REPORT ON TEST DRILLING PROGRAM, UPPER LAWRENCETOWN, HALIFAX COUNTY

BY: HEATHER J. CROSS

Wells L1, L2 and L4 were
plugged with crusher dust and Hole plug (medium) bentonite
on 5 August 1994 [illegible]

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Report On

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Upper Lawrencetown,

Halifax County

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N.S. Department of the Environment
Water Planning and
Management Division

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INTRODUCTION

Purpose and Scope

During March 1977, a test drilling program was undertaken in the Lawrencetown area because of several factors: (1) the number of complaints of salt water intrusion received (2) the rapid development in the area, both through subdivision development and ribbon development (3) the number of arsenic problems reported in the area.

The purpose of the project was to try to determine (1) any geological factors related to the intrusion problem (2) the geochemistry of intrusion (3) the position of the salt water/fresh water interface (4) the implications of these results for shoreline development along the coastal zone (5) what technical or management strategies could best be used to help reduce future problems.

Location and Access

Figure 1 shows the location of the site.

The site is located about 3 miles east of Dartmouth city limits, and just east of the discharge of Little Salmon River into Cole Harbour Estuary.

Primary access to the site is readily available by paved road - either by highway 7 to Ross Road (hwy 328) then east onto highway 207, or by Cole Harbour Road which becomes highway 207. Secondary access is on private property (Mr. Burton Patterson) by a gravel road about 200 feet long which was constructed specifically for the project.

Climate

The closest station with long term data is CFB shearwater (27 years). Climatic records from this station show an average annual precipitation of 54.4", with 46.4" occurring as rain and 80.4" as snow. The minimum mean monthly precipitation of 3.4" occurs in June, the maximum of 6.0" in November (Atmospheric Environment Service, 1972).

The mean annual temperature is 44.3°F, with the minimum of 15°F below zero occurring in January, and the maximum of 92°F in July. The mean frost free period is 173 days, with the first fall frost occurring on about 26th, and the last spring frost on May 15th (142 days with frost).

A summary of the precipitation and temperature data for 1971-79, and the 27 year mean, are presented in appendices 1 and 2.

Physiography, Drainage and Vegetation

The study area is located in the physiographic region known as the Southern Upland, a dissected tableland with maximum altitudes ranging from 600-700 feet (Goldthwaite, 1924). The Lawrencetown area has a rolling topography, with broad, smooth ridges and hollows generally following a northwest-southeast trend controlled by bedrock structure. Quartzite outcrops on many of these ridges.

The drainage is evidently controlled by bedrock structure - the depressions occupied by Lake Major and Little Salmon River to the west, and Long Lake and Robinson Brook to the east are linear in a northwest-southeast direction. The lakes were likely formed or scooped out and widened during Pleistocene glaciation, along pre-existing zones of weakness.

On a regional scale, faulting in a northwest-southeast direction is common. The local area of the test site is relatively flat, with a bouldery surface. Vegetation is mainly scrub and alder, with a few mature fir and spruce trees.

Drilling Program

Two piezometer nests were constructed. One, consisting of wells L1 and L2, was located about 50 feet from mean sea level, with L2 completed to 75 feet and L1 to 200 feet below ground. The second nest was constructed 100 feet inland from the first, with depths of 75 feet (L4) and 174 feet (L3).

The drilling was carried out by H. J. Edwards Well Drilling with the driller being Barry DuBaie and helper Harry Edwards. The total cost of road construction and drilling was $10,000.

The rig used was an Ingersoll-Rand air rotary with down the hole hammer. Drilling progresses by subjecting the rock to rapid high-speed impacts while the drill bit is rotated, causing compression and shear. The flusing medimum (air) is rock diverted under pressure to the face of the drill bit to cool the bit and lift cuttings. The advantages of this method are (1) speed of drilling (2) cheaper unit cost compared to a cored borehole, and (3) useful for monitoring (Horner et al, 1977). Some disadvantages of the method include (1) difficulties in keeping hole from collapsing in non-cohesive soils (2) high bit wear in siliceous rocks (3) jamming in highly fractured rock (4) the samples are of sufficient size to identify the rock, but are usually insufficient to determine intact physical properties and (5) noise and dust. During the program, there were two jamming problems, and the hammer broke three times (attributed to metal fatigue).

The penetration rate with this type of drilling depends on several factors (Horner, 1977): (1) rock type-hardness, strength and fracture frequency, (2) equipment-make and model of drill, bit size, type of bit, (3) flushing medium and pressure used, and (4) drilling personnel. This rate during the test program was generally from 0.5-1.0 foot per minute.

Literature Review

A literature search was conducted on salt water intrusion and is contained in Appendix 3. Topics covered include hydrogeological factors, interface dynamics, geochemical factors, and control methods.

GEOLOGY

Surficial

The surficial material consists of a yellowish-brown silty sand till with a large number of quartzite boulders. This till is thin (about 5 feet at most) and sporadic, and may infill between large boulders broken off the bedrock by weathering.

Bedrock

The bedrock is mapped as predominently quartzite/feldspathic graywacke of the Goldenville Formation of the Lower Ordovician Meguma Group. However, large percentage of slaty interbeds occur, especially where associated with quartz veins. As shown on figure 2, the site is just west of the Lawrencetown Gold District, and quartz veins are mapped near the site.

Drill chips indicated that the quartzite was predominently grey to greenish-grey with minor quartz and calcite stringers. The slate varied from a somewhat sandy to silty texture and was black to dark grey, depending on the proportions of clay and silt to sand size matrix. Quartz veins were more common in the slatey layers than the quartzitic ones. A summary of the testhole logs is contained in appendix 4. Detailed daily records are on file at N.S. Department of the Environment.

The bedding in the area strikes east to northeast and dips south to southeast at angles of 20-48^{o}. The site is located close to the anticline axis, where beds are distorted by the plunge of the fold (plunges west to southeast). A fault striking northwest runs about 0.5 miles west of the test site and controls part of the Little Salmon River channel. Fracturing associated with this fault may be the cause of the high yield encountered at L3. (discussed in a later section)

A survey of the fractures in the area (figure 3) indicated two major sets striking parallel and perpendicular to the bedding. Fractures parallel to cleavage or bedding were steeply dipping ($> 50^{\circ}$) in both directions. The fractures striking northwest perpendicular to the bedding showed a very strong northeasterly dip trend. This may be related to either the faulting or the plunging of the fold axis.

Surface Water Hydrology

Little Salmon River

This river discharges from the Lake Major Watershed into Cole Harbour Estuary (figure 1). The flow is regulated by a dam, because Lake Major serves as the water supply for the City of Dartmouth.

The mean annual flow of this river (Task Group, 1971) is 4.7×10^7 gpd. Montreal Engineering (1979) suggest a mean discharge of 2,634 $m^3/day/km^2$ (by comparison to the gauged Musquodoboit, Beaver, East and Shubenacadie Rivers). This works out to a mean long term flow of 4.6×10^7 gpd at Cole Harbour (drainage area 78.5 km^2), which is comparable to the Task Group result. Pol (1979) suggests a mean annual discharge in the order of 2.8 cfs/mi^2 for this area, which works out to 4.6×10^7 gpd for Salmon River at Cole Harbour, thus supporting the above. Mean monthly streamflows from the Musquodoboit, Beaver, East and Shubenacadie Rivers are: (in $m^3/day/km^2$):

May	June	July	August	September	October
3,253	1,603	898	917	912	2,068

The bed of Little Salmon River is predominantly bedrock and boulders, with channel direction being controlled mainly by bedrock structure.

Cole Harbour Estuary

(i) History

An estuary can be defined as a semi-enclosed coastal body of water having a free connection with the open sea and containing a measurable quantity of sea salt (Barnes). The tidal range supplies the energy to mix the fresh and salt water, with energy present being directly proportional to the square of the tidal range. The size and shape of the estuary, as well as volume of fresh water inflow, are important in determining the degree of mixing.

Cole Harbour is a large shallow estuary which opens to the ocean through a narrow channel to a shallow bay guarded by offshore bars. The estuary can be divided into essentially three parts; the west ear, east ear, and lower harbour, with the CNR track essentially bisecting the estuary (see figure 1). There are two trestles between the west ear and lower harbour and two between the east ear and lower harbour.

The outlet and flow characteristics of the estuary have been altered by sand and gravel extraction operations on the beach near Rainbow Haven. The causeway across what is now the outlet was used for trucks carrying aggregate. The former outlet was a narrow channel along the shore near Rainbow Haven. Removal of the causeway and deposition of sand resulted in closure of the main outlet and formation of Rainbow Haven Beach, which has a high recreation potential. Closure of the former tidal scour channel has resulted in the formation of a lagoon with a narrow outlet to the north into the lower harbour, and also in lengthening the course of water flow into and out of Cole Harbour.

Construction of the CNR causeway has reduced water flow in the west and east ears despite the trestle bridges provided. Until 15-20 years ago, sand and gravel operations on the flanks of the Little Salmon River, about one mile above Lawrencetown, resulted in rapid sedimentation in the western ear due to hydraulic washings. At present, the only activity appears to be a rock crushing operation using blasting since most of the unconsolidated aggregate has been removed. Little sediment is generated from the operation.

The estuary is in an advanced stage of eutrophication, and it can be expected that the development of salt marsh and tidal islands will continue at an accelerating rate. In turn, the volume of salt water will decrease, salinity will decrease, and eventually a fresh water marsh and meadow plant community will develop with the Little Salmon River as the principal fresh water source.

The rate at which this will occur is not known, but depends on sedimentation rate and amount of nutrients added from external sources. The above discussion has been summarized mainly from the Task Group, (1971).

(ii) Physical Characteristics

The estuary has a total area of about 2,470 acres at mean high tide and 740 acres at mean low tide.

The approximate surface areas of the three sections at high water are:

West Ear 45×10^6 ft^2

East Ear 20×10^6 ft^2

Lower Harbour 40×10^6 ft^2

(Total 2,410 acres)

The difference in area between mean high and low tide is about 1,730 acres. The western ear has an average depth of two feet at low tide. The volume of water exchanged in this ear is about 115×10^6 ft^3 outflow during a $7\frac{1}{2}$ hour period. Since the volume of water in the ear during low water is $95\text{-}100 \times 10^6$ ft^3, it is evident that the tidal flow is roughly equal to the low water volume. However, this can double under the correct meteorological conditions.

Dye testing in the western ear indicated sluggish flushing action, i.e. several tidal cycles are required for substances to be cleared from Cole Harbour. (Task Group, 1971).

The estimated mean annual flows for various inflows into the estuary (Table 1) total 64.3×10^6 gpd. The Little Salmon River is the most important inflow, making up 90% of the total fresh water discharge into the Western Ear, but only about 4.0% of the total volume of water stored in the Ear.

Based on table 1, stream discharge represents about 70% of total precipitation, groundwater discharge represents 10% of precipitation or 14% of streamflow, while the remaining 30% of precipitation is predominantly evapotransiration.

Groundwater Hydrology

Introduction

Porosity in fractured rock consists of: (a) intergranular, as void spaces between mineral grains, (b) vesicular, due to weathering, (c) fracture, referring to any discontinuity in the rocks, and including joints, faults, vugs, etc. (Streltsova, 1976).

Groundwater movement is almost entirely confined to fractures which provide the flow channels, while the storage of fluid is associated with the intergranular rock porosity. Rofail (1965) assumes that fractured rock consists of separated blocks which are considered as a porous medium, hence there are two flow systems (fractures and blocks), each flow with its own specific pressure.

Marine (1967) found two types of water-bearing openings-minute, and definite fracture zones. The minute openings pervaded the entire rock mass, and given enough time, the entire rock mass acts as a hydraulic unit. In this way, "dry" holes slowly fill with water to the static water level in that section of rock. The response is delayed and protracted compared to the larger openings. Many of these fine fractures occur at inhomogeneities in the rock, such as chlorite seams, contact of gneiss and schist, or along healed fractures.

The most active fracture flow is in the upper part of the bedrock where weathering activities have enhanced natural openings. Many authors suggest that fracture porosity and frequency are negligible in fractured rock below a depth of about 300 to 350 feet.

Lawrencetown Testholes

Figures 4 a, b, and c include the lithologic, E-log, penetration rate, and completion data for the testholes.

(i) Lithology

The lithologic data indicate a sequence of quartzite and slate, with some mixed zones. Quartz and minor calcite veins and stringers are common. A large number of fractures were intersected, but not all appeared to be water-bearing. Holes L2 and L4, which encountered no major fracture zones, were "dry" upon completion of drilling, but filled up slowly from minor fractures, as mentioned in Marine (1967). Holes L1 and L3 encountered major fracture zones. These zones corresponded mainly to changes in lithology, especially at the intersection of slaty beds with quartz vein material. This is attributed to differing competence of the units during rock deformation.

(ii) Penetration Rate (also referred to as drilling-Time log)

The penetration rate in L1 and L2 generally ranged from 0.5-1.0 feet per minute with the higher values being associated with the fractured zones. Hole L-3 illustrates a good correlation of penetration rate with lithogic data.

The rate in the upper 50 feet, which showed some weathering influence, was irregular but relatively rapid. The intermediate section of the hole, from 55-130 feet, showed a relatively slow rate, with little or no influence of the fractures on the penetration rate. As the major fracture zone between 160-165 feet was approached, the rate increased and became more irregular. In hole L4, the faster rates corresponded to the upper weathered zone and the minor fracture at 40 feet.

This information illustrates that a systematic record of penetration rate is useful in fractured rock, keeping in mind the various influencing factors discussed in the Drilling Program section.

(iii) E-log Data

Resistivity and spontaneous potential logs were run after completion of the wells. The resistivity of the formation reflects the resistance of the formation to the passage of electrical current.

Generally, dry or unfractured crystalline rocks indicate high resistivity values. The same unit with different chemical quality of formation water will show a resistivity variation inverse to the total dissolved solids content of the water.

The spontaneous potential (S.P.) curve reflects the naturally occurring potential difference between a surface electrode and an electrode in a column of conductive fluid (drilling mud or formation water) at any particular depth. Thus down-hole potentials are the results of currents of electro-chemical origin, eg. at the contact of the borehole water and water entering at a particular fracture where the qualities differ, and across finer grained layers above or below coarser layers. S.P. remains relatively constant against shales, with negative excursions across non-shaley beds.

Hole L1 shows a low resistivity (ρ) in the 140-150 and 170-185 foot zones, which both contain major fractures. The lower ρ at at 140-150 feet may indicate a more saline formation water here than at 170-185 feet. The increase at the bottom of the hole reflects unfractured rock. The S.P. is relatively featureless, but does reflect increasing negative valves corresponding to unfractured rock.

Hole L2 exhibits a low ρ and high S.P. at the 20-25 foot level, likely reflecting the influence of the bottom of the casing. The relatively constant S.P. from 50-75 feet suggests no major fractures but rather a slow seepage throughout the entire zone, as postulated previously. The S.P. is featureless.

Hole L3 indicates a low ρ corresponding to the major fracture zone. The ρ then increases steadily as the rock becomes unfractured, corresponding to an increase in negative S.P. valves. The variation in S.P. in this log is likely due to the rapid changes in lithology observed.

Hole L4 indicates a low ρ value at about 47 feet. Since no change in lithology was observed here, it is possible that this marks the inflow of slightly saline water into the well. The S.P. variation may be due to weathered bedrock influence in the upper 40 feet. Below 55 feet, ρ increases and S.P. decreases, indicating essentially unfractured rock.

These E-log results indicate the following:

(1) E-logging is a good back up tool, but would be difficult to interpret without the lithologic and fracture data in this type of bedrock.

