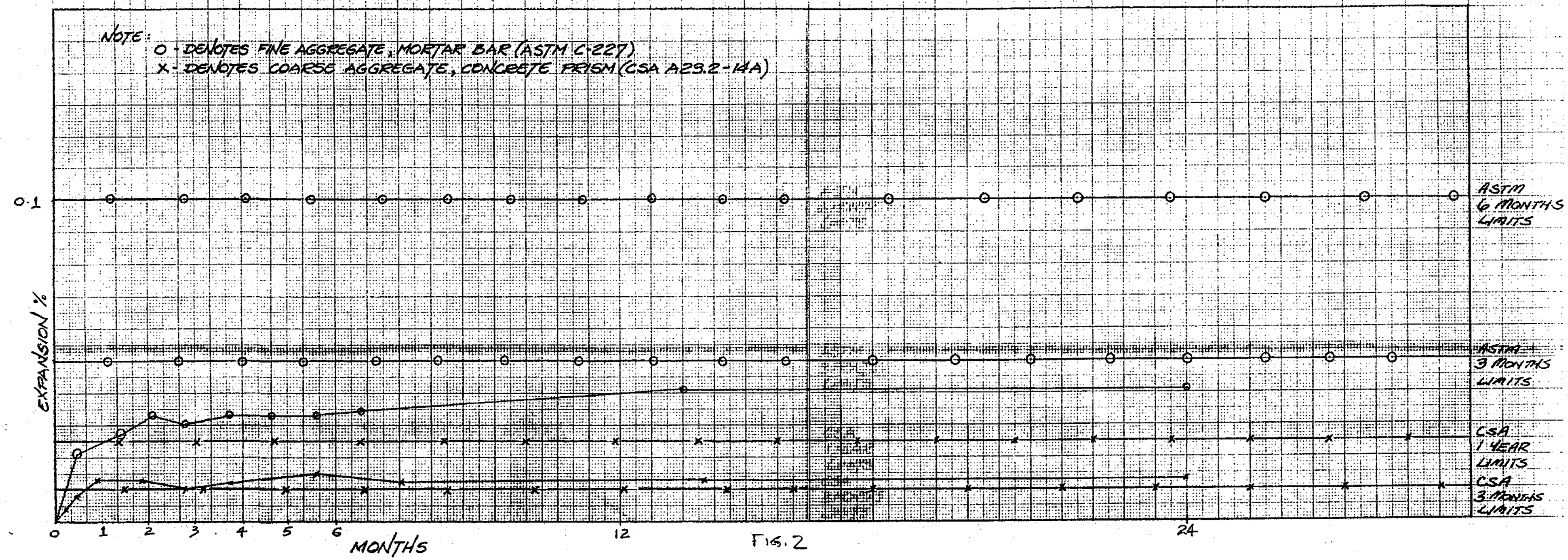


FIG. 2

SAMPLE NO. 3 - KING POINT

EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

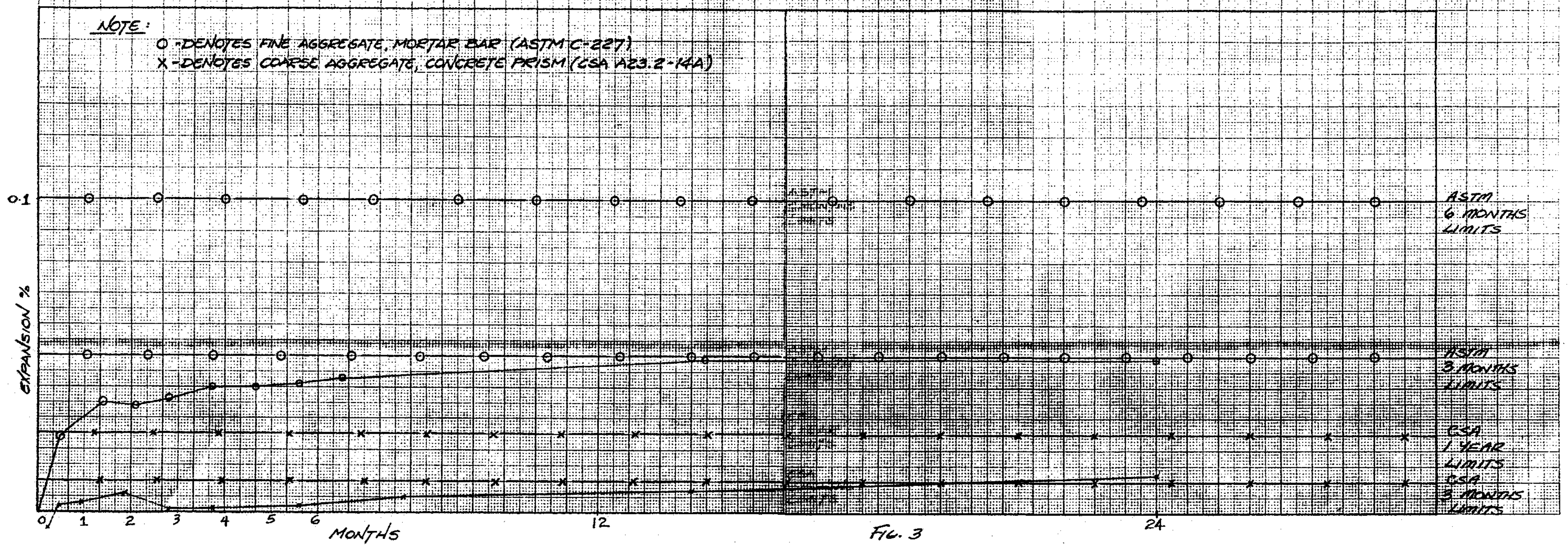


FIG. 3

SAMPLE NO. 4 - JACOBS RIDGE

EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

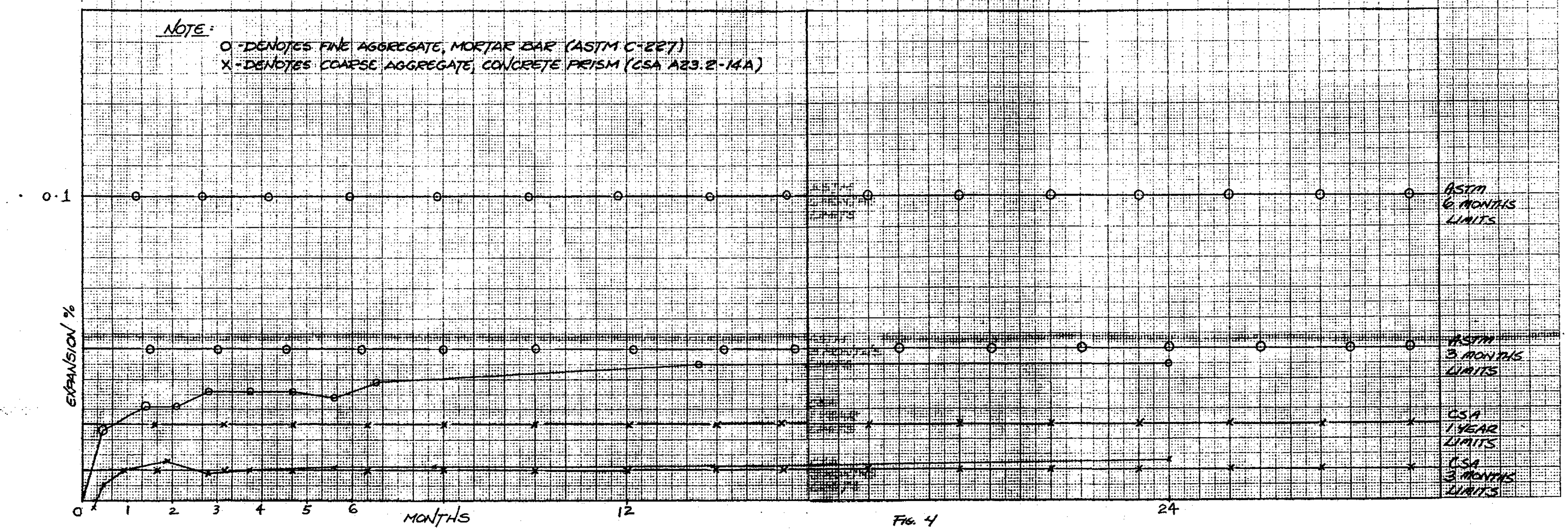


FIG. 4

SAMPLE NO. 5 COAL MINE LAKE – Moose Creek
EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

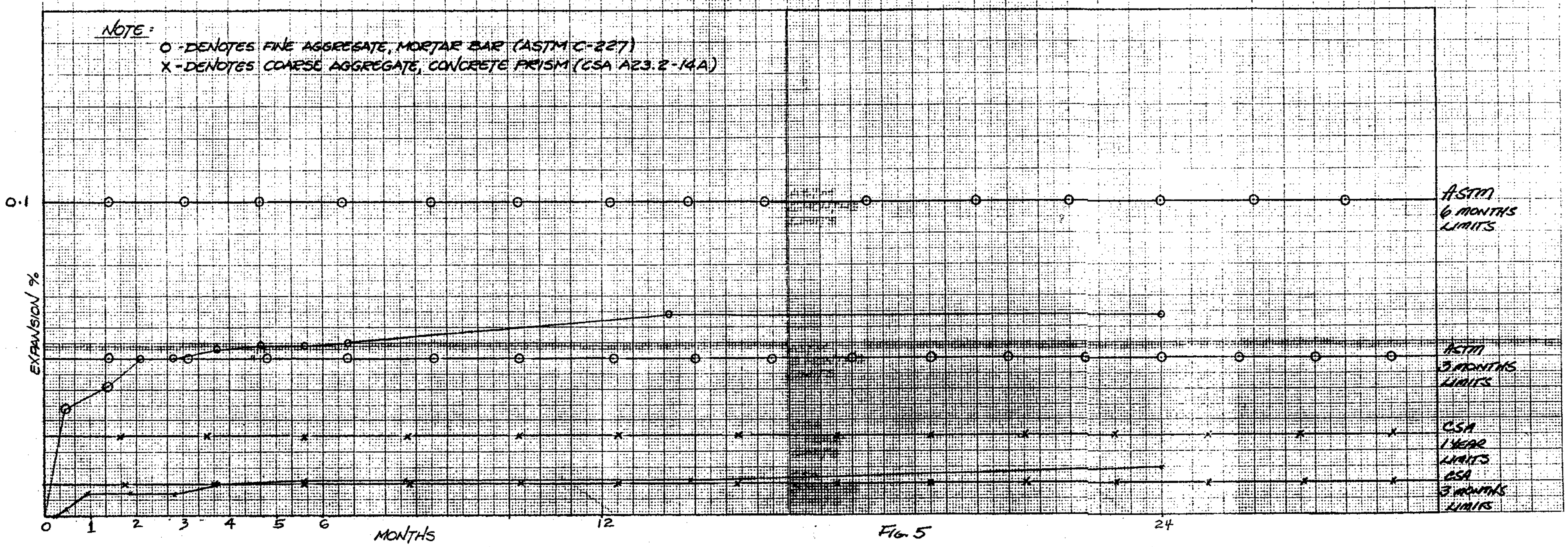


FIG. 5

SAMPLE NO. 6 - WILLOW RIVER
EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

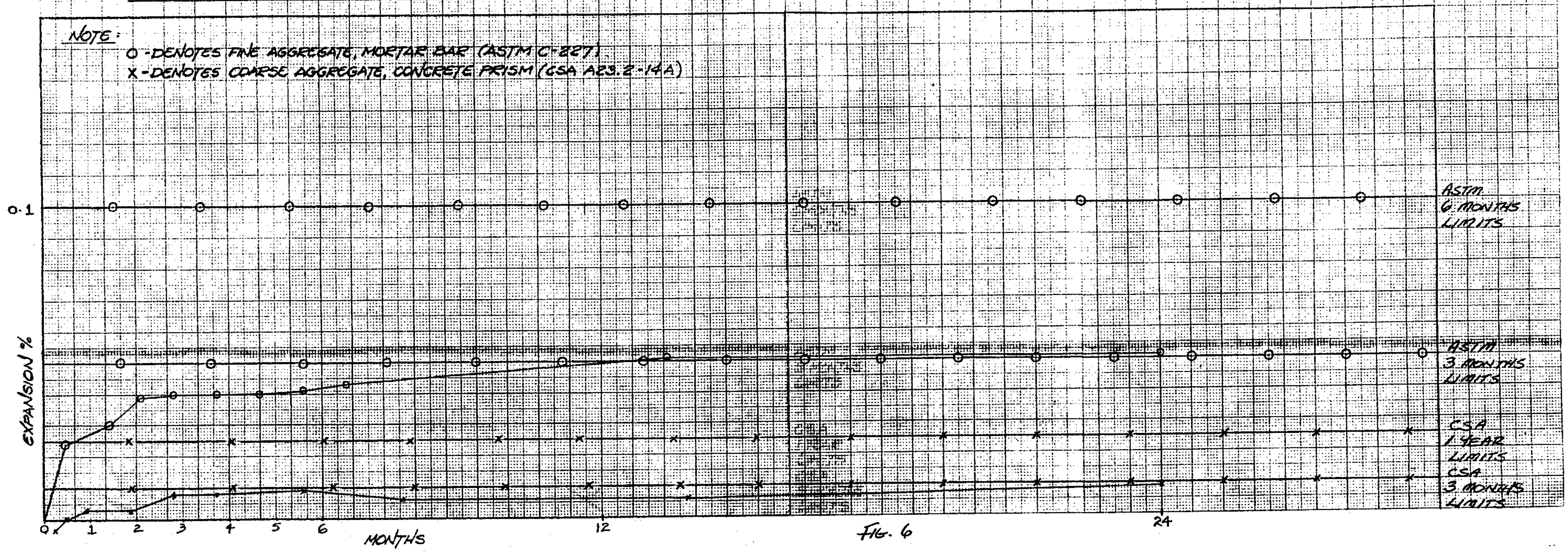


FIG. 6

## 3.3 SHORT TERM ARCTIC MARINE TEST

This program was inadvertantly terminated due to mechanical failure of a key thermostat, unnoticed, unfortunately, for several months. The result is there is no new data to report in this regard in this Part C of the study. Nevertheless we again include the data which has already been included, partly for ease of reference and partly to comment at the appropriate point in this report, in more detail on the results which are available.