(2) Many dry fractures, which could be selected for future stimulation of well yield, are not reflected by the E-log data.

(3) Increasing ρ values corresponding to increasing negative S.P. tend to reflect unfractured bedrock.

(4) The data from L2 and L4 suggest that there is little or no permeability below 50 to 55 feet, the limit of weathering, except in major fracture zones.

(iv) Aquifer Properties

(a) Transmissivity and permeability:

These were determined by pump test and slug test data. For the underlying assumptions of the various test methods the reader is referred to Kruseman and de Ridder (1970), Hvorslev (1951), Cooper et al (1967), and Bouwer and Rice (1976).

The Cooper method was attempted for L3 but the results were considered unreliable due to extremely poor match of the type curves. Marine (1967), however, successfully used the Cooper method to analyze the results of packer tests by swabbing.

The raw data and plots for wells L1 to L4 inclusive are contained in a appendices five to eight. Table 2 summarizes the results. In general, the T value of the rock matrix, as reflected by L1, L2, and L4 is in the order of 10.0 igpd/ft, while the T value in the fractured zone is in the order of 300 igpd/ft.

The difference in K values from pump and slug test data in L2 reflects the very long time lag which was extrapolated in the slug test. The pump test value is likely more reasonable. The slug test K value in L3 is felt to be more reasonable then the pump test value as a relection of fracture permeability, since the pump test data is more affected by limited aquifer boundaries. The K data suggests values in the order of 10^{-5} cm/sec for the matrix, and 10^{-3} cm/sec in fractured zones. These K values are apparent only since the saturated thickness is taken as the length of uncased hole. For example, if the slug test value of $K=2.0\times10^{-3}$ cm/sec (35.3 igpd/ft^2) for L3 is taken,

$$\text{effective thickness} = \frac{\text{average T}}{\text{average K}} = \frac{282.5}{35.3} = 8 \text{ feet,}$$

which represents only 28% of the open section. Generally it is not possible to define a specific length of the open hole through which the water is entering.

Examination of the pump test plots in appendices five to eight indicate a very definite break in slope of the drawdown data. This could be due to three factors (Uhl, 1978): (1) non-Darcian flow in the vicinity of the pumped well, since hydraulic conductivity is greatest in the predominant direction of fracturing and jointing. Generally, recovery T values were higher since turbulence is less and flow becomes more Darcian.

(2) insufficient pumping time for steady state conditions to be reached.

(3) Dewatering. With drawdown, the number of fractures contributing water

is decreased by dewatering, resulting in increased turbulence in the aquifer near the well face, increase in well loss and increase in slope of the plot, especially with increasing pumping rate. This is well reflected in table 2 by L3 data, where T decreases as Q increases. Generally the limited aquifer effect on slope is not as pronounced as that of dewatering (Uhl, 1978); for example, in L1, Q=2 igpm, the break in slope at 5 mins likely reflects a limited aquifer, while that at 20 mins reflects dewatering. In L3, Q=8 igpm, the break at 15 mins likely reflects a limited aquifer. Dewatering or limited aquifer effects occur at the following depths: L1 below 45 feet, L1 below 35 feet, L3 below 160 feet, and L4 below 28 feet.

(b) Storage coefficient and specific yield

Storage coefficient calculations for pumping well data are not reliable due to the uncertainty in r value, and the effect of well losses. However, some calculations were attempted using r = 0.25 feet (table 3). The results are widely scattered, and the pumped well results for L3 are unreasonable.

Specific yield was estimated by: (1) comparing hydrograph increase to precipitation data. (2) Sloto's recharge method. Hydrographs from June 1979 were used (figure 5). Data from L3 gave Sy=0.17, indicating a porosity in the order of 17% (assuming specific retention on fracture surfaces negligible). For one storm in July, 1979, Sy = 0.08 was obtained. This value is more comparable to that in the literature. For further calculations a porosity of 0.10 will be assumed.

The recession rate was remarkably constant in all three observation wells for June-July, 1979, with an average of 0.07 feet/day.

(c) Barometric and tidal efficiencies

Changes in atmospheric pressure produce sizable fluctuations in wells penetrating confined aquifers because aquifers are elastic bodies. The relationship is inverse. High barometric efficiency (BE) is usually associated with thick impermeable confining strata and is related to the storage coefficient. In a confined system, the pressure imbalance created by a change in barometric pressure is instantaneously transmitted without attenuation through the confining bed to the confining bed/aquifer interface. Hence, the fluctuations in water-level are in phase with and are a constant fraction of the barometric changes (Weeks, 1979). Barometric produced water level fluctuations also occur in unconfined water table aquifers because the pressure change is instantaneous in the well, but in the soil there is a temporary imbalance/time lag in the movement of air to the water table (Weeks, 1979) - these fluctuations are usually small.

In coastal aquifers in contact with the ocean, sinusoidal fluctuations of groundwater levels occur in response to tides. The relationship is direct. Tidal efficiency (TE) is also a measure of the incompetence of overlying confining beds to resist pressure changes. Tidal response is generally negligible unless porosity is low and the aquifer thick (Robinson etal, 1971, and Bredehoft, 1967). Evidence for tidal response is (Todd, 1959 b): (1) Two daily cycles of fluctuations, about 50 minutes later each day like the moon, (2) average daily retardation of cycles agrees with that of the moon's transit, (3) daily troughs of the water level coincide with transits of the moon at upper and lower culmination, (4) periods of large regular fluctuations coincide with periods of new and full moon, when tide producing forces are greatest (sun and moon work in the same direction). For further information on tides, please refer to Dohler (date unknown).

In the transitional, first and third quarter periods, the sun and moon do not have the same coordination, and the groundwater curve shows two maxima in 24 hours, but the amplitude is irregular and smaller (Nilsson, 1968). Tidal fluctuations are not generally observed in unconfined aquifers.

These responses of well aquifer systems are essentially equivalent to a seismograph (Cooper etal, 1965), since the well-aquifer system consists of a mass (the column of water in the well and some part of the water in the aquifer), a restoring force (the difference between the pressure head in the aquifer and the displaced water level in the well), and a damping force (the friction that accompanies the flow of water through the well and aquifer). The seismic disturbance is any wave that produces dilatation of the aquifer or vertical vibration of the well aquifer system (eg., ocean tides).

Equations relating BE and TE to storage coefficient and the modulus of elasticity of the aquifer are presented in Todd (1959 b), Domenico (1972), Sunada and McWhorter (1971), and Walton (1970). An important relationship is that BE + TE = 1.

Figure 5 illustrates water level data from L-1, L-2 and L-3 (L-1 and L-3 have identical hydrographs) for a one month period, along with barometric pressure (courtesy AES) and tidal data (Tide Tables, Halifax).

Calculations of BE by a method outlined in Clark (1967), as well as direct comparison of graphs, gave BE values of 0.35-0.80. Since BE is a constant factor, the variation may be related to effects of recharge and/for evaporation on the water level. Maximum tidal range in the aquifer during periods of essentially constant barometric pressure was 0.25 feet. The mean tidal range at Halifax is 4.7 feet. This would correspond to a TE of only 0.05, which was felt to be too low. The tidal range in the estuary is thusdamped out by the restricted access. An attempt to quantify the estuary range was made on January 18, 1980, when a tidal range of 6.4 feet was predicted for Halifax. (Tide Tables). The observed maximum difference in the estuary was only 1.3 feet, but unfortunately the time lag in the estuary was not known, so this value represents a minimum, ie, the estuary fluctuation is at least 20% of the tide table value. Therefore, minimum

range in estuary = 20% (4.7)
= 0.94 feet

$$\text{Maximum mean TE} = \frac{0.25}{0.94} = 0.27$$

Although the values above are approximate, for further calculations TE is taken as 0.25 and BE as 0.75.

θ = porosity = 0.10 (assumed to equal specific yield)

Ew = bulk modulus of elasticity of water = 300,000 psi

$$\frac{1}{Ew} = \beta = 3.3 \times 10^{-6}\ \text{in}^2/\text{lb} = 2.3 \times 10^{-8}\ \text{ft}^2/\text{lb}.$$

Es = bulk modulus of elasticity of rock.

$$BE = \frac{\theta Es}{\theta Es + Ew}$$

Therefore Es = 9.0×10^{6} psi. = 1.1×10^{-7} in/lb.

To estimate the effective saturated thickness of aquifer:

$$m = \frac{S \times BE}{\gamma \theta \beta} = \frac{10^{-4} \times 0.75}{62.4 \times 0.10 \times 2.3 \times 10^{-8}}$$
$$= 520 \text{ feet}$$

This value will be highly dependent on the S value. For S = 10^{-5}, m = 52 feet.

$$\text{Specific storage } (S_s) = \frac{10^{-4}}{520} = 2.0 \times 10^{-7} \text{ ft.}^{-1}$$
$$= 6.6 \times 10^{-9} \text{ cm}^{-1}$$

Checking S_s by another formula:

$$Ss = \frac{Pg\ \theta}{E\omega(\beta\varepsilon)} = \frac{1 \times 978 \times 0.10}{2.1 \times 10^{10} \times 0.75}$$
$$= 6.2 \times 10^{-9} \text{ cm}^{-1}$$

Marine (1967) found S_s to be in the order of 10^{-8} cm^{-1} for his wells in fractured gneisses and schists, hence the above calculation of $6.4 \times 10^{-9} cm^{-1}$ is felt to be reasonable.

To estimate a value of E_s, TE was used as follows:

$$TE = \frac{Es/\theta B}{S/\gamma\theta\beta m}$$

$$\frac{S}{\gamma\theta\beta m} = \frac{2.0 \times 10^{-7}}{62.4\ (0.10)\ (2.3 \times 10^{-8})} = 1.34$$

Therefore $Es = (1.34)\ (0.25)\ (0.10)\ (3.3 \times 10^{-6})$

$= 1.1 \times 10^{-7}\ in^2/lb.$ This value is reasonable as compared to the values of 1.7 to $2.5 \times 10^{-7}\ in^2/lb.$ (Gale, pers. comm.) for granite, and also to the value calculated from BE.

To give some idea of the competence of the strata, the factor "f" can be found (Sunada and McWhorter, 1971), where $0 \leq f \leq 1$:

$$\frac{\beta}{\alpha+\beta} = \frac{\beta}{Ss/\rho g\theta} = \frac{4.8 \times 10^{-10}\ cm^2/dyne}{6.6 \times 10^{-9} / (1 \times 980 \times 0.10)}$$

$$= 7.2$$

$BE = f - 1 - f\ (7.2)$
$BE = 0.75$

Therefore $f = 0.28$, which means that about 28% of the load is transmitted to the aquifer, ie., the aquifer is rigid. A value of $f = 1$ represents a thin soft confining strata where the entire load is transmitted, while a value of $f = 0$ is approached when the overlying strata are thick, rigid formations that transmit only a small part of the increased load to the top of the confined aquifer. This is consistent with the fact that the aquifer is a rigid competent quartzite/slate sequence.

It was also attempted to calculate TE from an equation by Ferris et al (1962) to check on previous calculations. The range of groundwater fluctuation in an observation well a distance x from the aquifer contact with the surface water body, whose stage is changing sinusoidally, is:

$$S_r = 2S_0\left[e^{-4.8x\sqrt{S/toT}}\right]$$

Sr = range of groundwater stage, feet

2So = range of surface water stage, feet

x = distance from well to surface water/aquifer contact, feet.

to = period of stage fluctuation, days, = 0.53

S = storage coefficient

T - Transmissivity, igpd/ft

$$\frac{Sr}{2S_o} = TE$$

Therefore at L1, x = 50 feet, T = 10 igpd/ft

TE theoretical = 0.35

at L3, x = 150 feet, T= 300 igpd/ft

TE theoretical = 0.64

These values are too high; the most likely explanation is use of $S = 10^{-4}$ in the equations.

To estimate lagtime of occurrence of a given maximum or minimum in groundwater, $t\ell$: $T = 0.60\ t_o\ S\ (\frac{x}{t}\ \ell)^2$

at L1, x = 50, T = 10. $t\ell$ = 2.1 hours

at L3, x = 150, T = 300, $t\ell$ = 1.2 hours

Actually the lags correspond in the two hydrographs, so the lag time is likely in the order of 1.7 hours. The precision of the hydrograph data is not sufficient to verify this calculation.

In summary, although a precise measurement of barometric and tidal efficiencies was not possible, utilization of available data and checking in different equations indicates reasonable values of 0.75 for BE and 0.25 for TE. Other parameters estimated are a) effective saturated thickness of aquifer of 520 feet, b) specific storage 6.4×10^{-9} cm^{-1}, c) aquifer bulk modulus of elasticity 1.1×10^{-7} in^2/lb, and d) lagtime in groundwater stage about 1.7 hours related to surface water.

HYDROCHEMISTRY

Surface Water:

The chemistry of Little Salmon River, from one point in time, is contained in appendix 9. The water quality was good as compared to the Guidelines for Canadian Drinking Water Quality.

Previous studies (Gautreau etal, 1971) have shown that Cherry Brook, which is the main input into Little Salmon River below the Lake Major dam, shows some degree of pollution from discharge of sewage treatment plant effluent (Westphal Mobile Home Park). The plant at Ross Road School also discharges into the Little Salmon River. The degree of pollution varies widely, related to streamflow, precipitation, and the maintenance status of the treatment plants. Humber Park Subdivision discharges into Broom Creek and thus into the estuary also.

Appendix 9 contains quality data from the estuary from September 2, 1977. Figure 6 shows the sampling stations. Generally, the estuary is about 85% sea water as compared to "normal" sea water (Hem, 1970). Previous information from the estuary is contained in appendix 11. A trilinear plot (figure 7) indicates that the estuary samples all plot along a line joining rainwater and seawater, close to seawater. Hence there is no modification of the chemistry, just simple mixing (Piper, 1944).

Groundwater:

Appendices 9 and 10 contain the groundwater data and figures 7 and 8 the trilinear plots. Most of the Lawrencetown analyses fall within the Na and Cl fields of the cation and anion triangles. If the river and dug well are considered representative of shallow groundwater, then L1 (50 feet) and L2 samples represent a simple mixture of shallow groundwater and seawater (Piper, 1944).

The L1 and L3 groundwater (figure 8) show no simple mixing pattern. Sulphate is essentially zero, and reducing conditions exist (high pH, hydrogen sulphide odor) in the deeper groundwater, indicating modification. The Lawrencetown testhole chemical data is different from other wells showing salt-water intrusion in fractured bedrock, mainly in the cation facies: the Lawrencetown water is depleted in Ca and Mg relative to the other wells. The reason for this is not known, possibly more reaction (dissolution or ion exchange) with calcareous materials has occurred in the other wells. Difference in construction may also play a role - L1 and L3 have much longer casing than most of the other wells plotted.

Essentially no change in quality was observed during the pumping of the test-holes, but the duration was only one hour maximum. A pumptest of several hours duration would be required to determine changes with time.

DISCUSSION OF RESULTS AND IMPLICATIONS IN THE LAWRENCETOWN AREA

Water Quantity:

In saline aquifers, when using head data to calculate gradients, density of the formation water must be considered (Lusczynski, 1961). For the purposes of this report, $\rho f = 1.000$, and a ρs significantly different is considered as 1.001. A $\rho = 1.001$ corresponds to 1400 mg/l Na Cl or 850 mg/l Cl (appendix 3). Since none of the testholes exceed this level, no correction was made between wells for density differences. The density of the estuary was estimated at 1.018, using an average Na Cl content of 25,000 mg/l.

The ground levels between sites L1-L2 and L3-L4 differ by an average of 4.7 feet, with L1-L2 being 4.7 feet above mean sea level. Well's L1 and L2 are approximately 50 feet from mean sea level. The lowest water levels in the wells are approximately as follows:

L1 - 4.0 feet a s l

L2 - 0.5 feet a s l

L3 - 4.7 feet a s l

L4 - 1.9 feet a s l

Using the midpoints of the open holes as the point of pressure measurement, a flow net was constructed (figure 9) assuming $Kh/Kv = \frac{10}{250}$ (used by Lin, 1975 for fractured rock in the Smith's Cove area) and using the low water level measurements so the worst conditions were calculated. Flownet adjustment for permeability variation was done using the method in (Cedergren, 1977).

For a section of aquifer 150 feet deep and 150 feet back from the shoreline, and using $K = 0.20$ gpd/ft^2 (0.03 feet/day):

$$q = Kh \frac{nf}{nd} = 0.03 \text{ feet/day} \times 4.5 \times \frac{37}{9}$$

= 0.56 cuft/day/foot of shoreline

= 3.5 ig pd/foot of shoreline.

Since this value appeared to be unreasonably low, a check was made by the following two rough methods:

(a) Total shoreline length contributing to west ear ≈ 6.3 miles (Table 1).

Discharge along shoreline = groundwater recharged into the areas contributing directly into the ear = 4.6 km^2 (Assuming groundwater recharged in a specific drainage basin discharges as stream baseflow at one point only - the stream mouth.)

Total surface water inflow from direct runoff areas = 2.7×10^6 gpd (Table 1).

Groundwater component = 14% of this

= 378,000 gpd

= 11.3 gpd/foot of shoreline

(b) Assuming infiltration = 10% of P = 0.138 m, then direct groundwater discharge along shore = $\dfrac{.138 \text{ m} \times 4.6 \times 10^6 \text{ m}^2 \times 220}{365}$

= 383,000 igpd

= 11.5 gpd/foot of shoreline

The higher values obtained from (a) and (b) suggest that (1) either more area than 150 x 150 feet is recharging the shoreline, or (2) the mean discharge is in order of 11.4 gpd/foot while the low value could be 3.5 gpd/foot. The results do indicate that direct discharge to the sea comprises <1% of the total surface runoff from the entire drainage area of the west ear, as found by McGuinness (1963) for U.S. coastlines.

Two significant features of the flownet are (1) the apparent localization of flow at the 50 foot level, which corresponds roughly to the base of the weathered zone of the bedrock. (2) The strong vertical component of flow. This can be quantified as follows:

$$T \text{ matrix} = 10 \text{ igpd/feet} = \sqrt{TxTy}$$

Assuming $\frac{Tx}{Ty} = \frac{10}{250}$, then Tx = 2 igpd/feet, Ty = 50 igpd/feet

Vertical gradient 0.03 ft/ft

Horizontal gradient 0.01 ft/ft

Thus, L = 1 foot,

$$\begin{aligned} Q \text{ vertical} &= TIL \\ &= 50 \times 0.03 = 1.50 \text{ gpd.} \end{aligned}$$

$$Q \text{ horizontal} = 2 \times 0.01 = 0.02 \text{ gpd.}$$

Therefore ratio Q vertical/Q horizontal = 75/1, or, there is 75 times the quantity of water flowing in a vertical direction as a horizontal direction. This will tend to reduce horizontal seepage pressures and facilitate intrusion.

The theoretical width of the gap, X_0, through which fresh water discharges to the sea is $\frac{Q}{2\gamma K} = \frac{0.56}{2(0.018)\ (0.03)} = 509$ feet (where $\gamma = \frac{\rho s - \rho}{f} = 1.018 - 1.000 = 0.018$)

To give some idea of required well distance from shore, the equation for oceanic islands could be used: if we assumed groundwater recharge rate, W, is 6 inches/year = 0.0014 feet/day, R = radius of peninsula = 250 feet, h = depth to interface at distance r (desire h = 200 feet or more below sea level).

$$h^2 = \frac{W}{.0512K} (R^2 - r^2)$$ (see appendix 3 for derivations)

Then r = 150 feet, or the well must be located at least 100 feet (R-r) from the shoreline and must not exceed 200 feet in depth.

The recession rate of the hydrographs from the testholes can be used to roughly estimate storage depletion under natural conditions. If we assume a constant rate of 0.07 feet per day, and an average freshwater head of 5 feet, then 71 days would be required for storage depletion to sea level. Under present climatic conditions, such a prolonged dry period is unusual. However, in areas of lower head, such as 2 feet, only 28 days (1 month) with no recharge would be required under natural conditions. This may occur during the summer period. Intrusion would likely begin before freshwater head reached zero, since reversal of gradient is not required for intrusion. Lowering of head by pumping would decrease the time factor calculated above.

The calculations in this section illustrate the following:

(1) Direct fresh groundwater discharge along the shoreline is <1% of total surface runoff, with mean value in the order of 11.4 gpd /foot of shoreline and a possible low of 3.5 gpd/foot.

(2) High vertical permeability imposed by the fracture system results in about 75 times more water flowing vertically than horizontally, which reduces horizontal pressure to offset intrusion.

(3) The required distances and depths of wells from the shoreline can be predicted if permeability is known.

(4) The natural water level recession under prolonged dry periods may be sufficient to induce intrusion without the effects of pumping.

(5) The significance of the 50 foot boundary in the flownet is not clear, but may reflect change in K_h/K_v ratio due to weathering effects.

Water Quality:

The overall hydrochemistry of the Lawrencetown wells was discussed previously.

Two methods were attempted, using chloride ion, to use water quality data for intrusion predictions:

(a) A plot of isochlors (figure 10) was compared to the Ghyben-Herzberg line as in Kohout (1960) and Cooper et al (1964). From this plot, the chloride value of 200 mg/L appears to be significant in relation to the Ghyben-Herzberg line, the zero horizontal gradient line, and the depth of 50 to 60 feet.

(b) Using the method outlined in Vacher (1980):

$$Cl_r = 40 \text{ mg/L}$$

$$Cl_R = 4.5 \text{ mg/L}$$

$$R = 54 \text{ inches/year}$$

$$r = 6 \text{ inches/year}$$

$$\text{Therefore } \frac{r}{R} = 0.11$$

$$\frac{Cl_R}{Cl_r} = 0.11$$

For E = 50% (depth to interface), $0.5 = \frac{Cl - 40}{17000 - 40}$

$$Cl = 8{,}520 \text{ mg/L}$$

For E = 1%, the upper limit of the transition zone, Cl = 210 mg/L. Hence, using Vacher's method, the 200 mg/L isochlor in the Lawrencetown area can be taken as the upper limit of the zone of diffusion. This isochlor penetrates about 140 feet inland at the testhole site.

Using L = 140 feet to estimate m, the effective saturated thickness of aquifer:

$$q = \frac{1}{2} \left(\frac{\rho s - \rho^{f}}{\rho f} \right) K \frac{m^2}{L}$$

$$m = \sqrt{\frac{0.56(140)(2)}{(.018)(.03)}}$$

$= 540$ feet,

which is similar to the value of m of 520 feet calculated using barometric efficiency data.

Although not mentioned previously, intruded waters may cause undesirable aesthetic effects such as corrosion, since groundwater containing dissolved salts is more corrosive to household plumbing and to steel casing lining the wells. Fink (1963) states that the degree of corrosive attack on steel in saline well waters with pH 5 - 9^{+} is usually controlled by the rate of dissolved oxygen diffusion to the cathode. Factors to be considered are:

(1) oxygen,

(2) low pH (< 5), especially in same salt marsh areas,

(3) salt increases the rate of corrosion and because chloride ion is relatively small in size as compared to, say, sulphate, it can penetrate protective, films.

(4) CO_2 and H_2S,

(5) bacterial action,

(6) velocity, cavitation, erosion/corrosion and deposit.

Fink (1963) suggests that case hardening of metals (which is done for drive shoes) may reduce corrosion resistance in a saline environment. Possible protective measures include a sacrificial anode and removal of free O_2, CO_2, and H_2S - these are unfeasible in most domestic wells. Hence, to solve this problem, a noncorrosive type of casing, such as thermoplastic, would be more realistic. Another environmental effect of intrusion is the health and aesthetic aspect. The GCDWQ suggest a limit of 250 mg/l chloride, based on palatability. There is no guidelines for sodium, however, a limit of 20 mg/l has been suggested for persons on low salt diets, with medical problems such as fluid retention, hypertension, etc. There has been some mention recently of a relationship between high sodium and crib deaths of babies.

In summary, the water quality may be considered from two points of view:
a) use as a tracer or predictive tool in intrusion. In Lawrencetown, the 200 mg/l isochlor appears to represent the upper limit of the zone of diffusion.
b) environmental effects of intrusion, such as corrosion and health. These effects will vary with water type and the individual consuming the water.

Concept of Safe Yield in Coastal Systems

Kazmann (1956) defines "safe yield" as the annual extraction from a groundwater unit which will not or does not 1) exceed average annual recharge, 2) so lower the water table that the permissible cost of pumping is exceeded, and 3) so lower the water table as to permit intrusion of water of undesirable quality. Wentworth (1951) defines "safe yield" as a concern for the continuance of a good potable water supply, while "unsafe yield" is one which under any given set of conditions leads to water shortage, inadequate reserve, quality impairment, etc.

There is some difficulty in estimating "safe yield" because of the time lag between excessive draft and the slow depletion of large water bodies, related to the storage feature of the Ghyben-Herzberg lens (Wentworth, 1942). Kazmann (1956) also brings out the point that once "safe yield" is set, based on prepumping steady state flow in the aquifer, this amount might not be available for pumping indefinitely because 1) transmissivity and maximum hydraulic gradient might become too small to allow the necessary rate of flow through the producing structure, and 2) the quality may change.

The data requirements to determine safe yield or optimum pumping rate are:

1. Fresh water inflow. This can be determined by methods such as;

a) Darcy's Law: Q = KIA or Q = TIL

b) Total draft: assume pumping wells consume all groundwater outflow, and use head in observation wells to determine recharge.

c) Hydrologic balance equation for long term. This works best for confined aquifers with well defined recharge areas.

d) In areas of well-developed stream networks, groundwater outflow from the zone of relatively shallow aquifer to the sea can be estimated by calculating average long term groundwater discharge by separation of stream hydrographs times the corresponding area of the coastal zone (Zektzen etal, 1973).

e) Computer simulation

2. Type of management strategy, for example:

a) Dispersed abstraction to try and obtain a uniform lowering of the piezometric surface and hence uniform head deficiencies.

b) Protection of fresh water from decreased recharge, from interference, and from surface contamination.

c) Raising head by artificial recharge.

d) Whether or not it is possible to accept a certain level of saltwater intrusion to effectively utilize the available groundwater on an economic basis.

e) Decreasing usage.

In the Lawrencetown area, fresh water inflow was determined by the Darcy method and by the hydrologic balance equation assuming coincident groundwater and surface watershed boundaries and uniform annual recharge of 6 inches to the groundwater system. At present, there is no management strategy proposed or in operation in the area. In the author's estimation, the best options would be decreased usage, dispersed pumping centres, and drilling limitation both depth and with distance from shoreline.

The Montreal Engineering Study (1979) suggests that no minimum flow is required in the Little Salmon River. If this is the case, it is felt by the author that the intrusion problem would be aggravated in the Western Ear of the estuary. Hence some minimum flow should be assumed. The quantity required should be determined by someone familiar with estuarine mixing dynamics.

CONCLUSIONS

(1) Saltwater intrusion in the testhole area is occurring naturally because of (a) low freshwater discharge (low bulk transmissivity), (b) low horizontal pressure due to the high vertical permeability imposed by the fracture system.

(2) The fractured rock has a bulk matrix transmissivity in the order of 10 igpd/feet, but where significant fracture zones are present, this may increase considerably (300 igpd/feet).

(3) A freshwater head is present, ie., no reversal of gradient has occurred, even under low water level conditions. However, intrusion is still occurring.

(4) The western ear of the Cole Harbour Estuary is about 85% normal seawater. Total freshwater inflow is $<$ 10% of the total low water storage volume of the ear, and groundwater input is $<$ 1% of the estuary volume.

(5) Little Salmon River comprises about 90% of the freshwater inflow into the western ear, hence, the assumption by Montreal Engineering that no minimum flow is required in the river must be seriously considered and the impact on the estuary determined.

(6) The long term trends of the estuary are not known. Predictions are that it will become a freshwater meadow, but the effects of long term rise is sea level must be offset against this.

(7) Because of anisotropy of the bedrock and large variations in T and S, calculations of flow and recharge rates are difficult and a large number of assumptions are required. A regional budget method is probably best to smooth out inhomogeneities.

(8) Despite the complexity of fracture flow and direction, chloride data appear to offer some potential for definition of the interface. A value of 200mg/L appears to represent the top of the zone of diffusion.

(9) The concept of safe yield in this type of anisotropic system is difficult to apply. Significant factors are comprehensive evaluation, planning, and effective management of the coastal area.

(10) The only effective management apparent at this time is reduction or rearrangement of groundwater pumping, or limitation on drilling in the coastal area. The feasibility of recharge or induced filtration is beyond the scope of this report.

RECOMMENDATIONS

1. On a short term basis, the following solutions are suggested.

(a) Spread withdrawals over large areas, ie. discourage concentrated development in subdivisions with a groundwater source.

(b) Minimize withdrawals close to the estuary.

(c) Use available source wells at rates to give the smallest drawdown consistent with yields for the present demand.

2. On a long term basis, the following could be done.

(a) refine the water budget and try to develope empirical relations between annual precipitation, recharge, and safe yield.

(b) try to determine optimum well depths and distances from the shoreline.

(c) consider the use of plastic casing to minimize corrosion.

(d) carefully supervise proper abandonment of existing contaminated wells.

(e) improve management of future requests for exploration/abstraction such as subdivisions. Site specific information on lithology, aquifer properties, chemical quality, and yield, both on an areal and vertical basis, are necessary because of the anisotropic aquifer media. A minimum requirement for any development should be a three well array consisting of a pumping well and two observation wells aligned parallel and perpendicular to strike and equidistant from the pumping well. Consideration should also be given, where conditions permit, to a limited number of pumping centres or a surface source located some distance inland, thus minimizing pumping near the shore.

(f) delineate problem areas and declare moratorium or a permit type system for any further drilling in these areas.

(g) assess the potential of geophysics as tool for delineating intrusion areas in fractured rock.

3. Additional future work could include.

(a) assessment of decreasing Little Salmon River streamflow on the intrusion process and on the eutrophication of the estuary.

(b) refirement of time lag and tidal range in the estuary as compared to the Halifax tidal gauge station.

(c) continuance of water level recorders on L2 and L3 to monitor changes in aquifer storage outside of concentrated pumping areas.

(d) establishment of a quality monitoring program for the testholes and estuary.