The data itself follows as figures 7 to 12 inclusive while our further comments are included as section 4.3.

## SAMPLE NO. 1 – SHINGLE POINT

EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING

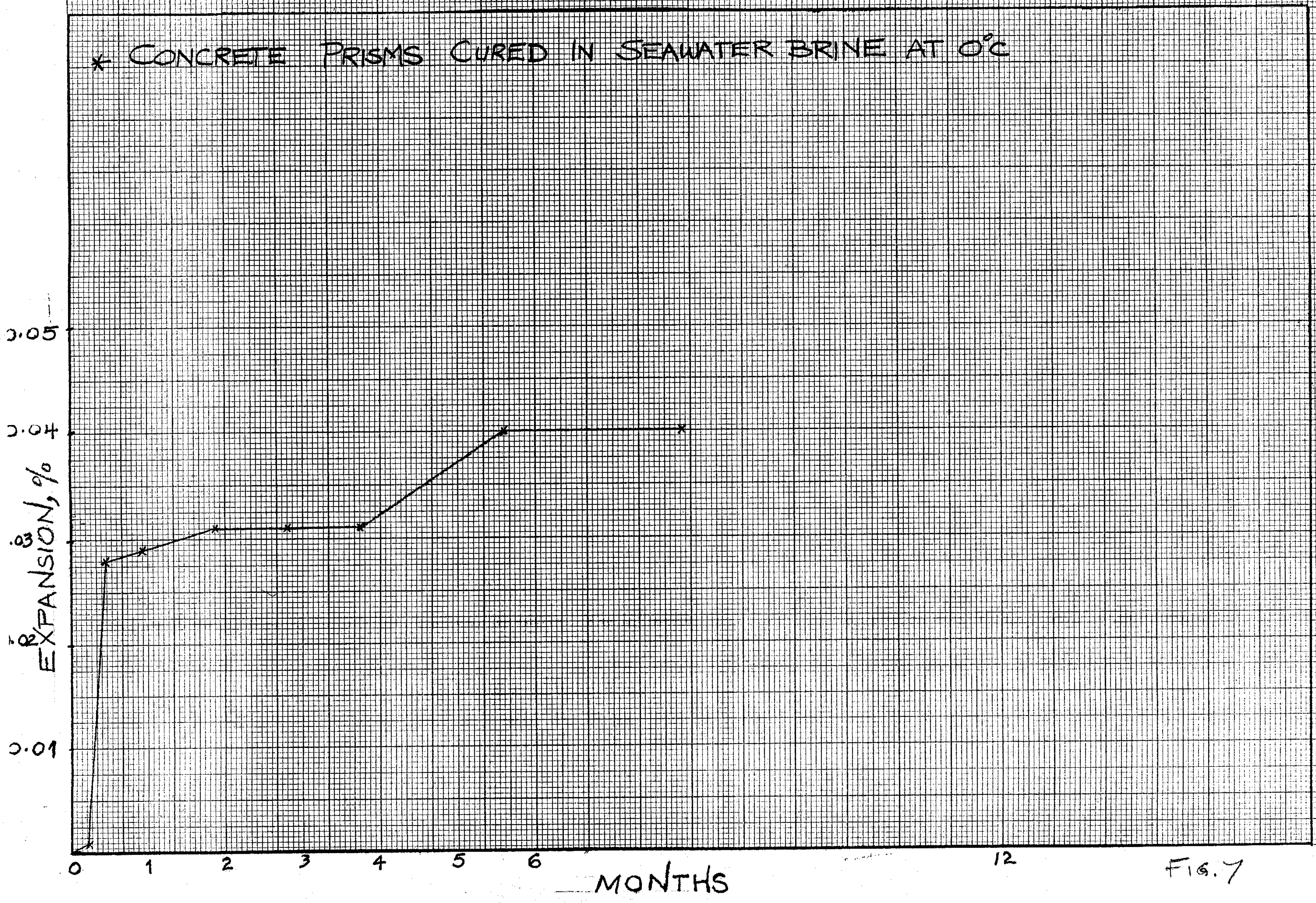
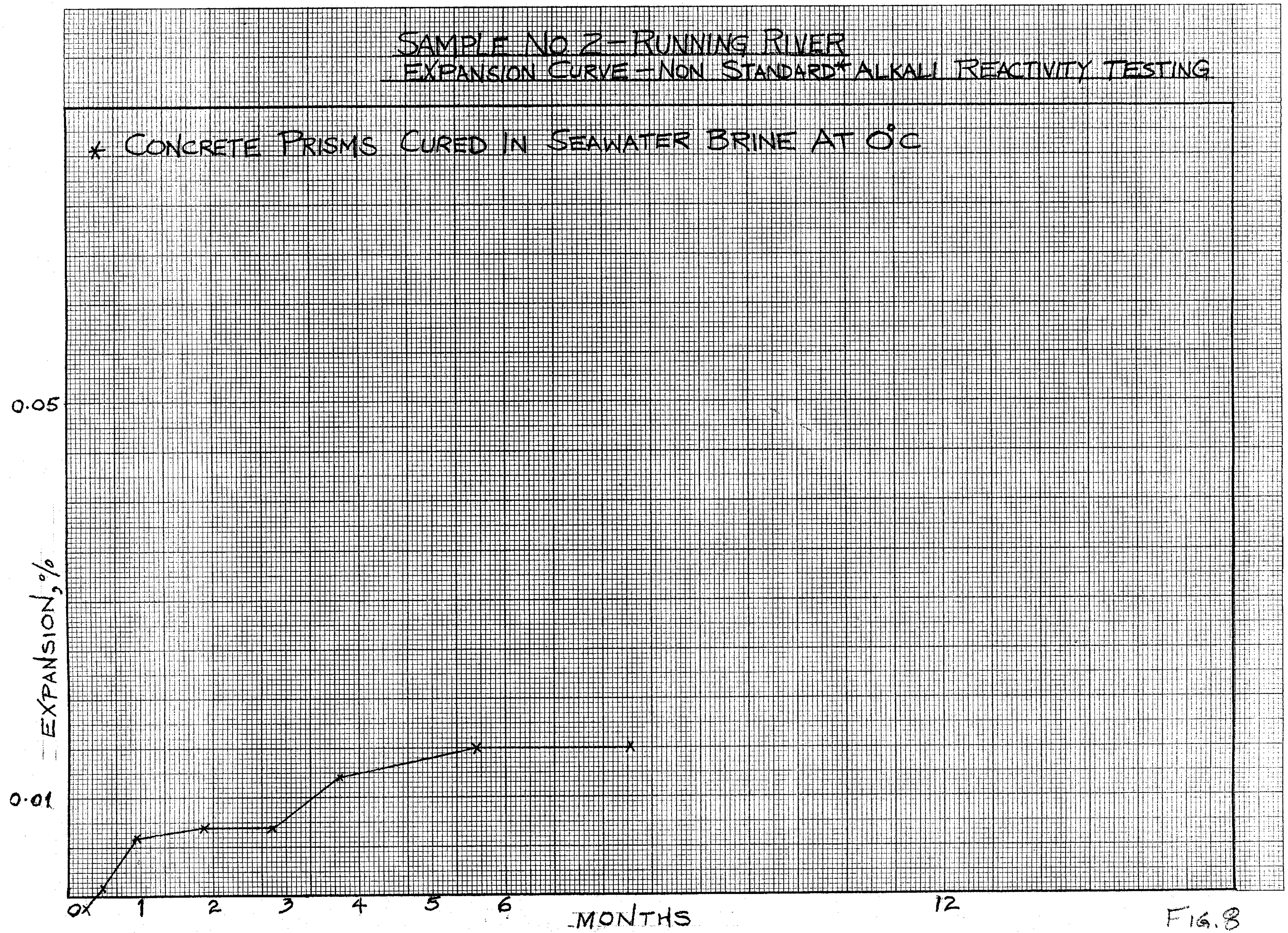