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F I G U R E S

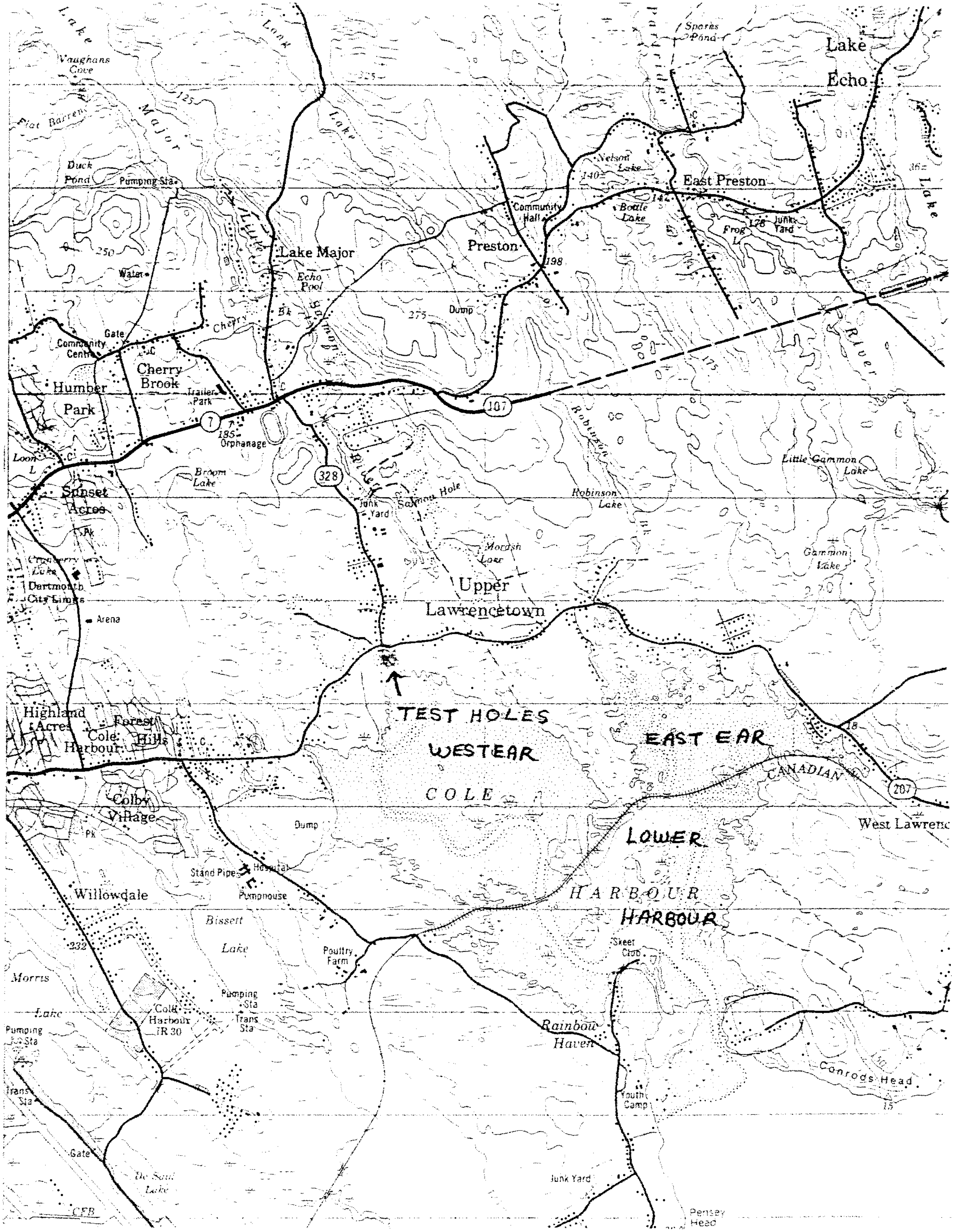
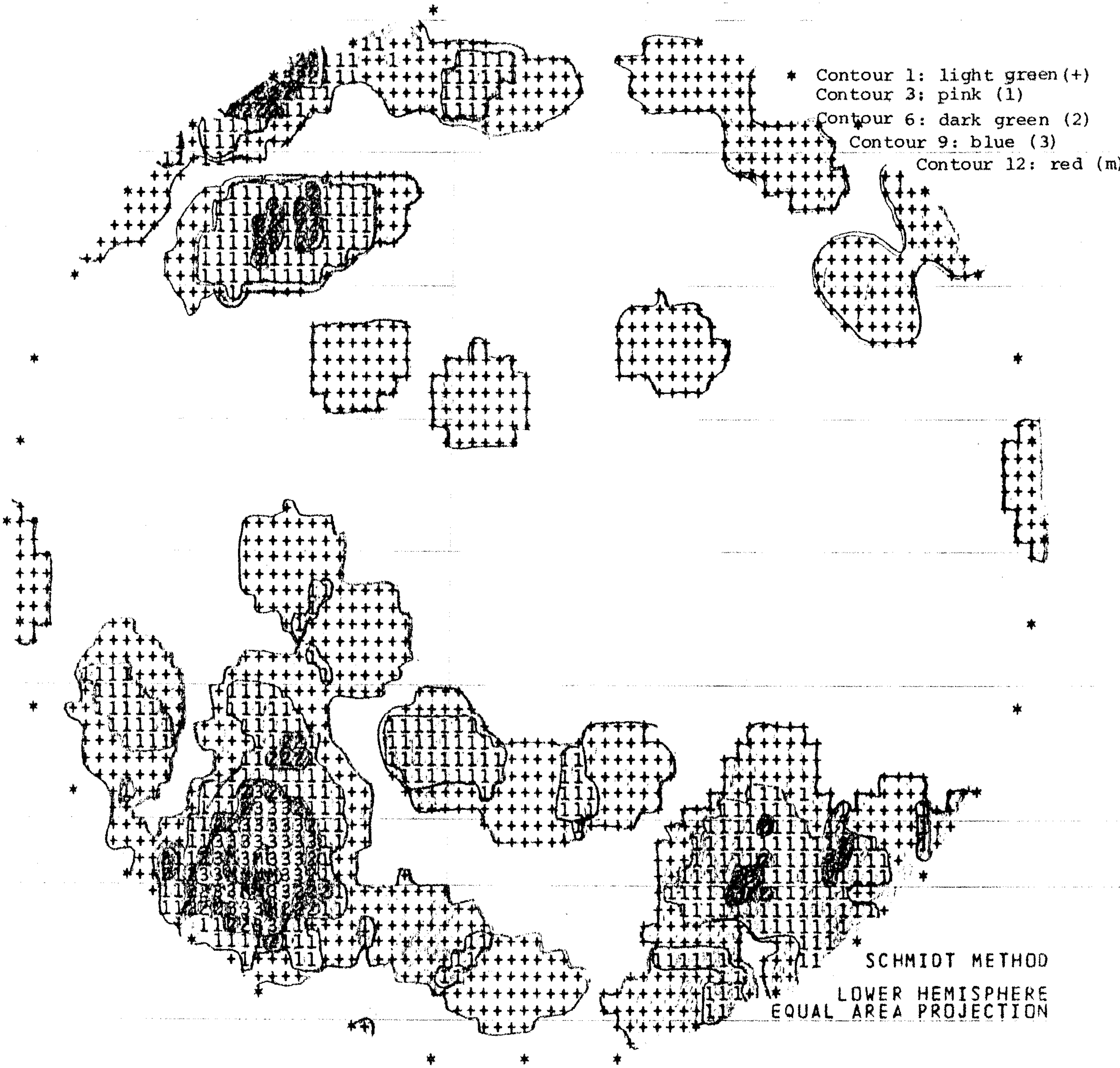


FIGURE 1: Testhole Locations
Scale: 1:50,000
NTS sheet 11D11

FIGURE 2: Geological Map
Scale: 1" - 1 mile

FIGURE 3



Contour 1: light green (+)
Contour 3: pink (1)
Contour 6: dark green (2)
Contour 9: blue (3)
Contour 12: red (m)
SCHMIDT METHOD
LOWER HEMISPHERE
EQUAL AREA PROJECTION

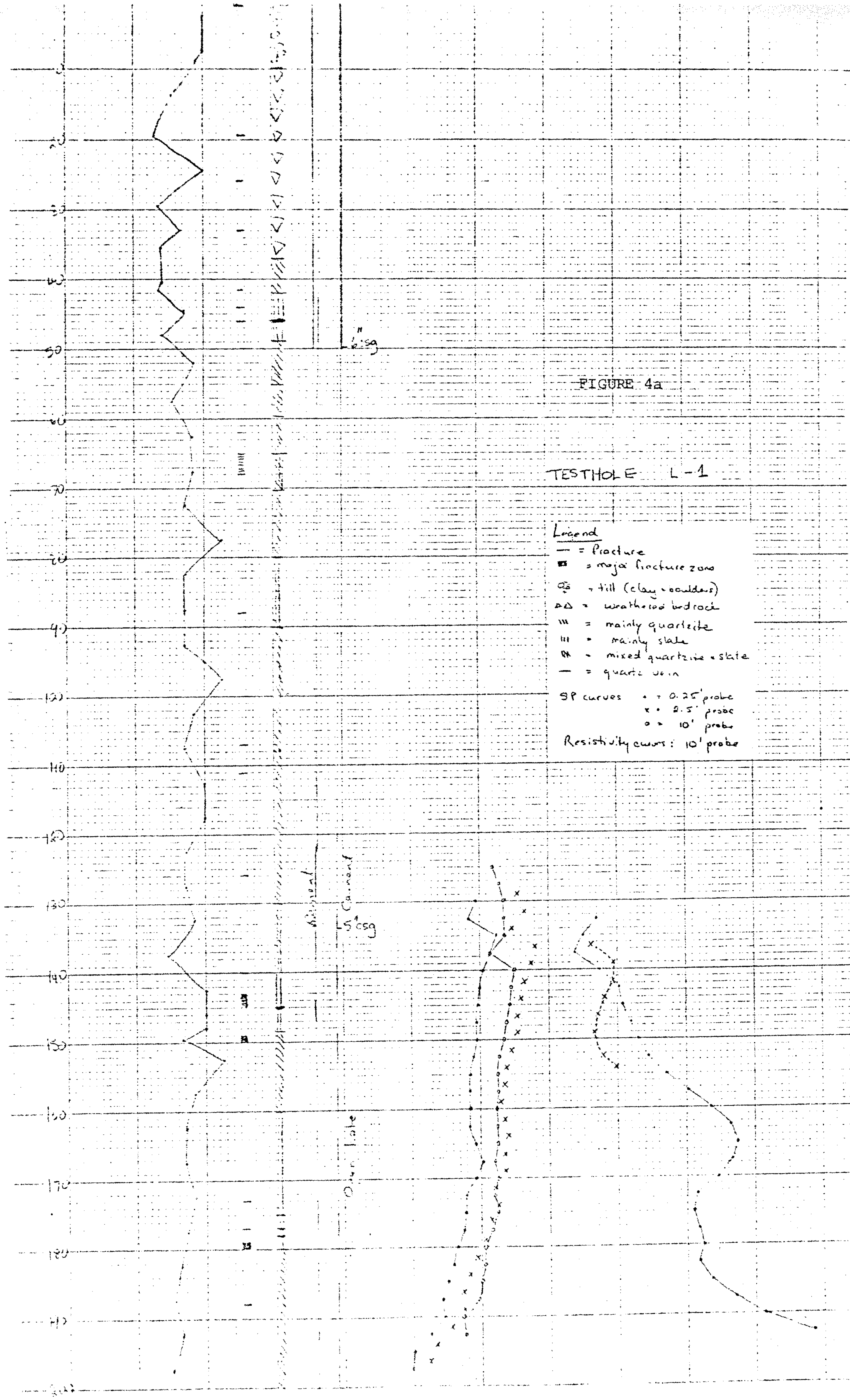
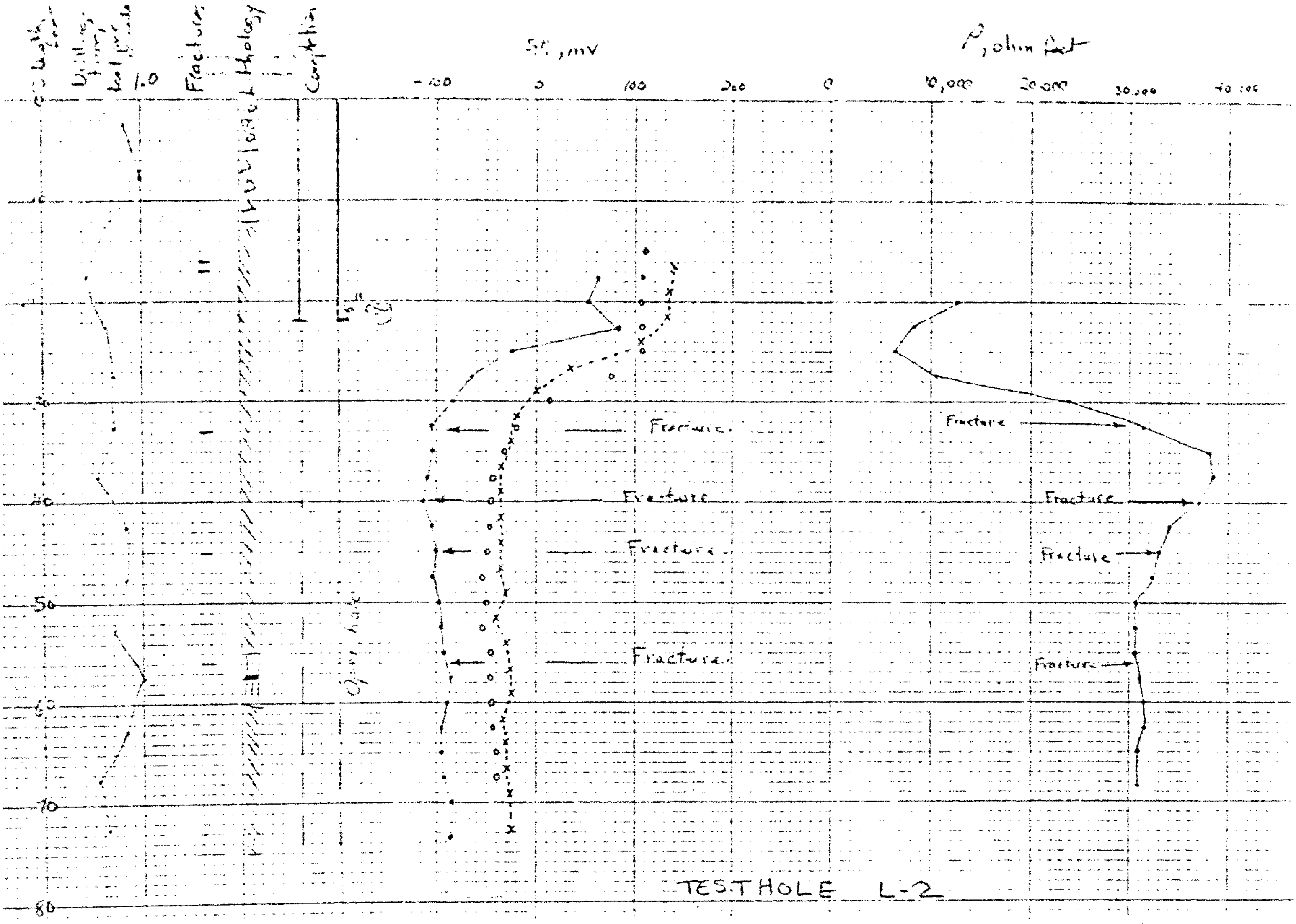
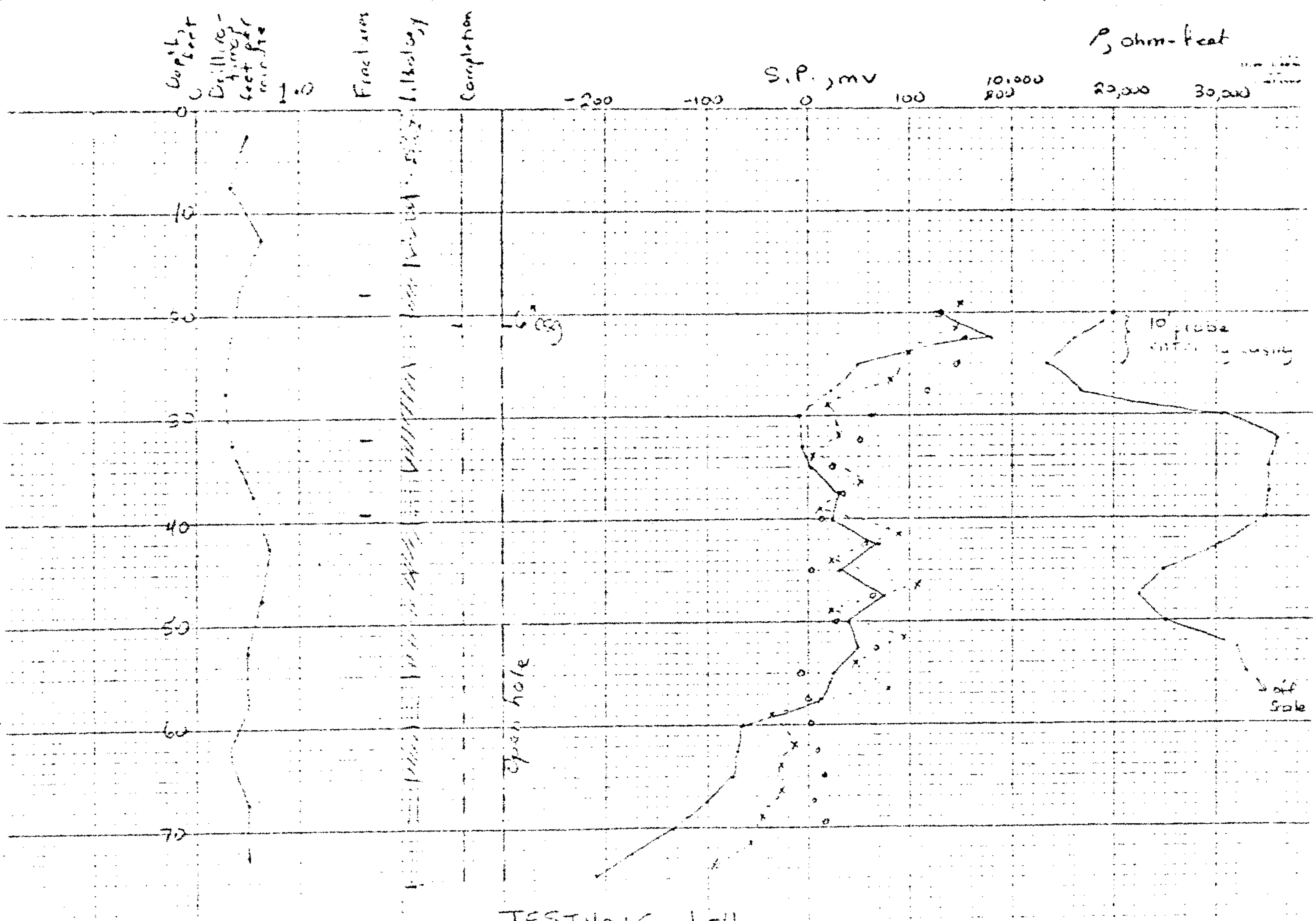