FIG. 7



SAMPLE NO 2 - RUNNING RIVER

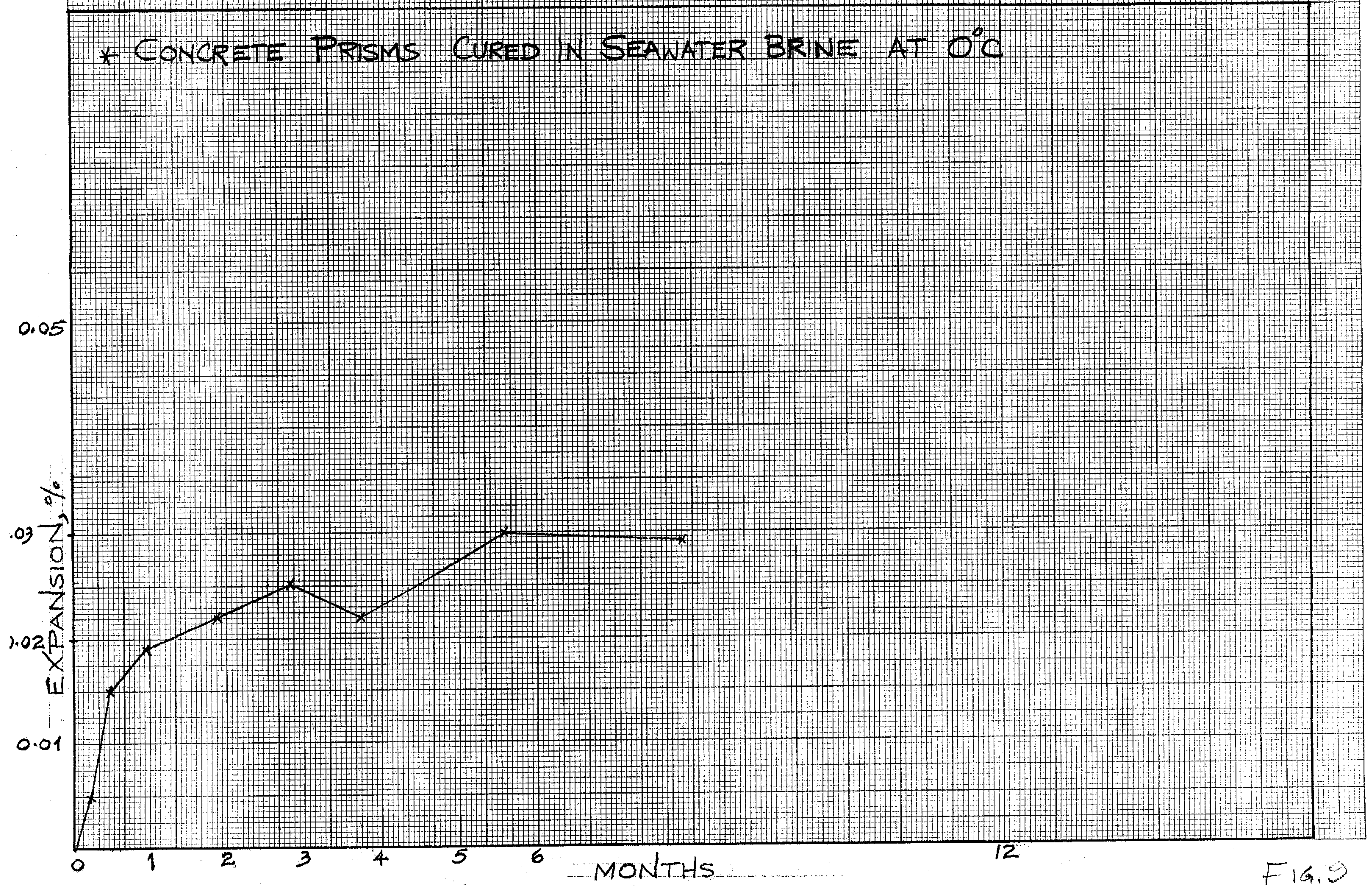
EXPANSION CURVE - NON STANDARD* ALKALI REACTIVITY TESTING

* CONCRETE PRISMS CURED IN SEAWATER BRINE AT 0°C

FIG. 8

## SAMPLE NO. 3 - KING POINT

## EXPANSION CURVE - NON STANDARD* ALKALI REACTIVITY TESTING

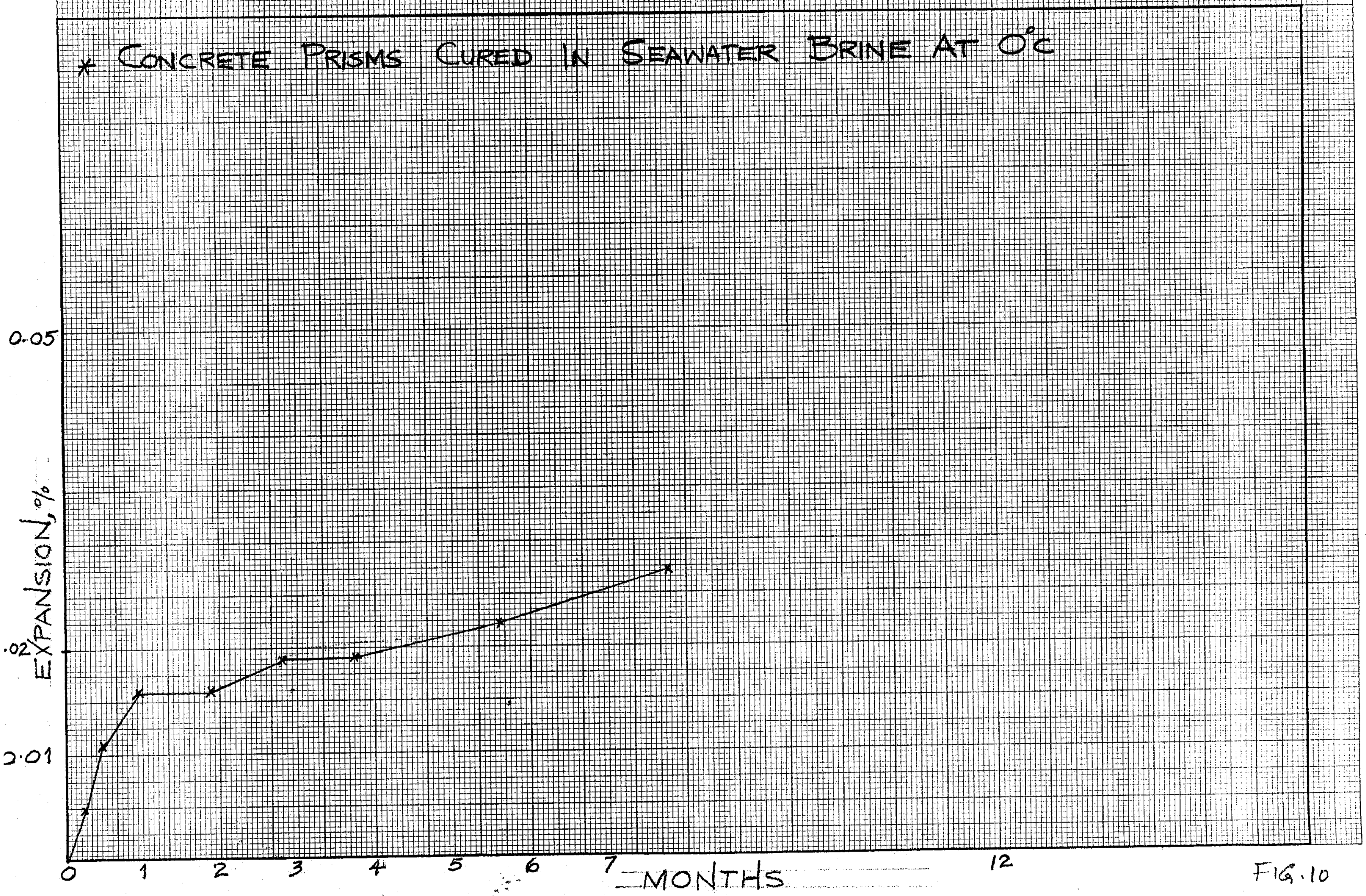


* CONCRETE PRISMS CURED IN SEAWATER BRINE AT 0°C

FIG. 9

## SAMPLE NO 4 - JACOBS RIDGE

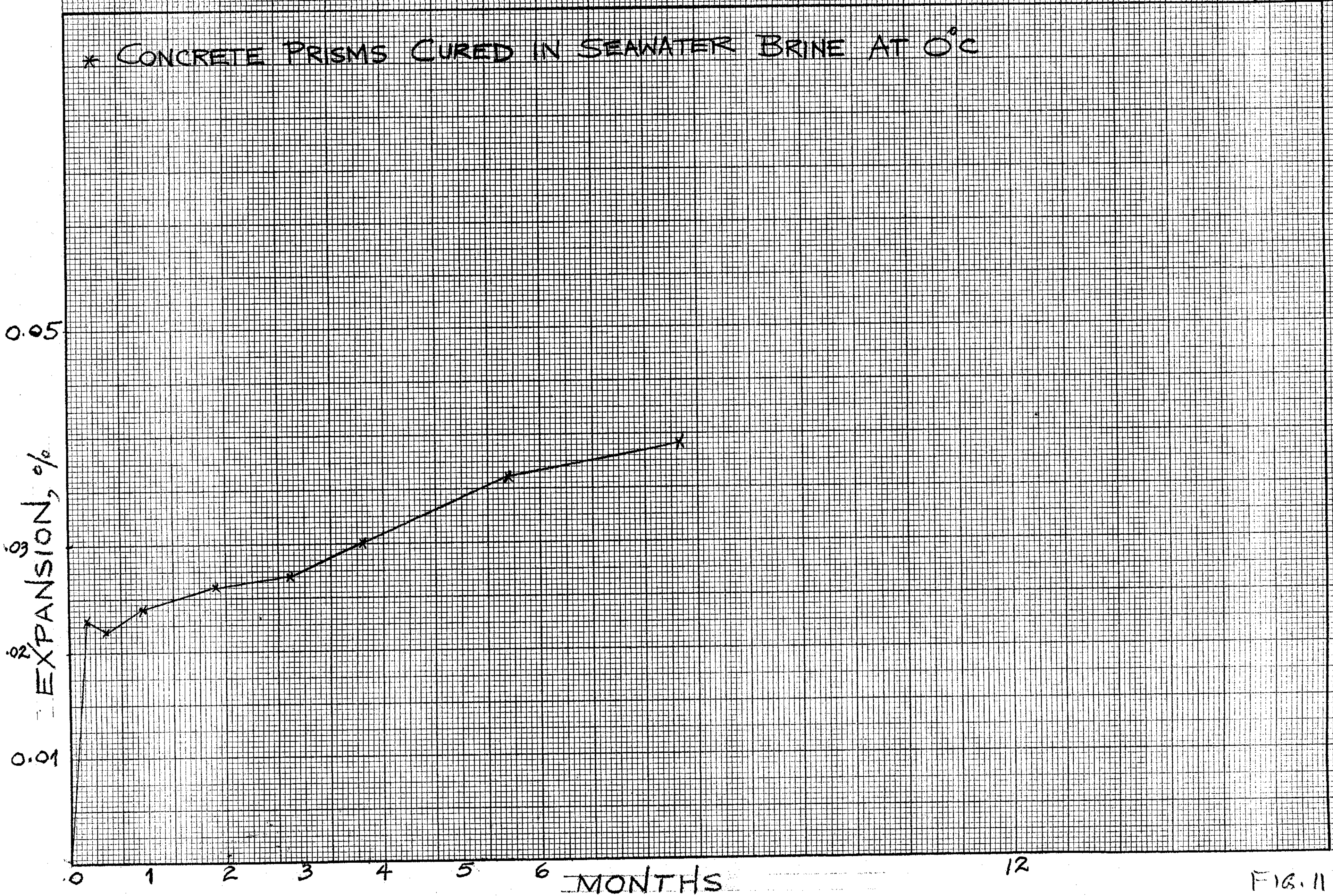
## EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING



* CONCRETE PRISMS CURED IN SEAWATER BRINE AT 0°C

FIG. 10

## SAMPLE NO. 5 – COAL MINE LAKE – MOOSE CREEK
## EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING

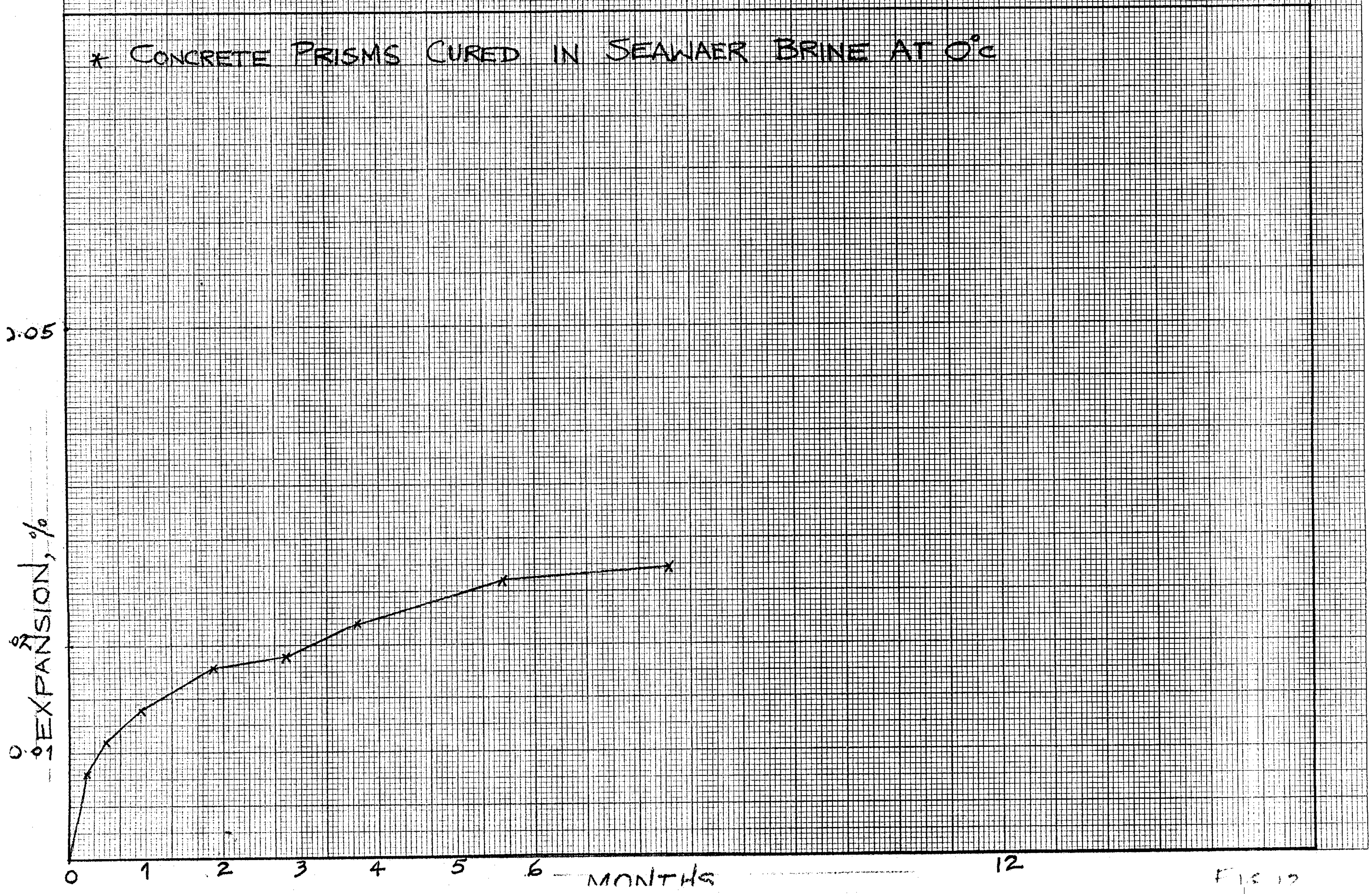


* CONCRETE PRISMS CURED IN SEAWATER BRINE AT 0°C

FIG. 11

# SAMPLE NO. 6 – WILLOW RIVER

## EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING



* CONCRETE PRISMS CURED IN SEAWAER BRINE AT 0°C

FIG 12

## 3.4 REPORT BY J.E. GILLOTT ON RAPID FREEZE-THAW TESTS

## 3.5 REPORT BY J.E. GILLOTT ON THIN SECTION PETROGRAPHY

## 3.6 REPORT BY J.E. GILLOTT ON SCANNING ELECTRON MICROSCOPY

These three items are addressed in Dr. Gillott's report which is attached as Appendix I in entirety and is to be considered as part of this report.

## 4.0 DISCUSSION

### 4.1 ORGANIC IMPURITIES

The previously unreported 91-day comparative compressive cube strengths continued to show in two out of three high colour sands, no adverse effect of the organic materials indicated by that colour. On that basis the sands from Jacobs Ridge and King Point are considered to be satisfactory in this regard for use in concrete.

The exception is the untreated or "as-is" sand from Willow River which at 91-days produced a lesser strength than the same sand with the organics neutralized. It is hypothesized from theoretical considerations that this gap could widen with the passage of time. In fact concrete produced from this sand could be expected to actually drop in strength rather than very slowly increase being the normal case.

This situation in the case of the Willow River sand is interesting in view of our comments in section 9.6 on page 42 of our original report (Klohn Leonoff, 1988a) being Part A of the study. That is to say a previous study (Hardy, 1976) of this Willow River sand, produced "as is" calorimetric values of 5+ being in the same range as Klohn Leonoff's 5. However with coal removed the Hardy value dropped to 2+ which would then be considered acceptable for use in first class concrete. This is opposed to the Klohn Leonoff value in this study, when coal and/or light weight particles were removed of 5 - virtually the same as prior to removal. In our opinion the 91-day compressive strength values presented here, are still more positive confirmation that harmful organics are indeed present in the sand sample from Willow River which we tested.

These conflicting results are further evidence that exploitation of the Willow River source should only proceed with extreme caution, particularly in view of our other adverse test data from this source.

## 4.2 LONG TERM STANDARD ALKALI-REACTIVITY TEST RESULTS

The graphs of Figures 1 through 6, being the standard tests we are discussing have been extended for a full two years which brings the readings up to date. Again, to be precise the concrete prisms which form the basis of the CSA test method A23.2-14A for evolution of coarse aggregate, were cast between November 14 and 20, 1986 for the six gravel samples. Similarly the mortar bars which form the basis of ASTM C227 for evaluation of fine aggregate, were cast between November 24 and December 3, 1986 for the six sand samples.

The two year period has therefore just been completed a couple of months ago, at the time of this writing.

From the readings of length change plotted on these graphs, which is the format recommended by the new CSA supplement, we conclude all the aggregates, both fine and coarse, have generally stabilized after rapid expansion during the first six months. In fact all the aggregates, fine and coarse, have expanded less than the allowable at six months even though twenty four months have now elapsed.

With the exception of the sand and gravel from Moose Creek as well as the gravel from Jacobs Ridge, all the samples have lengthened less than the allowable at three months, even after 24 months exposure.

In interpretation of these results one should refer to the scope of both the ASTM and CSA standards which were used. In the case of the sand we quote from ASTM C227:

> "1.2 Alkalies participating in the expansive reactions usually are derived from the cement; under some circumstances they may be derived from other constituents of the concrete or from external sources. Two types of alkali reactivity of aggregates are recognized: (1) an alkali-silica reaction involving certain siliceous rocks, minerals, and natural or artificial glasses and (2) an alkali-carbonate reaction involving dolomite in certain calcitic dolomites and dolomitic limestones. The method is not recommended as a means to detect the latter reaction involving dolomite in certain calcitic dolomites and dolomitic limestones. The method is not recommended as a means to detect the latter reaction because expansions produced in the mortar-bar test by the alkali-carbonate reaction (see Method C586) are generally much less than those produced by the alkali-silica reaction for combinations having equally harmful effects in service."