TESTHOLE L-1
Legend
= fracture
= major fracture zone
= till (clay + boulders)
= weathered bedrock
= mainly quartzite
= mainly slate
= mixed quartzite + slate
= quartz vein
SP curves
= 0.25' probe
= 2.5' probe
= 10' probe
Resistivity curves: 10' probe
6" csg
Cement
L5' csg
20
30
40
50
60
70
80
90
120
140
150
160
170
180

FIGURE 4a





Legend
— = Fracture
= major fracture zone
= till (clay, boulders)
ΔΔ = weathered bedrock
/// = mainly quartzite
||| = mainly slate
= mixed quartzite & slate
— = quartz vein

SP curves · = 0.25' probe
x = 2.5' probe
o = 10' probe
Resistivity curves: 10' probe

Figure 4c

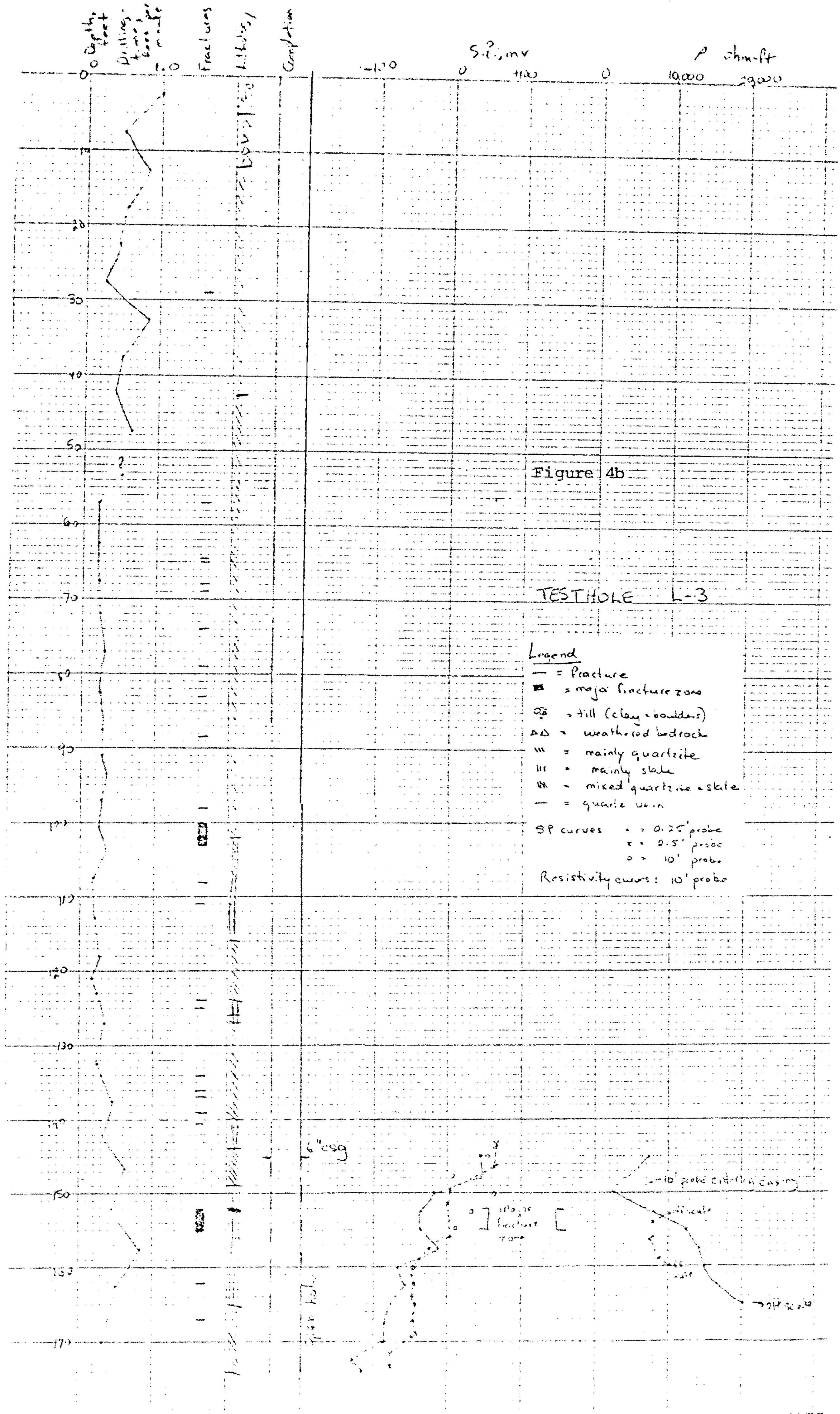
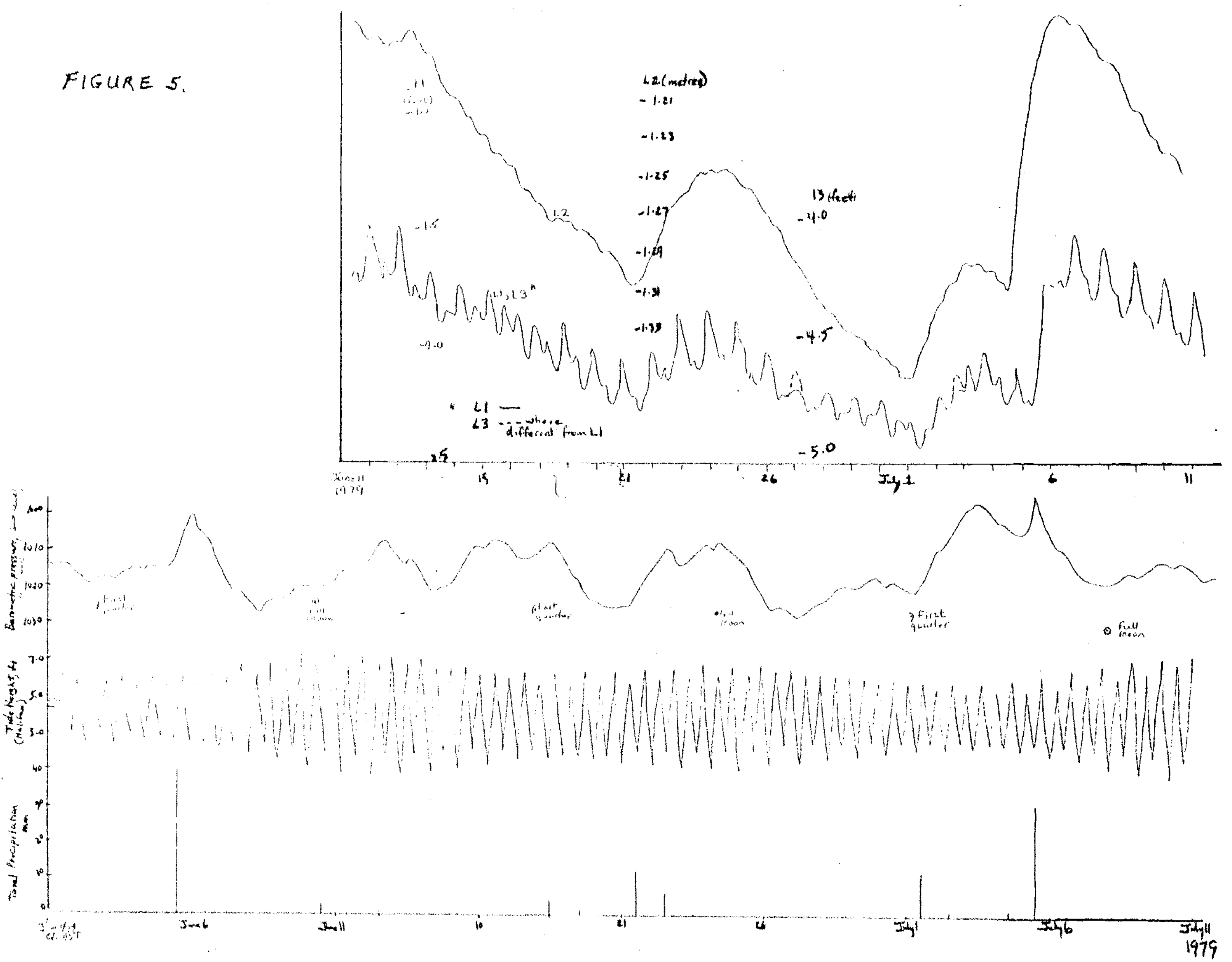

Depth, feet
Drilling time, feet per minute
Fractures
Lithology
Completion
S.P., mv
ρ ohm-ft
-100
0
+100
0
10,000
20,000
TESTHOLE L-3
Legend
— = Fracture
= major fracture zone
= till (clay + boulders)
ΔΔ = weathered bedrock
= mainly quartzite
= mainly slate
= mixed quartzite + slate
— = quartz vein
SP curves · = 0.25' probe
x = 2.5' probe
o = 10' probe
Resistivity curves: 10' probe
6" csg
major fracture zone
10' probe entering casing

Figure 4b

APPENDIX 11

Summer Water Chemistry and Flushing
Time - Cole Harbour, N.S.

FIGURE 5.


L2 (metres)
-1.21
-1.23
-1.25
-1.27
-1.29
-1.31
-1.33
L3 (feet)
-4.0
-4.5
-5.0
* L1 ——
L3 - - - where different from L1
L1, L3*
June 11 1979
15
21
26
July 1
6
11
Barometric pressure
Tide Height at (Halifax)
Total Precipitation mm
First quarter
Full moon
Last quarter
New moon
First quarter
Full moon
June 6
June 11
July 1
July 6
July 11 1979

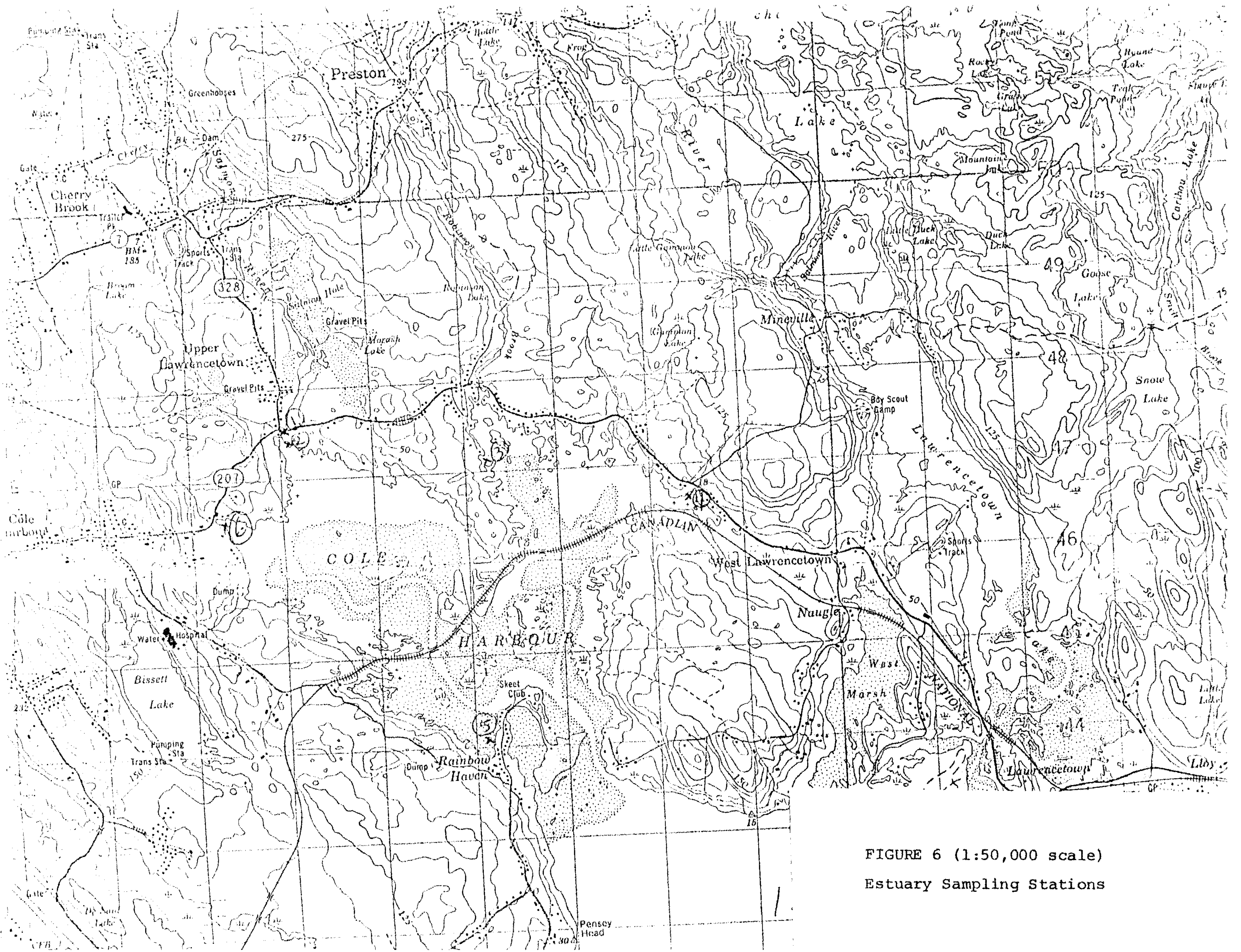


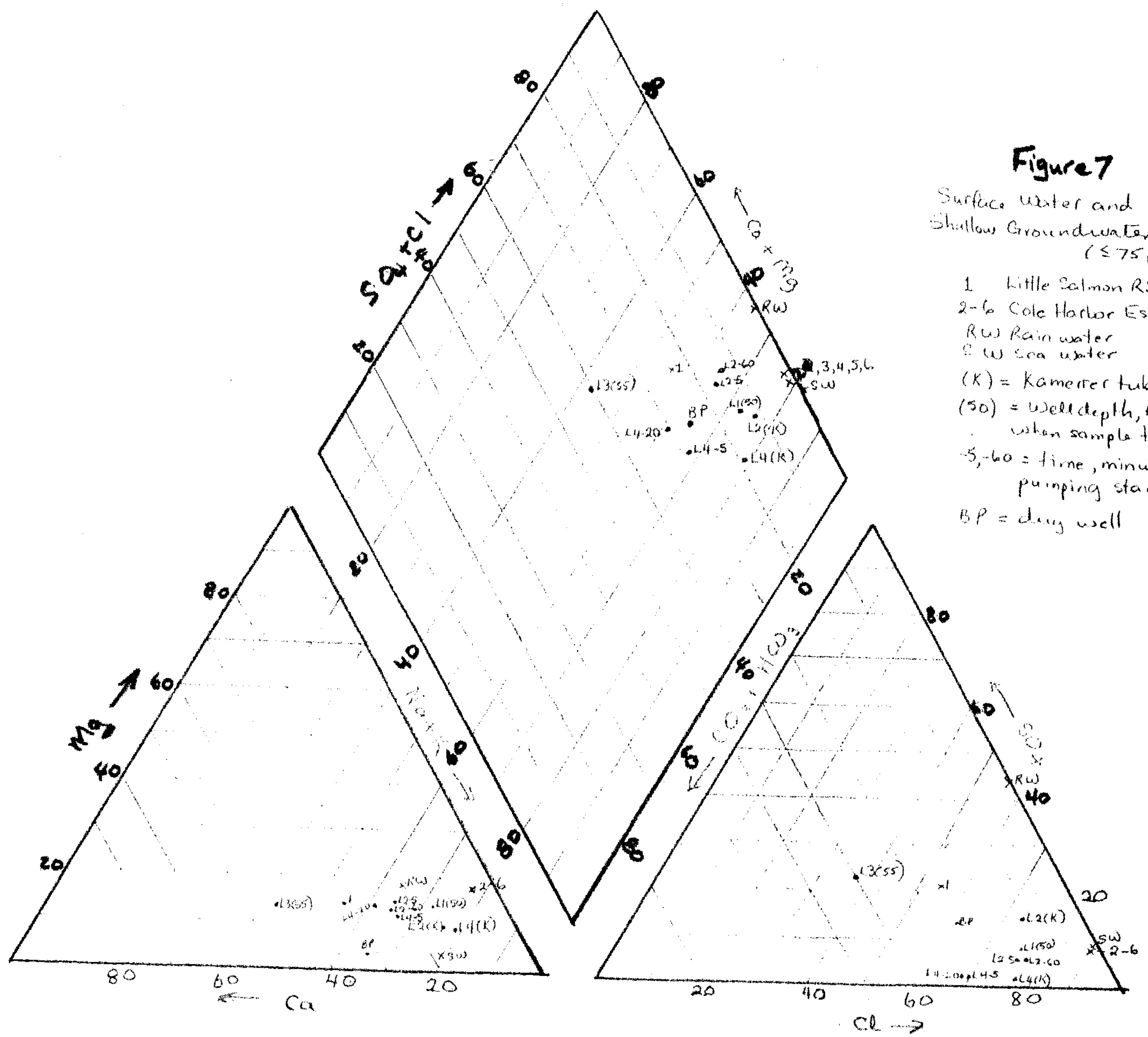
FIGURE 6 (1:50,000 scale)

Estuary Sampling Stations

Figure 7

Surface Water and
Shallow Groundwater
(≤75 feet)

1 Little Salmon River
2-6 Cole Harbor Estuary
RW Rain water
SW Sea water
(K) = Kamerrer tube sample
(50) = Well depth, feet, when sample taken
-5, -60 = time, minutes, after pumping started
BP = dug well



Percent of total equivalents per million

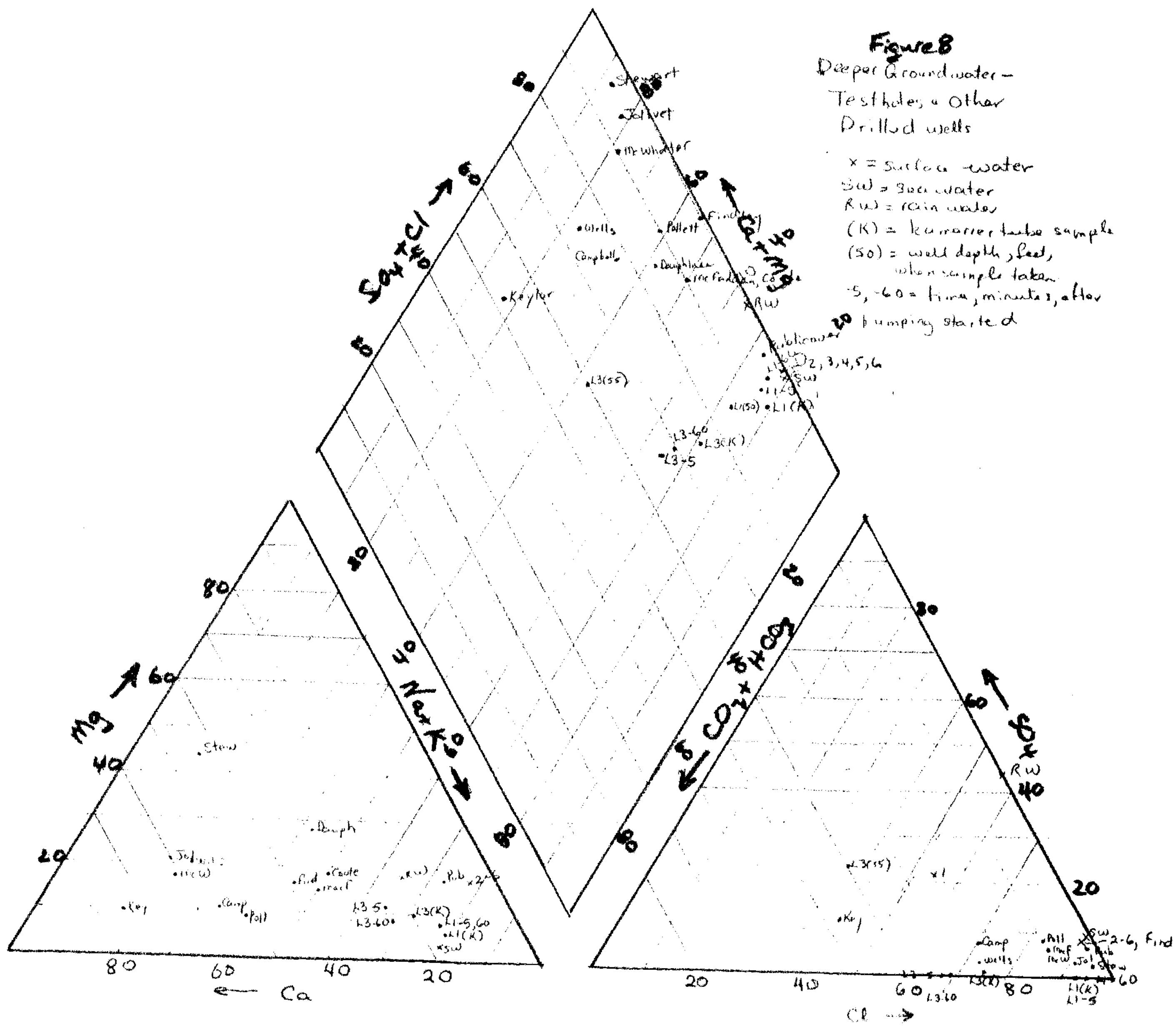


Figure 8
Deeper Groundwater –
Testholes & Other
Drilled wells

x = surface water
SW = sea water
RW = rain water
(K) = kamerer tube sample
(50) = well depth, feet, when sample taken
-5, -60 = time, minutes, after pumping started

Percent of total equivalent per million

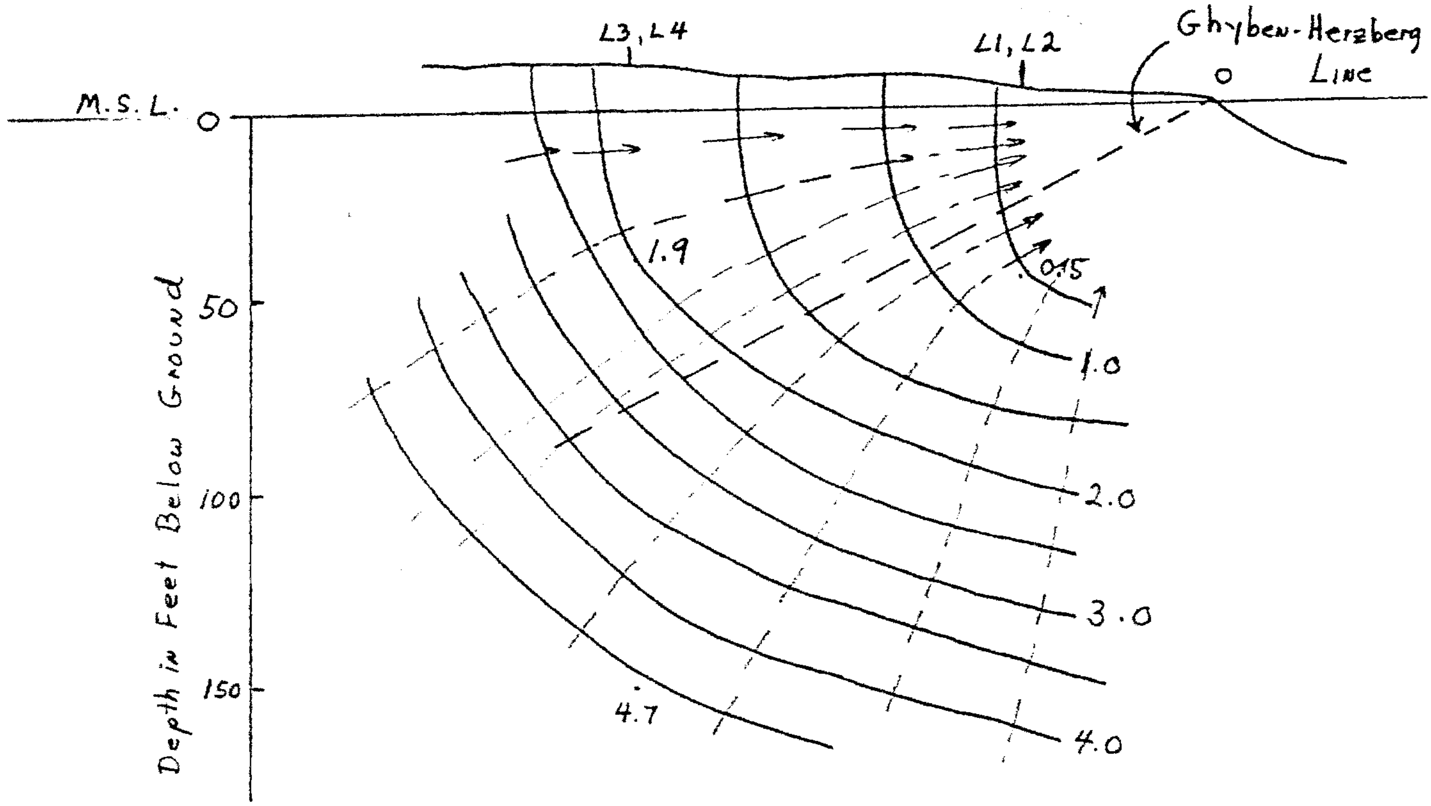


FIGURE 9: Flow net, 150 x 150 foot section of aquifer

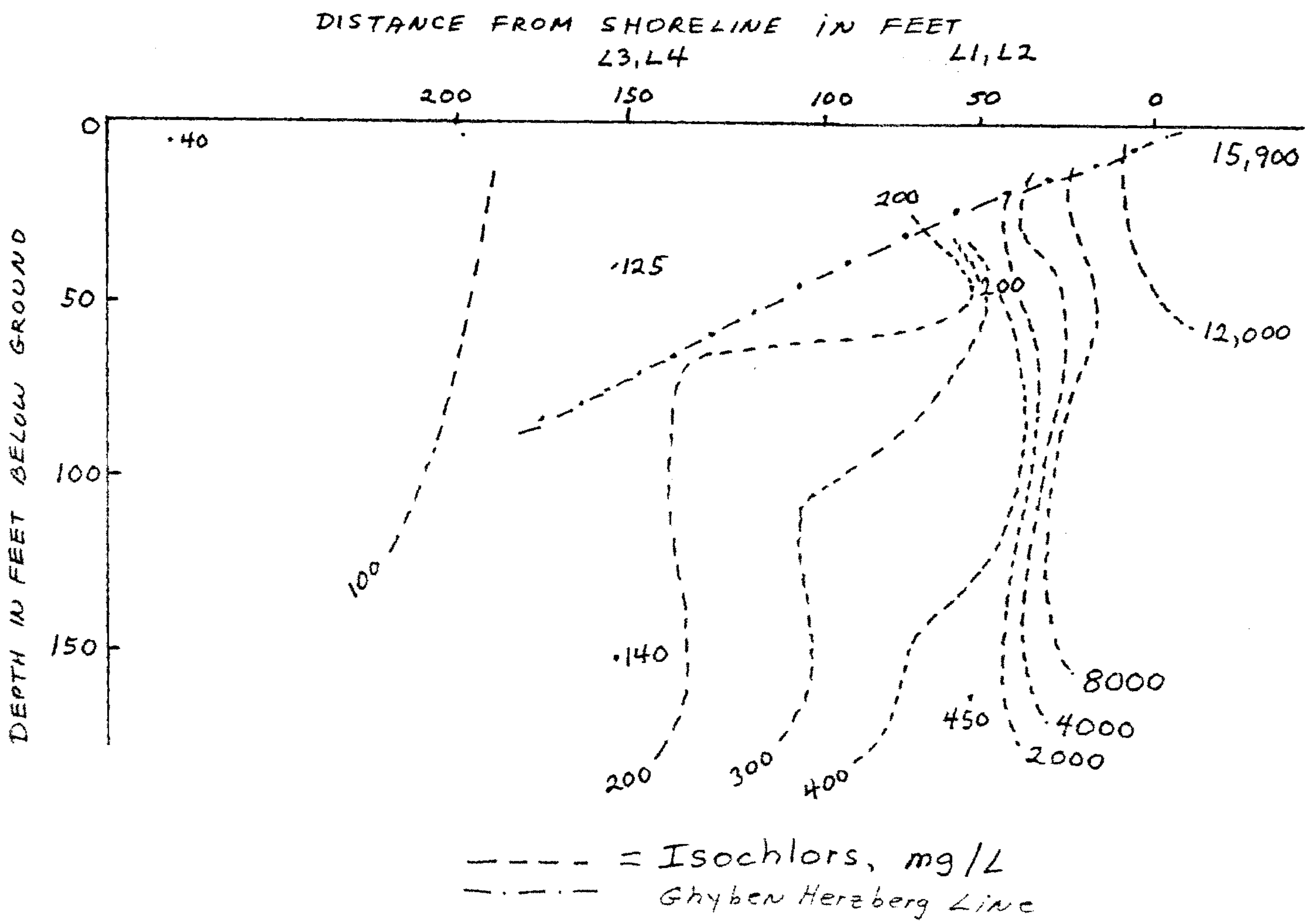


FIGURE 10: Isochlors in the test area

T A B L E S

TABLE 1: Annual Flows into Cole Harbour Estuary

Western Ear	Area (km^2)	Mean annual flow mgpd*	Percentage of Total
1. Little Salmon River	78.5	45.7	71.0
2. Broom Creek	3.7	2.2	3.4
3. Direct Discharge (No particular Brooks)	4.6	2.7	4.2
Eastern Ear			
1. Robinson Brook	8.3	4.8	7.5
2. Gammon Brook	3.6	2.1	3.3
3. Direct Discharge	4.7	2.7	4.2
Lower Harbour			
Direct Discharge	7.0	4.1	6.4
	Total.................	64.3 migd.	