In the case of the coarse aggregate tested in accordance with CSA A23.2-14A the restriction against using the test for alkali-carbonate reaction does not apply as noted by clause 1.2:

> "1.2 The test is suitable for the evaluation of coarse aggregate suspected to produce alkali-silica or alkali-carbonate reactions in concrete."

The petrographic analysis of the gravels was conducted in accordance with the requirements of CSA A23.2 Appendix B - Petrographic Analysis of Coarse Aggregate. The analysis was actually conducted by Dr. J.A. Leach, P.Geol. of Klohn Leonoff Ltd. with more than fifteen years experience in that type of analysis. The various constituents comprising the total coarse aggregate samples were presented in Appendix 10 of our original Part A report.

Specifically these were itemized as varying percentages of quartzite chert, sandstone, arkose and traces of siltstone. All these are siliceous rocks with the possible exception of siltstone.

The sands, from the same deposits, would presumably be derived from the parent gravels and hence we feel are also to a very large extent to be of siliceous origin.

That being the case, both the test methods for alkali aggregate reactivity are considered entirely appropriate for estimation of potential expansion problems in concrete due to both sand and gravel. These tests indicate the aggregates are satisfactory in terms of not likely producing excessive expansion when used in concrete under normal conditions. That at least is the basis on which the test was developed and on the limits which have been established for use with the test indicators.

## 4.3 SHORT TERM ARCTIC MARINE TEST

The results of these tests, at least up to the eight month period when they were terminated, are interesting, particularly relative to the standard alkali-reactivity tests after the same period of exposure. That is to say the concrete prisms for both the standard CSA test and this non-standard test, were both cast using the same aggregates and cement and measured by the same concrete technician using the same measuring equipment. Therefore the possibility of variation in results due to materials, human error, or equipment variables was at least minimized even if not eliminated. Despite this and despite the lower temperatures in effect in the non-standard test - which was expected to slow any adverse chemical action which might results in expansion - the result in fact was more, not less, expansion.

Reviewing this data in detail for each aggregate source after eight months of the two curing regimes, results in the following numerical data:

| SOURCE | Eight Month Curing | |
|---|---|---|
| | CSA 23°C Water Expansion % | Non-Standard 0°C Brine Expansion % |
| Shingle Point | 0.006 | 0.040 |
| Running River | *0.012 | *0.015 |
| King Point | 0.005 | 0.029 |
| Jacobs Ridge | 0.011 | 0.027 |
| Moose Creek | 0.011 | *0.039 |
| Willow River | 0.006 | 0.027 |

* Subjected to thin section petrographic examination.

The standard results by interpolating between the 3 months and 12 month limits would all indicate the aggregates are satisfactory after eight months of testing in terms of not exceeding the allowable expansion due to alkali-reactivity.

It is also important to understand that as these aggregates have been established as silicious then any alkali-silica reaction which is to occur will occur early and will usually occur at a high rate of expansion. In fact it is these characteristics that were used in setting the allowable limits to expansion.

However, the non-standard test does not come with a pre-packaged set of limits for use in interpreting the results. It is noted that in every case the expansion was greater at eight months under the non-standard curing regime by a factor ranging between a low of 1.5 in the case of the Running River aggregates and a high of 6.7 in the case of the Shingle Point aggregates, than the expansion under standard conditions. However, the expansions under the non-standard test appear to have stabilized in four of the aggregates, with the Jacobs Ridge and Moose Creek aggregates being unexplained exceptions.

The petrographic thin sections do not show a growth of rims on the aggregate - this will be discussed later in section 4.5 - which would be the result of chemical interaction between alkalies in the cement and the aggregate.

If the much greater expansion in the non-standard test is not due to unstable aggregate (alkali-reactive) then we preclude it is due to the action of the salt in the brine on the concrete itself.

It is well documented that chlorides have an adverse effect on concrete. In fact, a rate of chloride ion penetration is a function of the permeability of the concrete and varying degrees of concrete distress are readily visible on most concrete structures exposed to salt action on bridges, sidewalks, pavements, etc. This topic of the detailed effects of chlorides on concrete is extremely complex and really requires an extensive report in itself to adequately deal with the subject. This is considered to be beyond the scope of this particular report and so not appropriate to expand at length at this time.

However, we do conclude that it is not the aggregate under test that is the cause of the relatively higher expansions which were noted in the non-standard brine exposure test.

In any event the ultimate purpose of these laboratory tests is the attempt to predict whether or not concrete structures built with these aggregates and, in this case, exposed to the sea water at about 0°C would be durable. Now the non-standard test results did not produce total expansions, expressed as a percentage of more than 0.04%. This value in structural terms is referred to as strain. On the other hand the strain which concrete can tolerate prior to failure is of course a function of concrete strength but in general, for quality concrete is in the order of 0.3%. This is an order of magnitude greater than the maximum strain recorded in these non-standard concrete prism tests. It is therefore our opinion based on this reasoning that quality concrete cast using these aggregates would not fail as a result of exposure for eight months to sea water at 0°C.

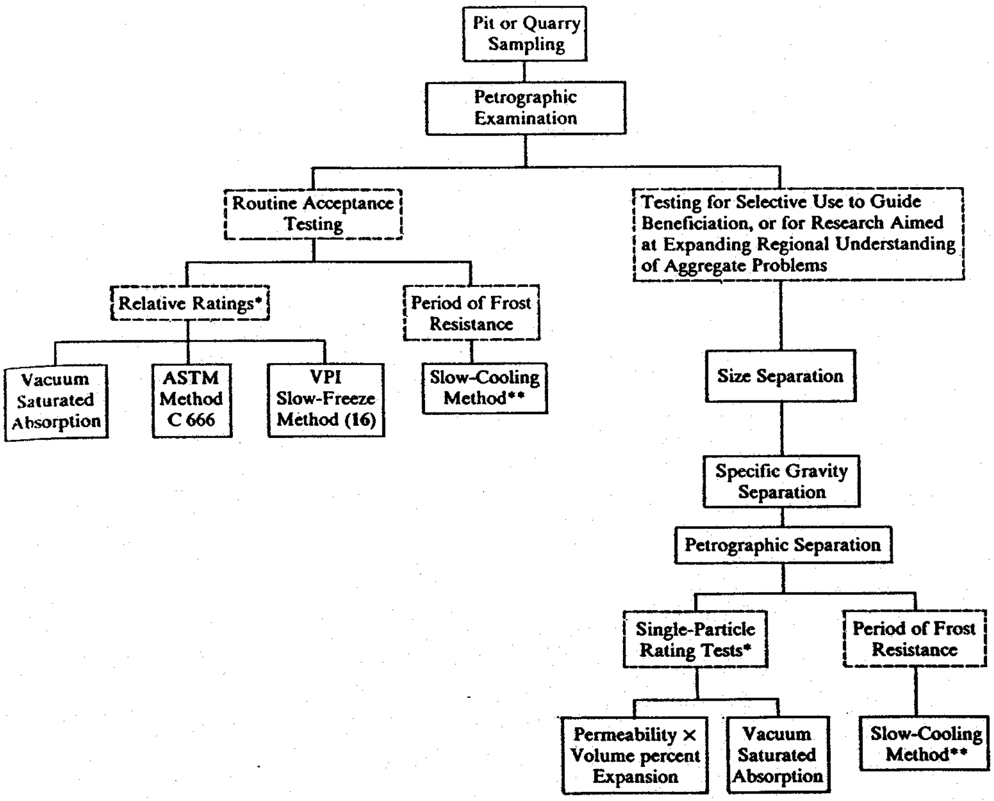
## 4.4 RAPID FREEZE-THAW TESTS

This test was conducted by Dr. J.E. Gillott of the Civil Engineering Department of the University of Calgary, according to the procedures of ASTM C666 - Resistance of Concrete to Rapid Freezing and Thawing. The results are part of Dr. Gillott's report included as Appendix I in this document.

In addition to the comments by Dr. Gillott we would add our own observations. Firstly the tests were conducted in the first place primarily because the scope of the test method under clause 3.1 states "Specific applications (of the method) include - ranking of coarse aggregates as to their effect on concrete freeze-thaw durability, especially where soundness of the aggregate is questionable."

Secondly it was conducted because the concrete samples which were submitted to Dr. Gillott for test had been exposed to the curing conditions, including saturation, to which it is expected the field concrete which the samples represent will be subjected.

Thirdly, in terms of why it was decided to run this test, we refer to ASTM C682 - Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained concrete by Critical Dilation Procedures. Again the scope of this specification under clause 3.1 states "--primarily intended to provide--a technique for estimating the frost susceptibility of concrete aggregates for known or assumed field environmental conditions. The significance of the results in terms of potential field performance will depend upon the degree to which field conditions can be expected to correlate with those employed in the laboratory".



* Interpretation depends on previous experience relating test results to field performance. This information generally is not available.

** As described in this recommended practice.

FIG. 1 Procedural Approaches to Frost-Susceptibility Tests (see Larson and Cady (4))

Finally we include from the same specification, ASTM C682, their Figure 1 - Procedural Approaches to Frost Susceptibility Tests. That figure it will be noted includes the use of ASTM C666 being the test we are discussing.

Despite all the foregoing reasons we feel the results, while accurately reflecting the durability of that particular concrete to rapid freeze-thaw cycles - very poor indeed - do not in fact measure the durability of the aggregates tested. The reason for this conclusion is that the concrete used in the prisms - while standard for that CSA A23.2-14A procedure - is not at all similar to the type of concrete which we would recommend for arctic and freeze-thaw conditions. That is, the concrete tested is lacking a low water/cement ratio; lacking fly ash; lacking a proper air-void system; lacking a water reducing agent and possibly a superplasticizing agent.

Still a comparison of the two specimens which had undergone non-standard conditioning - i.e. from sources at King Point and Running River performed in the rapid freeze-thaw test as they had in the rankings after the length change measurements. That is the King Point saturated concrete sample expanded twice as much as the Running River saturated concrete sample after eight months in 0°C brine and also the King Point rapid freeze-thaw sample deteriorated much faster than the Running River freeze-thaw sample.

## 4.5 THIN SECTION PETROGRAPHY

This work was conducted, as noted previously by Dr. J.E. Gillott of the Civil Engineering Department of the University of Calgary. His photographs and comments are contained as part of his report included as Appendix I of this document.

However, we reach different conclusions, based perhaps on our access to much more data than was available to Dr. Gillott. Nevertheless at this point we will comment initially on the thin section photographs.

Firstly we submitted three specimens for examination by this technique. Two were the concrete prisms cast in accordance with the methods of CSA A23.2-14A - Alkali-Aggregate Reaction. This specification calls for prismatic steel molds not less than 75 mm x 75 mm x 350 mm and it was this minimum size that was used for both the standard curing and the non-standard curing regimes. For the petrographic study we submitted on February 9, 1988 to the University of Calgary three of these specimens- one from the standard curing regime using coarse aggregate from Running River and two from the non-standard curing regime cast using the coarse aggregate sampled from Running River again as well as the coarse aggregate sampled from Moose Creek.

At the same time we submitted one mortar bar cast and standard cured according to ASTM C227 - Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method) which calls for molds 1 x 1 x 11 1/4 inch prisms (Imperial units only are used). The sand was from Moose Creek.

The various samples were about 14 months old when they were examined petrographically and the standard cured specimens had been exposed to that curing for the entire period. The non-standard specimens had been exposed to the brine immersion for the entire 14 months. However for eight of those months the temperature was 0°C while the temperature for the remaining six months is unknown precisely although it is suspected it was likely a few degrees lower.

By the way the choice of these particular specimens was, in the case of the standard cured concrete prism from Running River because it had the greatest expansion at that time; the non-standard concrete prism from Moose Creek also because it had exhibited the greatest expansion while the non-standard from Running River simply to compare with the standard cured specimens from the same aggregate source. Similarly the mortar bar for standard cured specimen using sand from Moose Creek was chosen because it has shown the greatest expansion to that point.

Now dealing with these in the order in which the photographs appear in Dr. Gillott's report we note the first in his Figure 2 is the mortar bar using sand from Moose Creek. The minerals are all siliceous and so, after more than a year in contact with high alkali cement (1.06% expressed as equivalent $Na_2O$) in a moist environment we would expect the maximum reactive material would have appeared if it were going to. Additionally grey wacke which is normally thought to contain $Na_2O$, happens to appear in this photo.

Despite this opportunity for alkali-reactivity to occur we see no evidence of rim material growing on the aggregate-paste interface. We therefore conclude that while some expansion has occurred it is not sufficient to develop sufficient strain to disrupt high quality concrete.

The second set of photographs in Dr. Gillott's report (Figure 3) are also of the same mortar bar using Moose Creek sand. The grey wacke particle is again relatively inert in terms of lack of rim material.

Chert is present but Dr. Gillott in his comments notes this type of chert is non-expansive.

The quartzite, while strained, does not in our opinion, show signs of significant expansion after more than one year in this environment.

Thus, this mortar bar cast using Moose Creek sand which had shown the greatest expansion after one year of all the sands which we had cured under standard ASTM C227 test conditions does not show signs of significant expansion indicative of alkali-reactive problems. This is precisely the information given by Figure 5 in our own report being the long term expansion curve for this Moose Creek sand.

Turning now to Figure 4 of Dr. Gillott's report we note these photographs are from concrete made from coarse aggregate sampled from Running River. The sand in these concrete prisms, cast using CSA A23.2-14A procedures was local from the pit of a large long-time Calgary ready mix concrete firm. That sand has the fineness modulus required by the test procedure and has a 75 year history of satisfactory use in concrete.

The Running River aggregate, in our own standard expansion tests, the results of which are shown in our Figure 2, did produce the greatest expansion at 14 months relative to all the other coarse aggregates tested. It is perhaps not unexpected therefore that we see some evidence of movement as indicated by the microcrack in Photograph A of Dr. Gillott's Figure 4. The grey wacke piece shown is a very fine sand particle and hence not from Running River. More importantly in our opinion is the absence of reactive gel around the rims of the three coarse aggregate pieces also evident in the photo and which are from Running River.

Figure 5, Photo A, from Dr. Gillott's report is also from a concrete prism cast using Running River coarse aggregate and which had been cured in a standard manner for 14 months at the time these photographs were taken. An entrapped air void is visible with gel adhering to its surface. The balance of the view is largely of mortar incorporating Calgary sand. It is possible but not positive that the fragments on the upper left and right of this photograph are parts of coarse aggregate which if that were the case would indeed be from Running River. In either case there is an absence of rim build up material which if present in large quantity would be the cause of significant damaging alkali-reactivity.

Figure 6, Photograph A, of Dr. Gillott's report is a concrete prism cast using Running River coarse aggregate but cured in non-standard cold brine for 14 months at the time the photograph was taken. We note no particular visible sign of expansion in the photo.

Similarly in Photo B of Figure 6, the only possible piece of coarse aggregate, which is the only portion of the prism which could be from Running River, is that shown in the left hand part of the photo. Excellent bond with the paste is evident concurrent with the absence of rim material on that piece.

We again conclude there is no evidence of significant alkali-reactivity with the coarse aggregate.

## 4.6 SCANNING ELECTRON MICROSCOPY

This portion of the total study was again conducted by Dr. J.E. Gillott of the University of Calgary and is included in his report forming Appendix I of this Klohn Leonoff Ltd. report.

The concept in this case was that our original petrographic analysis conducted according to CSA requirements, had identified a significant portion of chert in each of the coarse aggregates sampled from the six sites along the Beaufort Sea. Chert can be highly expansive and hence disruptive to concrete, or not depending largely on the size of its microcrystal line structure. Because of the presence of chert and the inability to determine visually whether or not it is reactive, the CSA standard requires that an inordinately high petrographic factor be assigned to chert. This has the effect, in total, of producing a high petrographic number which, in turn, is a signal that further testing is required prior to use in quality concrete. This characteristic of crystalline structure is best determined by use of the scanning electron microscope.

Accordingly chert samples from four of the five sample sites along the Beaufort, considered to be the most promising, were submitted to Dr.

Gillott for processing by the S.E.M. The locations and amount of chert in terms of percent of coarse aggregate are as follows:

| Location | % Chert |
|---|---|
| Shingle Point | 35 |
| Running River | 32 |
| King Point | 38 |
| Jacobs Ridge | 40 |

Based on the S.E.M. micrographs of Figure 7 we agree with Dr. Gillott's assessment of the chert, found on page 8 of his report:

> "Observation showed that the cherts were relatively dense and made up of irregularly shaped interlocking particles of quartz in the 2 to 4 μ size range (Fig. 7). The interlocking texture and apparent absence of pores suggests that these cherts may show only low reactivity in alkali and may not be of the frost-susceptible variety."

Phrased more forcefully we would in fact state that based on the above and all the other tests conducted in this study that the chert in all cases appears to be innocuous in terms of long term damage to concrete.

## 5.0 CONCLUSIONS

Based on a review of the three parts of this study, A, B and C, it is concluded that the aggregate sources which we sampled at Shingle Point, Running River, King Point and Jacobs Ridge are the most likely sources of concrete aggregate for use along the western Beaufort Sea from the points of view of accessability, area requirements and quality. Further field testing to determine both quantities available and more sampling to determine homogeneity or otherwise of the various sources would be required since discontinuities compared with some other test data by others based on samples from the same sources have already been noted.

On the basis of the full suite of test results, we disagree with the conclusions of Dr. J.A. Gillott that the aggregates will be unsuitable due to alkali-aggregate reactivity. On the contrary, satisfactory concrete may be made in our opinion, using aggregate from the four sources noted - subject to further sampling to verify the degree of homogeneity - provided proper concrete design and construction considerations for arctic conditions are followed. These would include as per our initial report of this series (Klohn Leonoff, 1988a - section 11) an appropriate water/cement ratio; an adequate air-void system; use of appropriate additives including fly ash, water reducing agents and likely a superplasticizing agent; proper curing and likely prestressing forces in the order of 10 MPa to minimize crack widths.

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Yours very truly,

KLOHN LEONOFF LTD.

L.E. RODWAY, Ph.D., P.Eng.
Manger, Materials Engineering Division

LER/sh

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# APPENDIX I

## Report on Beaufort Sea Aggregates

by J.E. Gillott, D.Sc., P.Geol.

# "Concrete Aggregate Tests, Beaufort Sea".
# Properties of Concrete Prisms Subjected to
# Cyclical Freezing and Thawing
# and Petrographic Characteristics

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# "Concrete Aggregate Tests, Beaufort Sea". Properties of Concrete Prisms Subjected to Cyclical Freezing and Thawing and Petrographic Characteristics

by

J.E. Gillott, D.Sc. (Eng.), P.Geol.

M & S Job 279

March 1988

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## Summary

1. Three concrete prisms were subjected to cyclical freezing and thawing in an apparatus similar to that described in ASTM C666. Changes in fundamental transverse frequency, weight and dimensions were determined.
2. A sample is considered to have failed the test if its fundamental frequency drops below 60% of its initial value in less than 300 cycles of freezing and thawing.
3. Sample KP6 failed the test after 4 cycles, sample SP3 failed after 57 cycles and sample RR6 failed after 59 cycles. Durability factors were low in all three cases. Concrete of that sort may be expected to show poor resistance to frost action.
4. Sample KP6 registered the largest increase in length and the greatest decrease in weight of the three samples.
5. Petrographic examination showed that the aggregate was made from a gravel composed of an assortment of rock types. Greywacke, chert, quartzite, argillite and limestone were present with smaller particles of similar rocks and individual grains of strained quartz in the sand fraction.
6. Aggregate particles showed only a few rims and the concrete and mortar displayed relatively little microcracking. Voids were generally empty but a thin lining of silica gel was present in a few instances.
7. Rocks and minerals of the type present in these gravels have been associated with alkali-aggregate reactions in other places.
8. If used as aggregate in portland cement concrete it is very probable that durability problems will develop due to deleterious alkali-aggregate reactions.

## Introduction

The following report was prepared at the request of Dr. L.E. Rodway, Manager, Materials Division, Klohn Leonoff Ltd. The work involved determination of changes in properties of three concrete prisms subjected to cyclical freezing and thawing using an apparatus similar to that described in ASTM C666. Also petrographic examination was made of several samples of chert, separated from aggregate, and of three concrete prisms and a mortar bar.

## Cyclical Freezing and Thawing Test - Samples SP3 (standard), KP6 (non-standard) and RR6 (non-standard)

Three concrete prisms (76.2 mm x 76.6 mm) were subjected to cyclical freezing and thawing as described in ASTM C666, Procedure A. Two of the concrete prisms had been stored in near-freezing brine (non-standard conditions) and the third sample had been stored under standard conditions. Prior to test all three specimens were immersed in water and initial weight and dimensions were established by taking a series of readings. Weight was measured to the nearest 0.1 g and length was measured to the nearest 10 $\mu\varepsilon$ on steel end studs using a dial guage extensometer. Very little variation in weight or dimensions occurred after 48 hours and a reading taken after 6 days was used as the zero value.

During test the temperature of the specimens submerged in water was alternately lowered from 40° to 0°F (-17.8° to 4.4°C) in cycles according to the specification. Changes in fundamental transverse frequency, weight and dimensions were determined after 5, 10, 14, 30, 40, 49, and 64 cycles of freezing and thawing. Specimens were continued in the test until the fundamental transverse frequency dropped below 60% of its initial value.

In general, a sample is considered to have failed the test if its fundamental frequency drops below 60% of its initial value in less than 300 cycles of freezing and thawing. A durability factor was calculated for each specimen using a similar method to that described in ASTM C666. It is noted that the procedure in ASTM C666 is intended to apply to concrete prisms cast and cured for testing according to certain requirements. In the present case two samples had been stored in near freezing brine so conditions were non-standard.

Results are presented graphically in Fig. 1. The fundamental transverse frequency of sample KP6 (non-standard conditioning) dropped below 60% of its initial value after only 4 cycles of freezing and thawing. The two remaining samples, SP3 (standard conditioning) and RR6 (non-standard conditioning) performed better but the fundamental transverse frequency dropped below 60% of its initial value after 57 cycles in the case of sample SP3 and after 59 cycles in the case of sample RR6. Durability factors were 0.8 % for sample KP6, 11.4 % for sample SP3 and 11.8% for sample RR6.

The largest decrease in weight was shown by sample KP6 which scaled badly early in the test. Sample RR6 showed an intermediate rate of weight decrease and SP3 showed the smallest rate of weight loss of the three samples.

The most rapid and largest increase in length was registered by sample KP6. Sample SP3 showed intermediate values of expansion and sample RR6 showed the smallest expansion of the three samples.

In conclusion all three concrete prisms subjected to cyclical freezing and thawing in an apparatus similar to that described in ASTM C666 showed evidence of deterioration after a relatively small number of cycles. All samples are considered to have failed the test.

## Petrographic Examination: Concrete RR1 (standard conditioning); RR4, CM4 (non-standard conditioning); Mortar CM1; Four Chert Samples

Petrographic examination was carried out on three concrete prisms, one mortar bar and four samples of chert separated from a gravel aggregate. Two of the concrete prisms had been subjected to non-standard conditioning (RR4, CM4) and one concrete prism had been subjected to standard conditioning (RR1). The mortar bar (CM1) had also been subjected to standard conditioning for ASTM C227. The samples of chert were from four localities: Shingle Point (SA1), Running River (SA2), King Point (SA3) and Jacob's Ridge (SA4). Thin sections were prepared from the three concrete prisms, from the mortar bar and from four pieces of chert judged to be representative, one piece having been selected from the pebbles supplied from each sample location. Samples of the chert were also examined on the scanning electron microscope.

Prior to examination of the thin sections part of the concrete prisms was removed with a diamond saw and broken up with a hammer and chisel. Fracture surfaces were examined with the unaided eye and on the low power binocular microscope. In general, voids were empty or contained only a very thin lining of deposit which in some instances included needle-like crystals of ettringite; some of the other void linings were shiny and may have contained very small quantities of silica gel. For the most part aggregate particles showed little or no rim formation.

The thin sections showed that the aggregate in all samples was similar and made up of a heterogeneous assortment of rock types. Particles were composed of greywacke, chert, quartzite, argillite and limestone with smaller particles of similar rocks and strained quartz grains or crystals in the sand fraction (Figs. 2 to 6). It is possible that the proportions of the rock types differed in the different sections but sampling may account for the apparent differences.

The greywacke contained angular grains of quartz with sufficient chert for these rocks to be classed as lithic greywackes. Plagioclase feldspar showing lamellar twinning with extinction angles indicative of the Na-rich end of the series, iron minerals and flakes, sometimes bent, of biotite, muscovite and chlorite were also present. The fine matrix contained sericitic and clayey material. Some particles of greywacke contained calcite in the matrix whilst others did not. The predominant types of greywacke were similar in the different samples, e.g. CM1 (Fig. 2B) and RR1 (Fig. 4B). In one or two instances greywacke particles were cracked and occasionally the crack within the aggregate particle could be seen to continue into the cement paste (Fig. 4A).

The chert consisted mainly of microcrystalline silica which was sometimes partly recrystallized (Figs. 3A, 5B). Iron minerals (iron pyrite and limonite) were sometimes present and particles were frequently cut by thin veins of quartz.

The quartzite consisted mainly of interlocking quartz crystals with very small amounts of other minerals (Fig. 3B). In places the outline of detrital sand grains was visible enclosed within an overgrowth of secondary quartz deposited in optical continuity with the parent grain.

The argillite consisted of microcrystalline quartz and oriented fine grained micaceous material composed of muscovite, biotite and chlorite. Thin veins of quartz were common (Fig. 6A).

The concrete and mortar contained a few microcracks and voids which were generally empty but occasionally voids contained a thin lining of cryptocrystalline material which was in part composed of silica gel (Fig. 5A)

Thin sections were also examined made from four of the chert particles - one from each locality. The particles of chert were supplied and had evidently been separated from gravels. The material came from Shingle Point (SA1), Running River (SA2), King Point (SA3) and Jacob's Ridge. The chert was composed of microcrystalline silica together with iron minerals and was similar to the material incorporated as aggregate in the concrete and mortar.

Particles of chert were fractured, mounted on stubs, etched for 10 seconds in 10% hydrofluoric acid and vacuum coated with gold prior to observation on the scanning electron microscope. Observation showed that the cherts were relatively dense and made up of irregularly shaped interlocking particles of quartz in the 2 to 4 μm size range (Fig. 7). The interlocking texture and apparent absence of pores suggests that these cherts may show only low reactivity in alkali and may not be of the frost-susceptible variety.

### (i) Discussion of Petrography

The aggregate contains rock types which have been associated with alkali-aggregate reaction. Greywacke and argillite were prime suspects in concrete durability problems in Nova Scotia, Ontario and elsewhere (Dolar - Mantuani, 1969; Duncan et al., 1973). Chert has been long recognized as potentially alkali reactive and some quartzites have been found to be alkali expansive (Gillott, 1986). Limestones, when siliceous, and certain dolomitic varieties have also been found to be alkali reactive.

Greywackes are a variety of sandstone containing more than 15% fine-grained matrix material. Framework grains consist of poorly sorted and angular quartz, feldspar and rock fragments. The matrix generally contains fine micaceous material or clay minerals, iron minerals and sometimes calcite. The fine matrix was originally considered to have been the muddy component of turbidity currents deposited at the same time as the coarser grained mineral and rock fragments. More recently authors have supported the suggestion made by Cummins (1962) that much of the matrix formed after deposition by diagenesis or low grade metamorphism of unstable minerals or rock fragments.

Argillites are compacted or weakly metamorphosed mudstones in which fine quartz grains are distributed in a matrix of fine clayey material.