Western Ear

Total = 50.6 migd.

Percentages from different sources		
	(1)	90.3
	(2)	4.4
	(3)	5.3

Little Salmon River

Total Precipitation = 1,381.2 mm = 108.4×10^6 cu m over 78.5 km^2/year.

Total Streamflow = 46×10^6 gpd = 75.87×10^6 cu m /year

Thus Streamflow/precipitation = 70%

Groundwater input $\simeq$ 10% of precip = 10.8×10^6 cu m

$\simeq$ 14% of Streamflow

= 6.4×10^6 gpd.

Total groundwater input to western ear = $0.14 \times 50.6 \times 10^6$

= 7.1×10^6 gpd

= 4930 igpm

*Using Pol's (1979) figure of 2.8 cfs/mi^2,

$$\text{Flow} = \frac{A}{2.59\ km^2/mi^2} \times 2.8\ cfs/mi^2 \times 6.23\ gals/ft^3 \times 86{,}400\ \frac{secs}{day} = A(km^2) \times 581{,}916$$

TABLE 2: Transmissivity and Permeability Data

T, igpd/ft				K* (Pump Test)		K (slug test) cm/sec			
Testhole	Q, igpm	Drawdown	Recovery	gpd/ft^2	cm/sec	Hvorslev Time Lag Method	and Bouwer Rice Method	Observation Well	Distance Drawdown
L1	1.0	12.6	12.6						
	2.0	5.9	5.1						
	4.0	8.3	10.5						
		Average	9.2	0.17	9.6×10^{-6}				
L2	1.0	4.3	16.5	0.19	1.1×10^{-5}	1.0×10^{-6}	8.0×10^{-7}		
		Average	10.4						
L3	1.0								
	2.0	340.7	340.7						
	4.0	240.0	310.6						
	8.0	185.3	277.9					164.4 (OW=L-1)	3673.0 (using L-1)
		Average	282.5	9.7	5.5×10^{-4}	2.1×10^{-3}	1.6×10^{-3}		
L4	1.0	8.0	8.0	0.15	8.6×10^{-6}				

*K = T/b where b = length of uncased portion of hole, feet. This represents apparent permeability.

TABLE 3: Storage Coefficient Data

WELL	t_o Mins.	Q	$S = \frac{0.3Tt}{r^2}$	
L1	3.2	1.0	0.13	
	0.88	2.0	0.04	
	0.8	4.0	0.03	
L2	0.55		0.02	
	1.05	2.0	1.19	
	0.62	4.0	0.57	
	0.5	8.0	0.39	Pumping well data
	1.0		3.4×10^{-6}	(using obs. well L-1, T = 164.4 gpd/ft r = 100 ft)
			1.0×10^{-3}	
L4	0.48		0.013	(using L4 and distance drawdown, T = 10 gpd/ft, r = 11 ft, t = 75 mins.)

T = Transmissibility in gpd/feet
r = radius in feet
t = time,
days to zero drawdown = $t_o/1440$

Reasonable S values assumed:

For L2, L4 (unconfined) 10^{-2}

For L1, L3 (confined) 10^{-4}

A P P E N D I C E S

Mean Total Precipitation, Shearwater Station (Millimetres)

Month	Water Year 1971-72	1972-73	1974-75	1975-76	1976-77	1977-78	1978-79	Mean (27 years of record)	
Oct.	67.3	188.2	95.5	176.8	152.9	141.9	103.7	113.3	(8.2%)
Nov.	168.9	224.8	130.6	117.6	94.4	82.2	36.8	151.9	(11.0%)
Dec.	95.3	191.8	91.9	225.1	180.2	212.4	123.9	148.3	(10.7%)
Jan.	160.0	111.0	181.4	130.6	107.4	225.8	262.2	147.3	(10.7%)
Feb.	137.2	135.6	59.7	128.4	95.6	38.4	115.1	128.3	(9.3%)
Mar.	228.1	118.9	143.0	91.6	96.6	84.9	167.2	111.8	(8.1%)
Apr.	120.4	134.6	86.6	87.7	80.5	157.0	99.2	105.4	(7.6%)
May	163.6	101.6	66.0	130.7	72.6	59.7	117.6	109.5	(7.9%)
June	115.3	123.2	65.3	65.6	131.3	100.4	72.3	85.1	(6.2%)
July	82.8	161.5	36.1	145.4	168.1	54.9	115.5	91.9	(6.7%)
Aug.	71.1	99.8	1.9	115.8	49.3	24.5	192.9	94.0	(6.8%)
Sept.	39.4	24.1	73.3	101.5	126.2	89.2	93.2	94.2	(6.8%)
TOTAL	1449.4 (+68.2)	1651.1 (+269.9)	1031.3 (-349.9)	1516.8 (+135.6)	1355.1 (-26.1)	1271.3 (-109.9)	1499.6 (+118.4)	1381.2	

Mean Total Snowfall, Shearwater Station (Centimetres)

Month	Water Year 1971-72	1972-73	1974-75	1975-76	1976-77	1977-78	1978-79	Mean (27 years of record)	
Oct.		8.6	33.0	1.0	-	-	1.0	0.3	(0.1%)
Nov.	5.1	14.0	2.5	-	13.2	5.7	9.4	6.9	(3.4%)
Dec.	32.5	41.4	19.8	34.7	20.4	51.1	44.5	39.9	(19.9%)
Jan.	67.3	29.7	74.9	22.5	31.5	36.4	25.2	47.5	(23.6%)
Feb.	50.3	74.7	42.9	49.2	29.2	38.4	8.2	53.3	(26.5%)
Mar.	34.3	15.5	45.5	41.3	31.1	30.5	6.5	40.1	(20.0%)
Apr.	83.6	6.4	22.6	2.1	11.2	19.2	15.1	11.9	(5.9%)
May	15.7	-	4.3	-	5.2	-	-	1.0	(0.5%)
TOTAL	288.8 (+87.9)	190.3 (-10.6)	245.5 (+44.6)	150.8 (-50.1)	141.8 (-59.1)	181.3 (-19.6)	109.9 (-91.0)	200.9	

Note: Numbers in parentheses refer to the difference from the 27 year mean.

Appendix 2

Mean Monthly Temperatures, Shearwater Stations
(Degrees C)

	Water Year							
Month	1971-72	1972-73	1974-75	1975-76	1976-77	1977-78	1978-79	Mean (27 years of record)
Oct.	10.5	7.6	6.8	9.1	8.7	9.8	8.5	9.9
Nov.	3.8	2.3	4.2	5.6	3.0	5.3	2.6	4.9
Dec.	-2.9	-3.1	-0.6	-2.5	-3.1	-1.5	-2.0	-1.1
Jan.	-4.8	-4.4	-3.8	-4.6	-6.1	-4.0	-2.8	-3.8
Feb.	-6.4	-5.3	-6.3	-2.7	-3.9	-5.4	-7.1	-4.2
Mar.	-2.4	0.1	-1.7	-0.6	1.6	-2.1	0.6	-0.7
Apr.	1.9	4.0	3.1	4.7	3.8	2.7	4.0	4.0
May	8.6	7.9	8.5	9.7	9.3	9.2	10.3	9.0
June	13.3	14.8	13.9	16.1	12.7	13.2	14.9	13.7
July	17.4	18.6	18.1	17.4	17.8	17.1	17.6	17.6
Aug.	17.5	18.1	17.4	18.3	18.1	19.3	17.4	17.8
Sept.	14.8	13.9	14.9	14.8	13.8	12.4	14.2	14.8

APPENDIX 3

BACKGROUND INFORMATION ON SALT WATER INTRUSION

HYDROGEOLOGICAL FACTORS AND INTERFACE DYNAMICS

NATURAL CONDITIONS
(1) THEORY
(2) FRACTURE EFFECTS

ALTERED CONDITIONS

GEOCHEMICAL FACTORS

CONTROL METHODS

APPENDIX 3

SALT WATER INTRUSION

Hydrogeological Factors and Interface Dynamics

Natural

(i) Theory

Under natural conditions, fresh groundwater is discharged into the ocean at or seaward of the coastline. For nearly horizontal flow, the balance between fresh and salt water in hydrostatic equilibruim can be described by the Ghyben-Herzberg relation. Referring to figure 3-1 (Todd, 1959), the total hydrostatic pressure at A is

$$PA=p_s g h_s$$

Where p_s is the salt water density, g is the accleration of gravity, and h_s is as shown. Similarly, at B, which is at the same depth as A but inland from it, $P_B = p_f \, g h_f + p_f \, g h_s$ where p_f= fresh water density and h_f is as shown. Equating these yields the equation

$$h_s = \frac{p_f \, h_f}{p_s - p_f}$$

[illegible]

Field measurements in several coastal areas have substantiated this result. The intrusion is of course limited by the extent of the aquifer and the elevation of the water table. Also, near the shoreline the relation must break down to form a seepage face for fresh-water outflow. Hydrostatic equilibrium implies no flow, yet groundwater flow invariably takes place near coast limes. From density considerations alone, without flow, a horizontal interface would develop with fresh water everywhere floating above salt water. Because the total pressure along an equipotential line is constant and the flow lines are sloping upward, the depth to the interface given by the Ghyben-Herzberg relation is less than the actual depth. For flat gradients the difference remains small, but for steep gradients large errors may be incurred.

For confined aquifers the above derivation can also be applied by replacing the water table by the piezometric surface. It is important to note from the Ghyben-Herzberg relation that fresh-salt water equilibrium requires that the water table, or piezometric surface

(a) lie above sea level, and (b) slope downward toward the ocean. Without these conditions, sea water will advance directly inland.

The length of the salt water wedge which exists at the intersection of an aquifer with the ocean can be calculated. Assuming that a seaward fresh-water flow q per foot of ocean front exists, then the approximate relation for a confined aquifer,

$$q = \frac{1}{2}\left[\frac{(p_s - p_f)}{(p_f)}\right]\frac{Kb^2}{L}$$

can be derived starting from Darcy's law, where p_f and p_s are fresh and salt-water densities, respectively, b and L are as defined in figure 3-2, and K is the coefficient of permeability. The evaluation indicates that the length of the intruded wedge is inversely proportional to the fresh-water flow or magnitude of fresh water head for uniform aquifer and fluid conditions. The equation can also be applied to unconfined aquifers by replacing b by the saturated thickness, providing the flow does not deviate greatly from the horizontal.

Glover (1959, and in Cooper et al, 1964) suggests that under dymanic conditions, fresh water flows through a narrow gap between a freshwater-seawater interface and the water table outcrop at the beach. The width of this gap can be calculated (see figure 3-3). The significance of this solution is that if the fresh water supply decreases, Q decreases, and x_o, the gap width, also decreases. Hence in times of drought the fresh-water body is conserved because of decrease in gap width and in seaward flow (seaward flow proportional to square of potential at a distance x=x from the shoreline). Glover's solution also assumes essentially static salt water.

In practice many workers have shown the presence of a zone of diffusion in which cyclic flow occurs (figure 3-4 illustrates this flow). The salt water cycles from the sea floor to the zone of diffusion and back to the sea. Kohout (1960, and in Cooper et al, 1964) estimates that as much as 10% of the total seaward flow may be seawater. To maintain this, a mechanism is required to transport salts from the sea floor through the aquifer to the zone of diffusion. Concentration gradients are too small for dispersion transport, hence flow accompanied by a loss of head is proposed. The salt transport is accompanied by a loss of head in the salt water flow, thus lowering the extent to which saltwater occupies the aquifer. The forces causing the circulation must be powerful enough to recreate the zone of diffusion continuously. Molecular diffusion is too feeble. Cooper et al (1964) suggests that the mechanism may be reciprocative motion of the salt water front resulting from ocean tides and from the rise and fall of the water table due to variations in recharge and other forces (including pumping). During movement of the front the convection component of dispersion causes elements of each fluid to be transferred into the opposite environment, where they become inseparably blended with the other fluid by mixing and molecular diffusion.

Water level data obtained by Kohout (in Cooper et al, 1964) do indicate this cyclic flow. Significant features in the flow net (figure 3-5) are the zero horizontal gradient line, which separates the seaward and landward horizontal components of flow, and the streamline that separates the water into two regions depending on its origin as fresh or sea water. These two lines intersect at the base of the aquifer. Thus when freshwater head is high after a recharge event, water in all parts of the aquifer moves seaward. As head decreases, a landward gradient forms in the lower part of the aquifer, and salt water flows inland into the zone of diffusion to a line along which there is no horizontal component of flow; salt water then moves up and returns to the sea. This cyclic flow thus limits the salt water intrusion into the aquifer.

On an oceanic island (and also on peninsulas), recharge is a function of rainfall. Using Dupuit assumptions and the Ghyben-Herzberg relation, an approximate fresh water boundary can be defined (see figure 3-6). Assume a circular island of radius R, as shown, receiving an effective recharge from rainfall at a rate W. The outward flow Q at radius r is

$$Q = 2\pi rK(1.025H)\frac{d(0.025h)}{dr}$$

where K is permeability and h is shown. The change in flow through a cylinder of radius r and thickness dr amount to

$$dQ = 2\pi rW\,dr$$

Inte~~r~~grating, and noting the Q=0 when r=0, yields

$$Q = \pi r^2 W$$

Equating the above gives:

$$\frac{Wr\,dr}{0.0512K} = h\,dh$$

Intergrating and applying the boundary condition that h=0 when r=R,

$$h^2 = \frac{W}{0.0512K}(R^2 - r^2)$$

Thus, the depth to salt water at any location is a function of the rainfall recharge, size of the island, and permeability. Tidal and seasonal fluctuations may form a transition zone between the fresh - and salt water bodies; likewise, seepage surfaces at the coast line are subject to varying salinity concentrations (Todd, 1959b).