Chert consists of interlocking microcrystals of quartz less than about 30 µm across; it sometimes contains opal, or other non-crystalline forms of silica, together with impurities such as carbonates and iron minerals. In the aggregates under discussion chert occurs as independent fragments and also as a significant framework constituent in the greywackes.

Quartzites are also composed primarily of quartz crystals but their dimensions are larger than those in cherts. The quartz crystals are welded together due either to recrystallization, caused by metamorphism, or to the deposition of secondary silica on the grains of a sedimentary sand. The larger crystal size implies that quartzites are less reactive chemically than cherts. As mentioned previously, some rocks of this kind, such as the Mount Wilson quartzite in Western Canada, show alkali expansivity.

The "classical" explanation for the role of aggregates of this type in alkali-aggregate reactions is that they contain or are composed of poorly ordered silica minerals. Chert is present both as individual particles in the aggregates and as sand-sized grains within the greywackes. The greywackes also contain fine silica and fine micaceous minerals in the matrix which may also contribute chemically or physically to alkali expansivity.

A number of authors have shown that various minerals including feldspars may release alkalies into concrete pore solutions and so increase the potential risk of alkali aggregate reactions (Hansen, 1944; Hadley, 1964; Stark, 1978; Grattan-Bellew and Beaudoin, 1980; Stark and Bhatty, 1985). The greywackes in the present aggregate contain a soda-rich variety of feldspar and as a result these rocks probably have a relatively high content of $Na_2O$ - as commonly reported in typical greywackes. These rocks therefore not only contain minerals which may lead to expansion but also have a potential to release additional alkalies into solution which would aggravate the situation. Hence, if these rocks are used as aggregates in portland cement concrete, there is a strong probability of deleterious alkali-aggregate reactions.

## Conclusions

The cyclical freezing and thawing test indicates that the concrete samples supplied have a low resistance to frost-action. Petrographic examination showed that the aggregates are composed of gravels made up predominantly of greywacke, chert, quartzite, argillite and limestone; the sand fraction contained fragments of similar rocks and quartz grains which were sometimes strained. Rocks and minerals of this type have been associated elsewhere with alkali-aggregate reaction. If used as aggregate in portland cement concrete there is every likelihood that durability problems will develop due to deleterious alkali-aggregate reactions.

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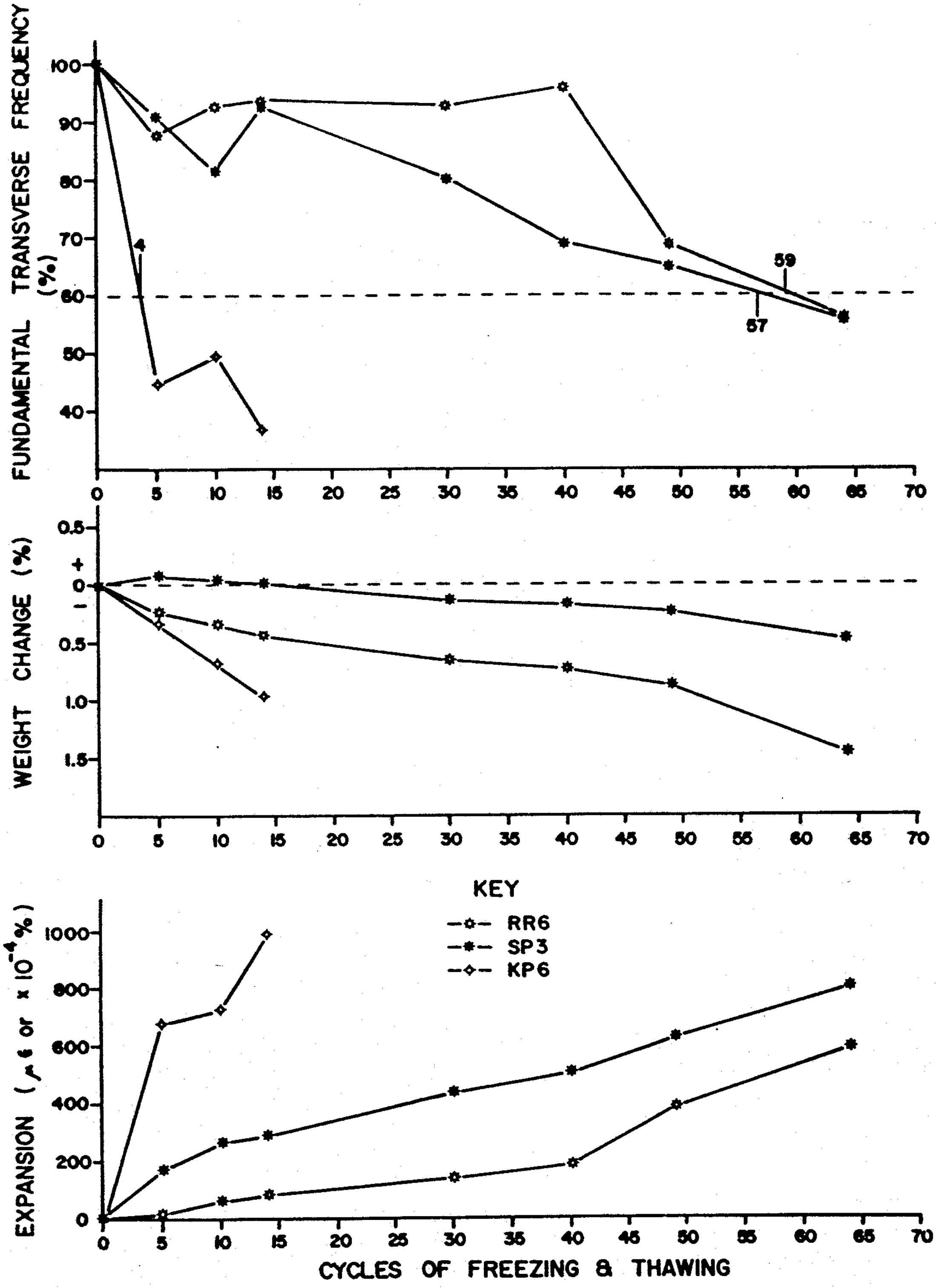
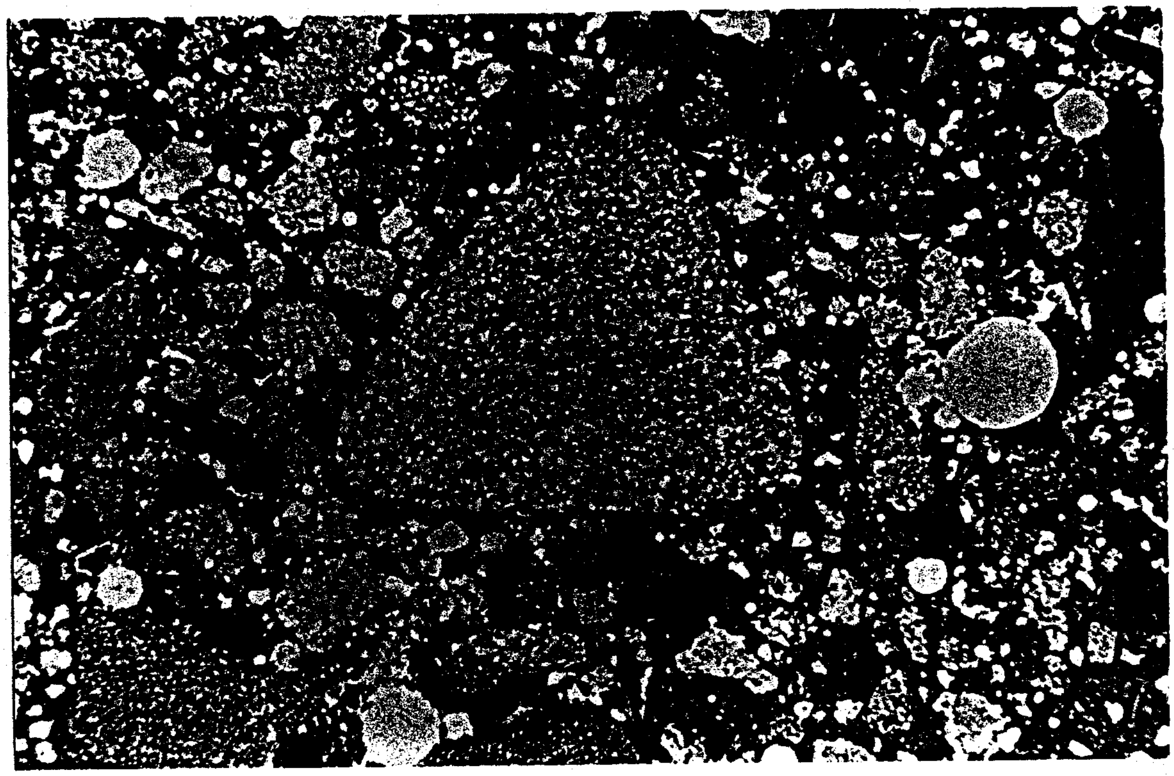
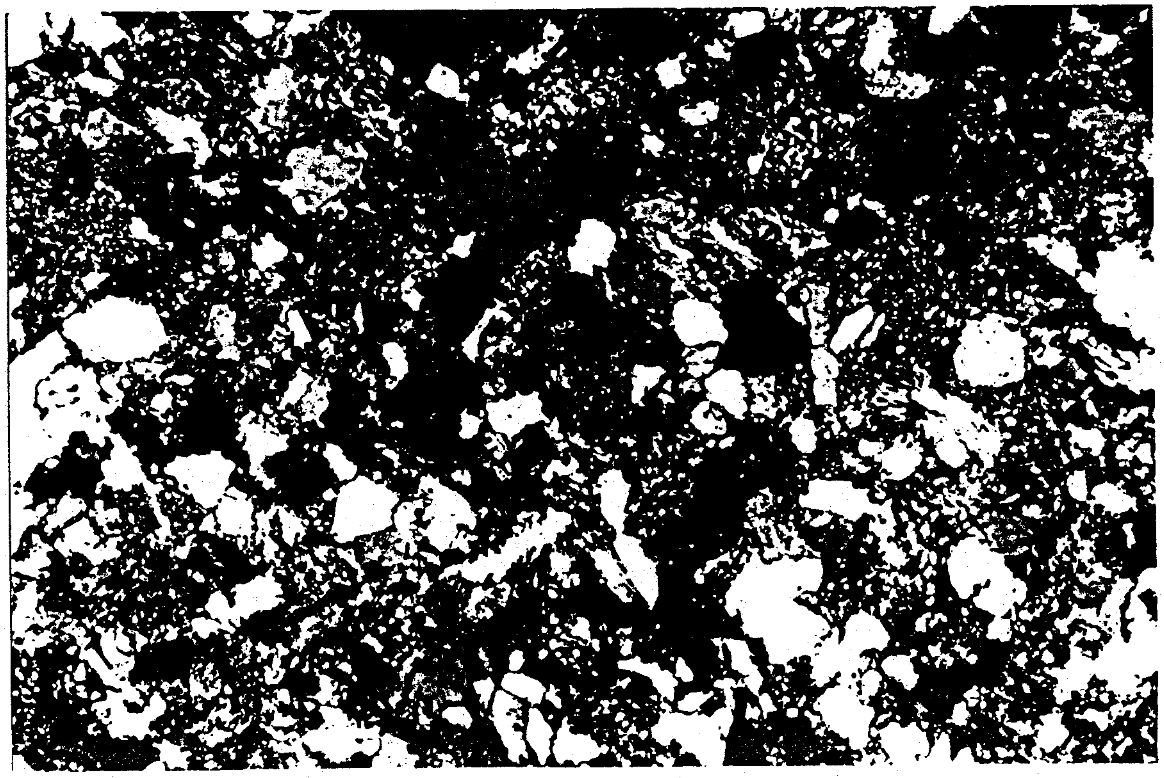


FIG. I CHANGES TO PROPERTIES OF CONCRETE PRISMS SUBJECTED TO RAPID FREEZING & THAWING (ASTM C666).

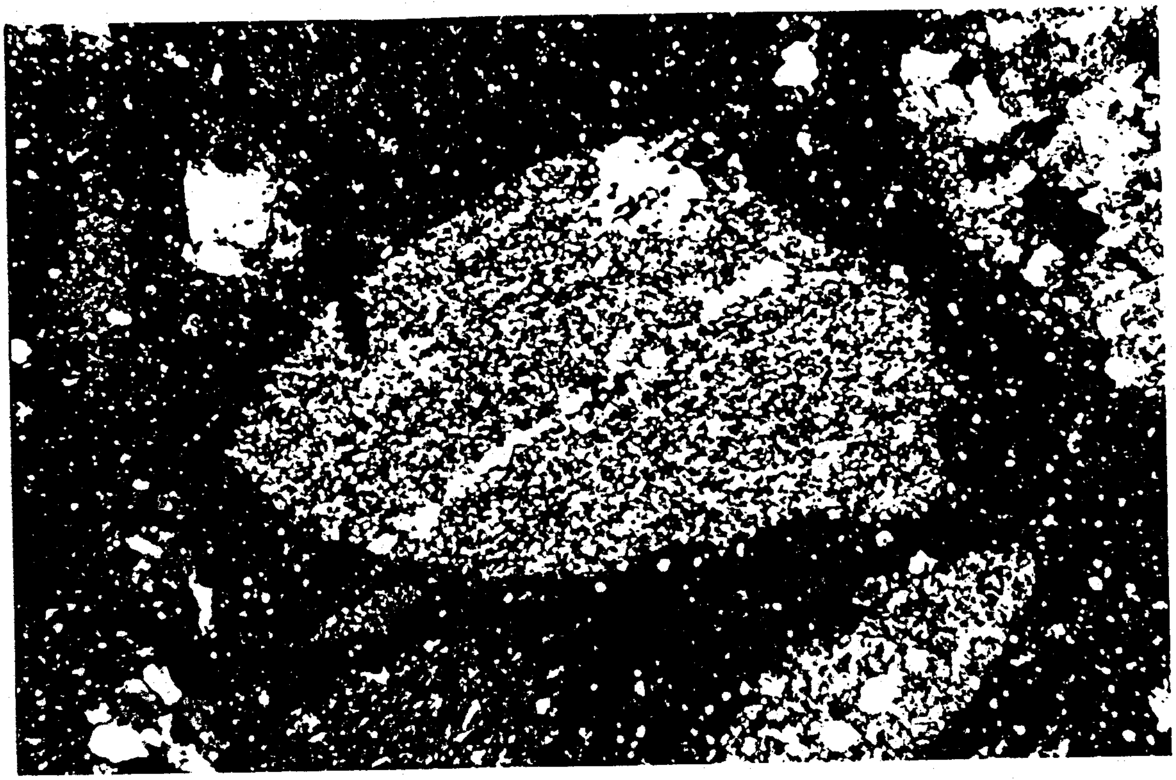


A. Particles of Greywacke and Quartz. Ordinary Light.

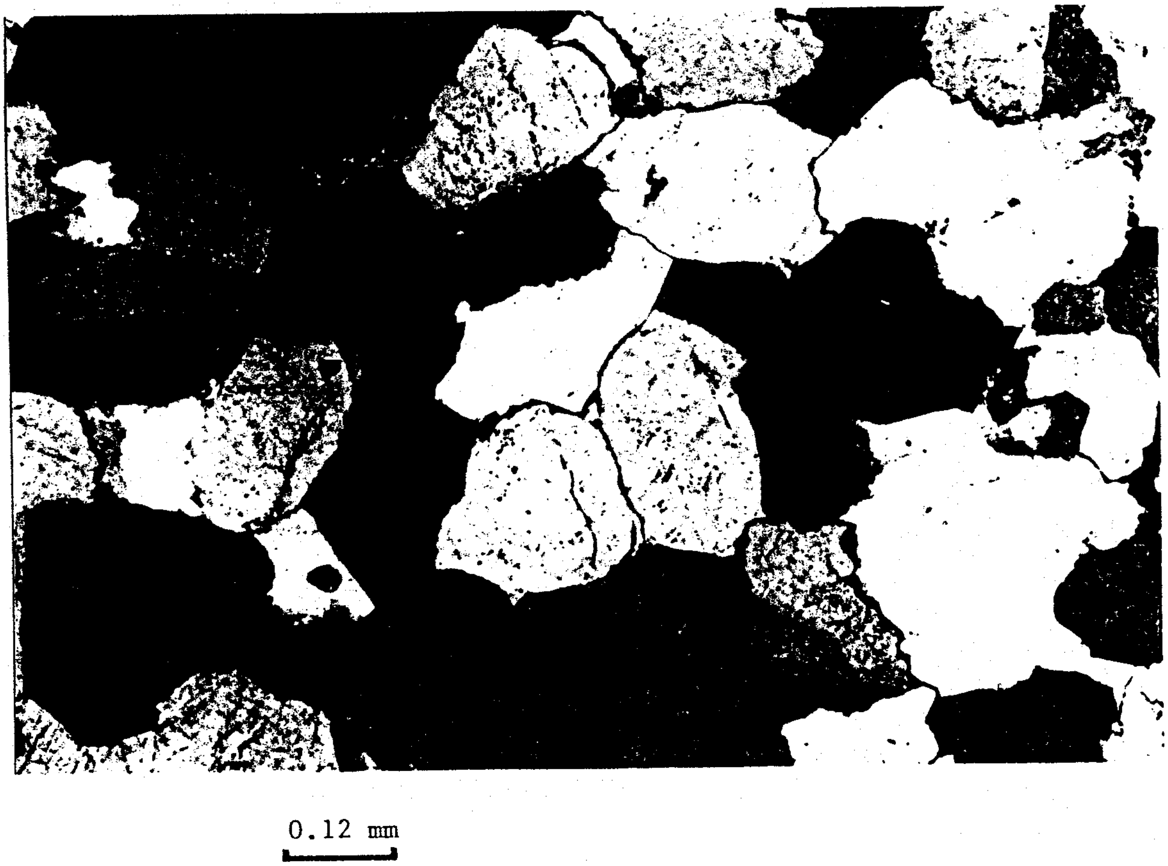


B. Greywacke containing Chert, Quartz, Feldspar and Mica Grains. Crossed Polarizers.

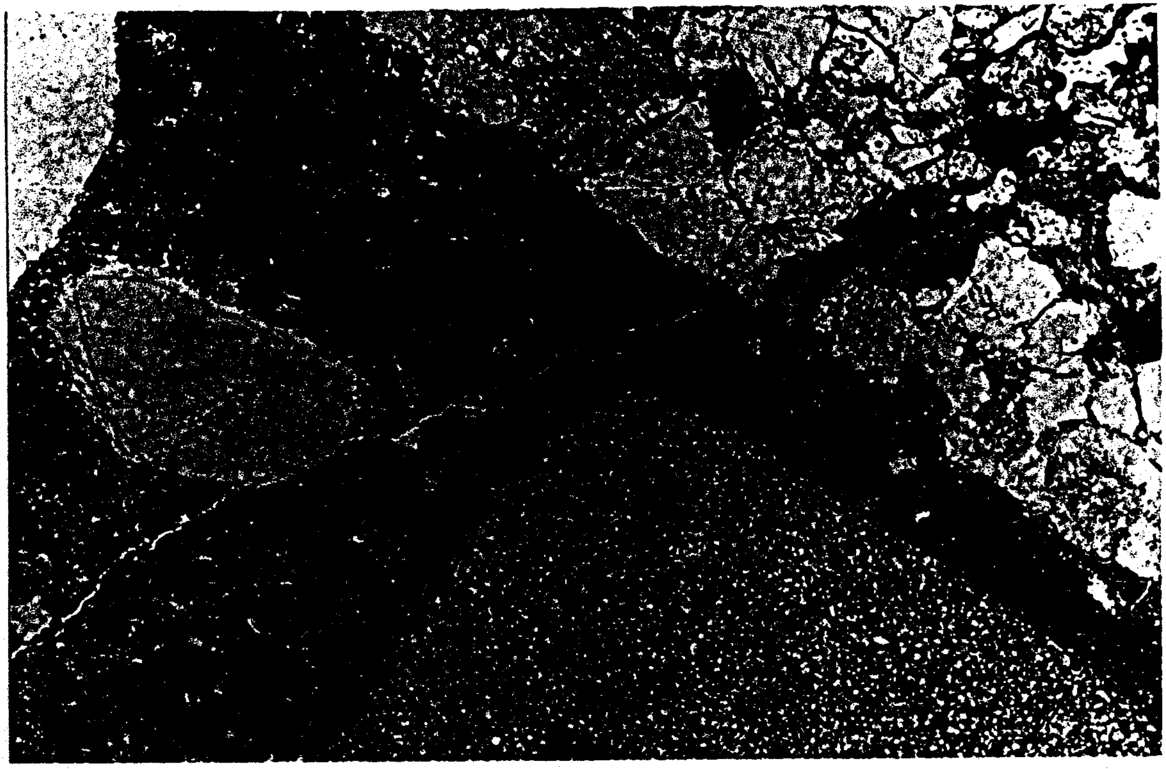
Fig. 2. Optical Micrographs of Mortar, CM1



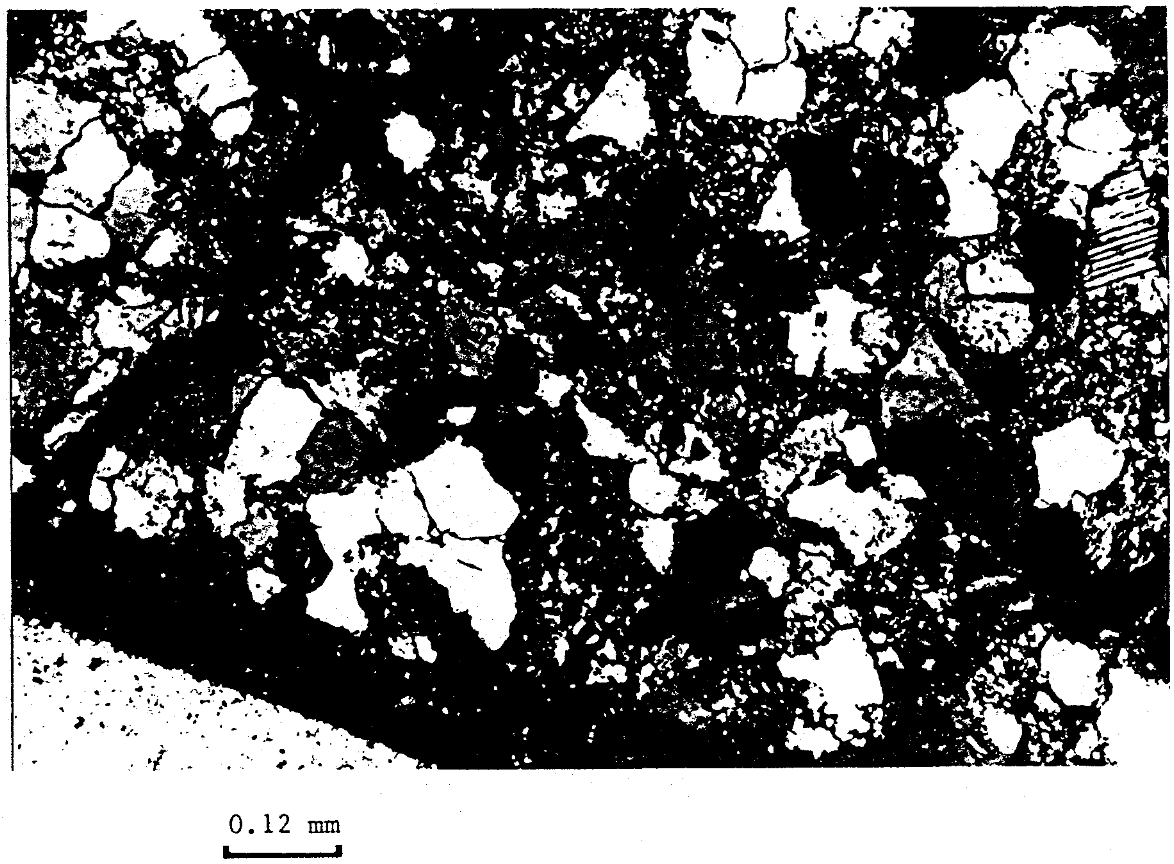
A. Chert and Greywacke Particles. Crossed Polarizers.



B. Quartzite Particle.
Fig. 3. Optical Micrographs of Mortar, CM1.
Crossed Polarizers.

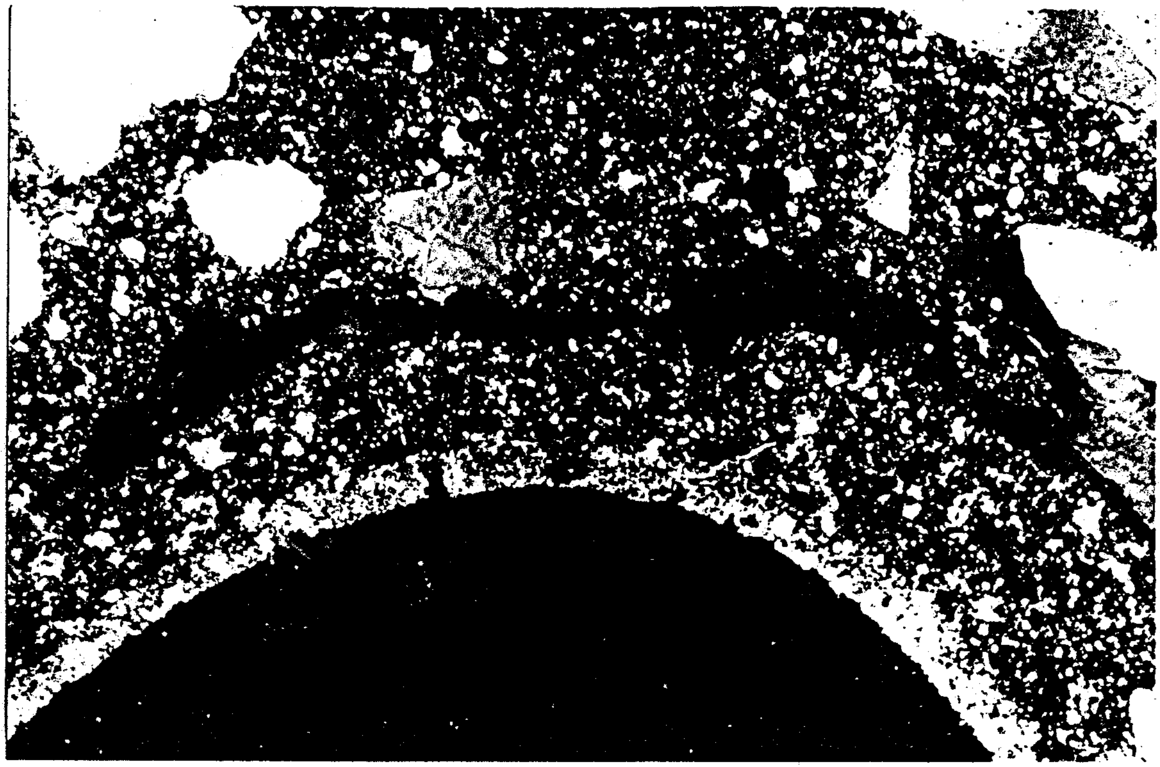


A. Microcrack in Cement Paste linked to cracked Greywacke. Ordinary Light.

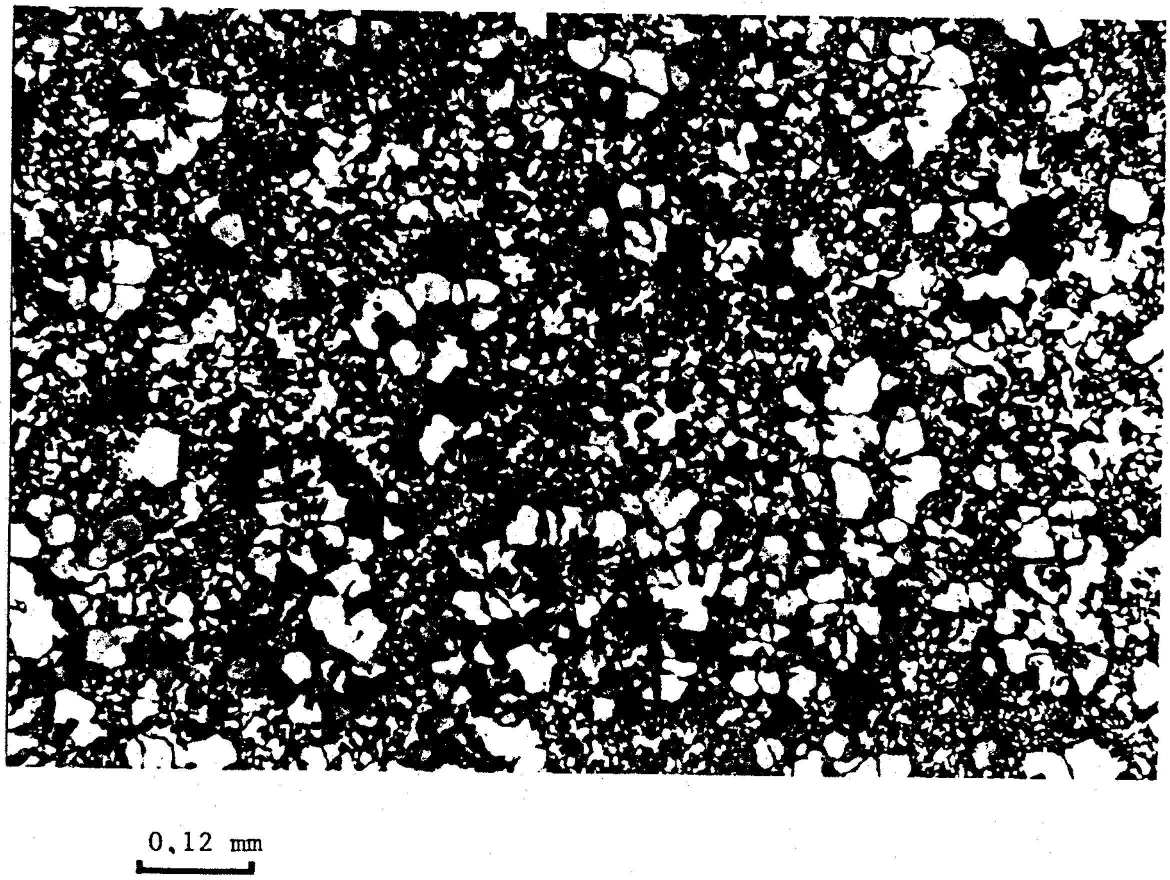


B. Cracked Greywacke. Quartz, Chert, Plagioclase, Mica in Fine Matrix. Crossed Polarizers.

Fig. 4. Optical Micrographs of Concrete, RR 1.



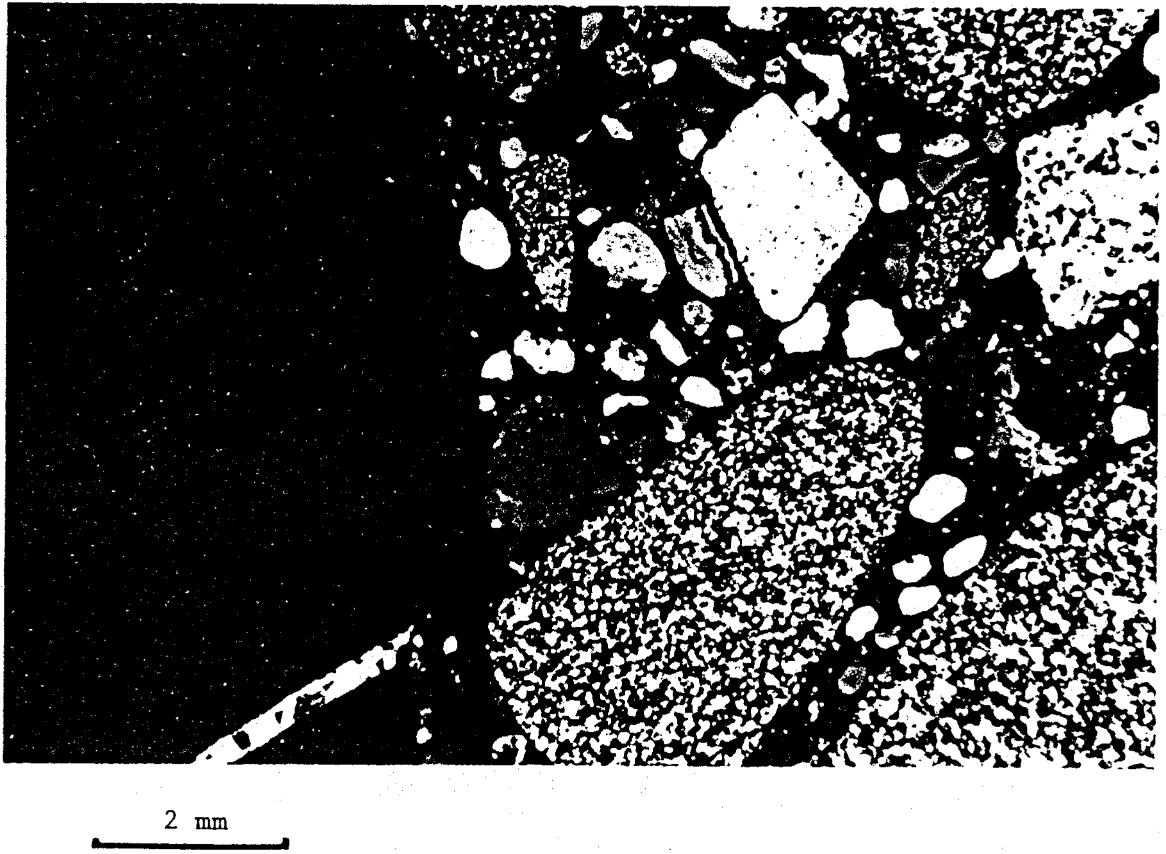
A. Void with thin lining of Silica Gel and concentric Crack. Crossed Polarizers.



B. Partly Recrystallized Chert Particle. Crossed Polarizers.

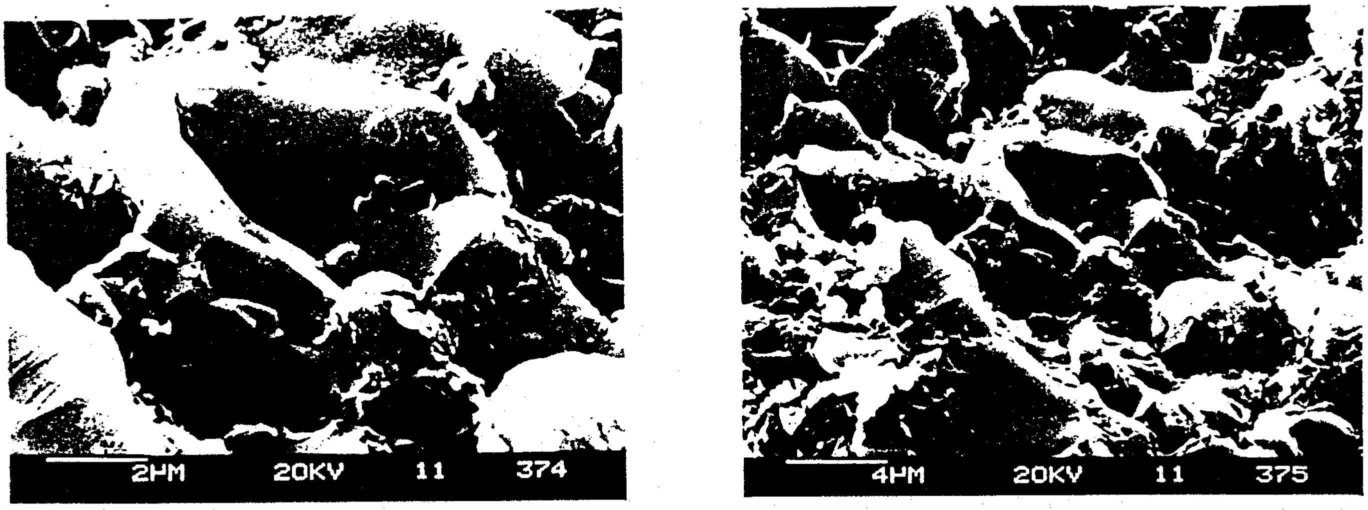
Fig. 5. Optical Micrographs of Concrete, R.R. 1.

A. Argillite showing Oriented Layer Silicates and thin Quartz Vein. Crossed Polarizers. Concrete R.R. 4.



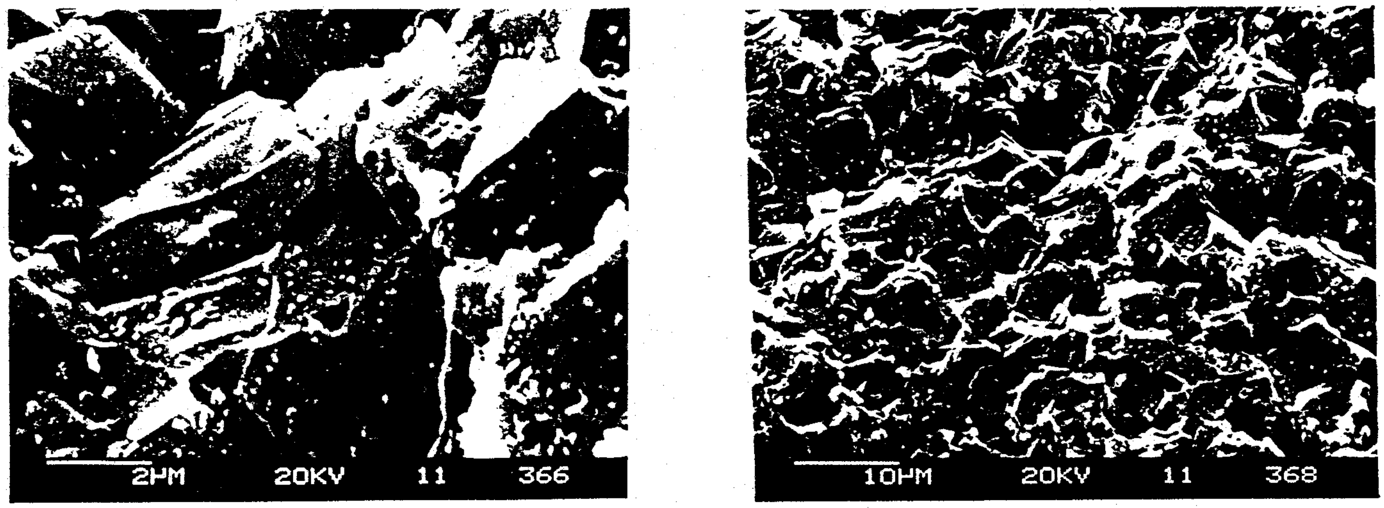
B. Particles of Chert, Greywacke, Limestone and Quartz Sand. Crossed Polarizers. Concrete R.R. 1.

Fig. 6. Optical Micrographs of Concrete, R.R. 4 and R.R. 1.



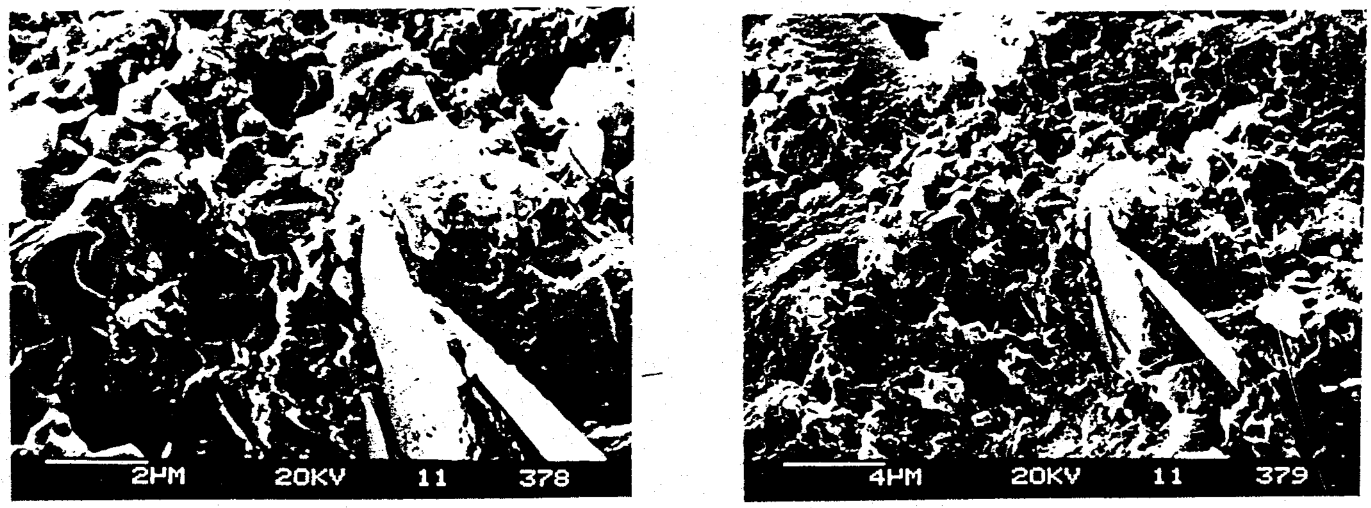
A. SA1. Shingle Point.

B. SA1. Shingle Point.



C. SA2. Running River.

D. SA2. Running River.



E. SA4. Jacob's Ridge.

F. SA4. Jacob's Ridge.

Fig. 7. Scanning Electron Micrographs of Chert Particles. Etched 10 secs., 10% HF.