The rate of change of the hydrologic system to the "pumps" of fresh water recharge and cyclic circulation can be very slow. Lau (1967) and Wentworth (1942) discuss the storage consequences of the Ghyben-Herzberg theory. By this theory, the "bottom storage" below sea level is 40 times that of "top storage" above sea level. After the system is in operation for a long period, equilibrium is established. While a change in water table leads to an immediate response tending towards equilibrium, the 40/1 ratio in amounts of water to be moved precludes any prompt change in bottom storage. The time lag may be months or years, depending on the magnitude of changes and aquifer permeability (Lau, 1967). No independent forces can be applied to bottom storage except change in sea level. Even use in a well reflects drawdown at the top of the well. Any pulse at water table is thus followed by changes in bottom storage at a declining rate set by total permeability and water quantities. This storage feature thus invalidates any safe yield predictions based on short term data.

Under natural conditions, as discussed above, fresh water generally discharges to sea water. However, seawater intrusion can still occur by mechanisms such as: (1) tide and wave action forcing seawater inland, especially in areas of low permeability (Lazreg, 1972), (2) flooding of salt water over flat and low areas of shoreline, such as tidal marshes, (3) groundwater flow parallel to a fresh-saltwater boundary which accelerates mixing and increases thickness of the diffusion zone (Lazreg, 1972), (4) leakage around faulty casings, (5) sea spray, (6) decreased fresh-water recharge (natural climatic variations). Since the length of the intruded salt wedge varies inversely with the magnitude of the fresh water head, reverse flow is not required for intrusion, only a freshwater head reduction (EPA, 1973) and (7) long term changes in mean sea level.

An important point to note in comparison of heads is that seawater head or environmental sea water head must be converted to freshwater head for comparative purposes (Kohout, 1960, Ferris et al, 1962, Cooper et al, 1964, Luscynski, 1961 and 1966):

$p = \rho g l$ $\quad$ p = pressure at bottom of casing

ρ = density of water in casing

g = acceleration due to gravity

l - measured length of water column above casing terminus

$$l_f = \frac{\rho_s \; l_s}{\rho_f}$$

The fluid density in the well can be calculated by the following equation (Pinder et al, 1970):

$P = C_W + C = \rho_o + (1-E)c$ $\quad$ ρ = desired density

C = salt concentration in g/cc or g/ml

$E \sim$ = 0.3 for c values to as much as seawater

eg. For 35% salinity, NaCl = 35,000 mg/l

= 35 g/l

= 0.035 g/ml

Therefore, $P = 1.0 + (1-0.3)\ (0.035)$

$= 1.025$

(ii) Fracture Effects

The effect of fracture orientation on intrusion must be considered, especially in the Nova Scotian context of islands and long peninsulas. In Maine, where major groundwater aquifers are limited by low permeability of crystalline rock, it was found that foliation joints parallel to strike were the prime and most throughgoing fractures, and thus control the preferred permeability. Where the preferred permeability strikes parallel to the peninsula, intrusion is rare. In areas with a large number of joints crosscutting the foliation, intrusion is more common. (Source: tape on fractured Rock Hydrology, NWWA conference, Boston, 1978).

There is different views in the literature as to whether the direction of preferred permeability in fractured rock is parallel or perpendicular to strike. For example, Marine's (1966) pump test data from schist, gneiss and quartzite indicated that for a similar distance perpendicular and parallel to strike, the drawdown was greater perpendicular to strike, and on the updip side of the pumping well, ie. hydraulic communication was better in a direction perpendicular to strike. In contrast, Vecchioli (1965) found that in fractured shales with bedding plane fractures and vertical fracture systems parallel and perpendicular to strike, all wells which interfered were aligned parallel to strike. J. Gibb and J. Vaughan also found this in sedimentary rocks (pers comm). Vecchioli (1965) suggests that if it is assumed that direct interference indicates that two wells tap a single fracture system, and that small drawdown indicates poor hydraulic connection, then the smaller T value indicates the direction of greater permeability, or the alignment of the major fracture system. Hence parallel to strike would be the major fractures.

DeWiest (1969) suggests that T is usually larger parallel to strike rather than perpendicular.

Although these various reports are somewhat contradictory, they do suggest that if a large number of joints strike perpendicular to a peninsula, salt water instrusion is more likely. Also, wells should be aligned so as to minimize drawdown.

In sedimentary sequences, Vander Camp (pers comm) found greater intrusion where bedding dipped inland. In fractured rocks, as in Lawrencetown, the presence of fractured zones with enhanced permeability may be considered analagous to varying permeability layers, each zone with its own discrete pressure. Thus, it may be possible for fresh water to occur below salt water in fractured rocks. Also, the interface will likely move further inland at higher permeability zones. Since the seawater is differentially flushed from more permeable zones (Hariss, 1967), stratified zones of fresh and seawater may occur more quickly, and more than one interface may be encountered in a vertical section. Hence, an irregularly shaped intrusion front both in vertical and horizontal profile can be expected in the field.

Altered Conditions

The main casual factor of altered natural conditions is pumping which either reduces or reverses head and decreases fresh water flow to the ocean. Other factors which may play a role are: a) drainage channels to reduce water table elevation b) accidental or inadvertent destruction of natural barriers c) additional salt sources, such as formation or road salt and d) connate saline water.

In the Lawrencetown area the main factor is increased pumpage due to development, along with naturally low permeability of the bedrock. The interface dynamics have been discussed in the previous section, however an important point to stress is that reduction of freshwater head by one foot allows the interface to rise 40 feet.

A point which remains to be discussed is the relationship of intrusion to permeability and well depth. Cooper et al (1964) suggest that in aquifers of alternating high and low permeability beds, if the interface moves x units inland in high K units, it may only move 0,1x units in a low K unit. A. Vander Kamp (pers comm) found that in horizontal bedded, fractured sandstone, each bed had its own fresh/saltwater interface, hence fresh water occurred below salt water. Carr (1969) also found that the base of the fresh flow system varied depending on lithology and on the type of flow system penetrated. He found a zone of diffusion from sea level to 185 feet into bedrock, fresh groundwater from 185-600 feet, then seawater. Kohout et al (1976, 1977) found fresh water below the continental shelf to be related to Pleistocene recharge and slow adjustment of the trapped fresh water. This lends credence to Wentworth's theories (1942, 1948).

Geochemical Factors

The recognition of sea water in groundwater from chemical quality data has been widely different between investigators. Todd (1956b) suggests the use of bar graphs, with contaminated sea water showing much higher total salt levels and increase in the ratio of $CL/(CO_3+HCO3)$. Carr (1969) suggested that on Prince Edward Island, $CL > 40$ mg/l indicated intrusion, with $CL \geq 100$ mg/l representing zone of diffusion waters. Carr (1969) found a linear relationship between Cl and TDS and Cl and conductivity in simple mixing of fresh and salt water, except where $TDS \leq 500$ mg/l. Mink (1960) found a linear relationship of Ca, Mg, and (Na+K) with chloride in well waters and diluted sea water (seawater diluted to the chloride content of well water) for simple mixing. New Brunswick DOE found contaminated water to have TDS >700 ppm, as compared to about 150 ppm for uncontaminated water, with the contaminated water being Na-Ca, $Cl-HCO_3$ on Schoeller semilog plots.

Piper (1944) used trilinear plots: mixtures of two waters in all proportions, if all products remain in solution, plot on the three fields on the respective straight lines that join the points representing the respective chemical characters of the two waters mixed. Hence, progressively deteriorated water in coastal areas must plot on a set of three vectors directed toward the composition of ocean water for simple mixture of fresh and salt water.

The linear equation is:

$Vm\ Cm = (Ca\ Va) + (Cb\ Vb)$

Va = volume in mixture M of water composition A

Vb = volume in mixture M of water composition B

Ca = concentration of chemical constituent in mixture A

Cb = concentration of chemical constituent in mixture B

(in epm or ppm)

When a simple admixture is contraindicated, the main modification process expected to produce anomalies are (Revelle (1941) and Mink (1960):

1. Addition of constituents by solution. This is negligible in crystalline rocks except for silica.

2. Precipitation/dissolution of insoluble compounds. eg. seawater can hold about three time the $CaSO_4$ it contains, hence ocean water in contact with gypsum/anhydrite will dissolve it.

3. Ion exchange eg.

$2Na^+ + Ca, Mg\ clay = 2\ Na\ clay + Ca^{++}, Mg^{++}$.

Revelle (1941) presents equations relating percentage of sodium in water to that in soil.

4. Sulfate reduction by reaction with organic material and anaerobic bacteria in sea bottom sediments or in reduced groundwater, and substitution of carbonic or other weak acid radicals.

$$\text{eg. } R^2 + SO_4 + \overset{\text{(Organic)}}{2C} \rightleftarrows RS + 2\ CO_2$$

$$RS + 2\ CO_2 + H_2O \rightleftarrows R^2 + (HCO_3^-)_2 + H_2S$$

5. Loss of HCO_3^- by release of pressure as the water moves into the well, with loss of CO_2 and precipitation of Ca CO_3

$$Ca\ (HCO_3)\ 2 \rightleftarrows Ca\ CO_3 + CO_2 + H_2O$$

6. Reduction of NO_3 to NH_3

These reactions can be generally grouped into (Lusczynski et al, 1966).

1. Reactions between rocks and minerals and the water.
2. Reactions between the water and its constituents.
3. Dispersion by convection and ionic diffusion.

Rates of dispersion attributable to tidal effects in varying permeability sequences are considerably larger than those in homogeneous beds, resulting in a wider zone of diffusion.

Recent work in sedimentary rocks (S. Hattie, per comm) suggests that a Ca/Na (mg/L) ratio of 0.3-0.8 indicates salt water intrusion as compared to formation salt.

Vacher (1980) uses a "relative sainity" value defined by chloride concentration to determine the depth to the transition zone and interface. This method will be used in the text.

Control Methods

The main methods of control of saltwater intrusion will only be summarized here for further details, refer to EPA (1973), Bruington (1969) Todd (1959b), and Bruington (1972):

1. Increased seaward gradient.
2. Artificial recharge. The limitation here is that the aquifer may not have the permeability to conduct the required additional flow.
3. Fresh water ridge parallel to coast - a pressure barrier.
4. Extraction barrier parallel to coast to create a pumping trough.
5. Combination of extraction/injection.
6. Subsurface barrier eg. trench backfilled with puddled clay, or a line of grout wells. This is feasible only in shallow formations.
7. Tide gate control.
8. Reduce pumping-either terminate, decrease individual well extractions, or decentralize wells. This assumes decreased demand, or availability of an alternate source.
9. Continue pumping in the historical pattern but use a supplemental source to infiltrate at a basin recharge area. This assumes that the recharge area is known and that the aquifer has sufficient permeability.

Freeze and Cherry (1979) state that the only method proven effective and economic is either reduction or rearrangement of groundwater pumping.

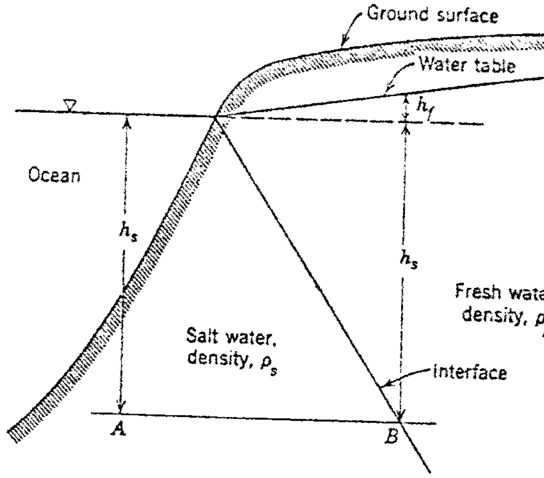


Figure 3-1a: Idealized sketch of fresh- and salt-water distributions in an unconfined coastal aquifer to illustrate the Ghyben-Herzberg relation.

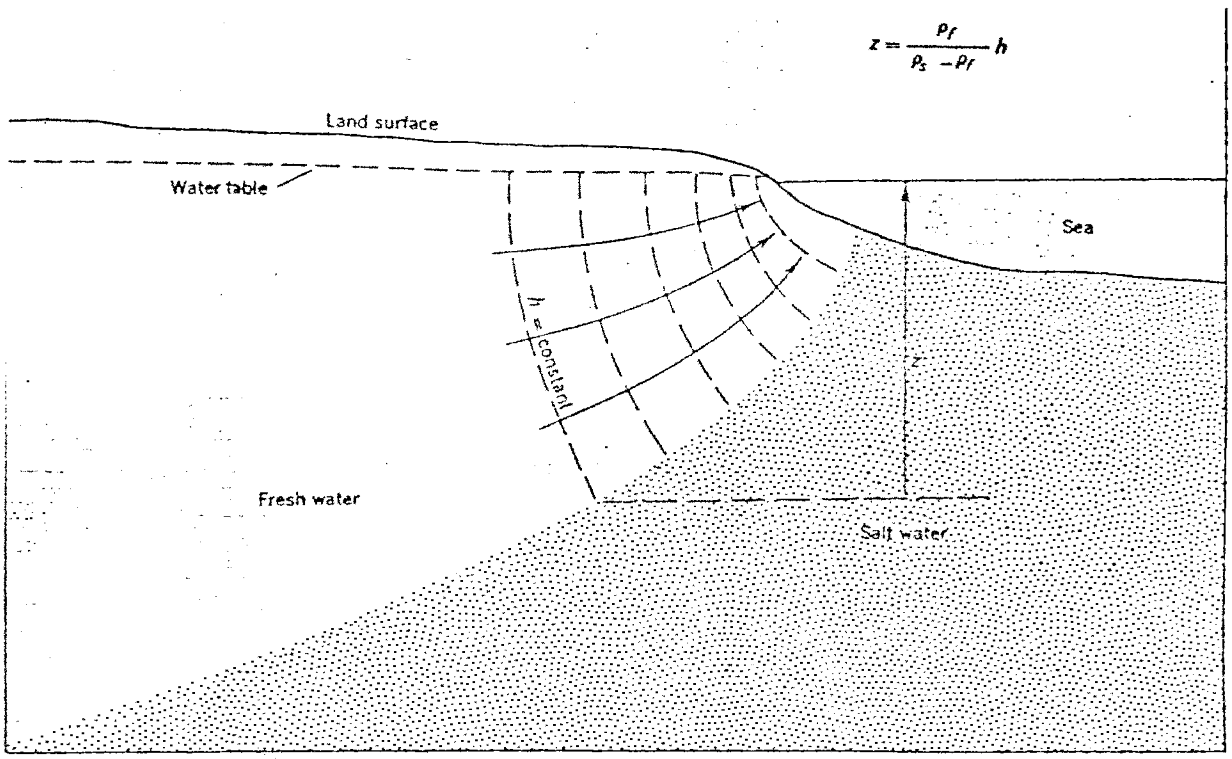


Figure 3-1b: Balance between fresh water and salt water in a coastal aquifer in which the salt water is static.

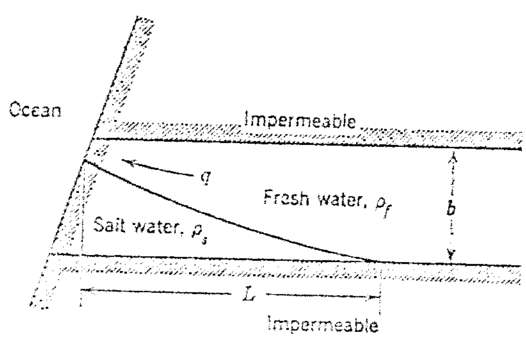


Figure 3-2: Salt water wedge in a confined aquifer.

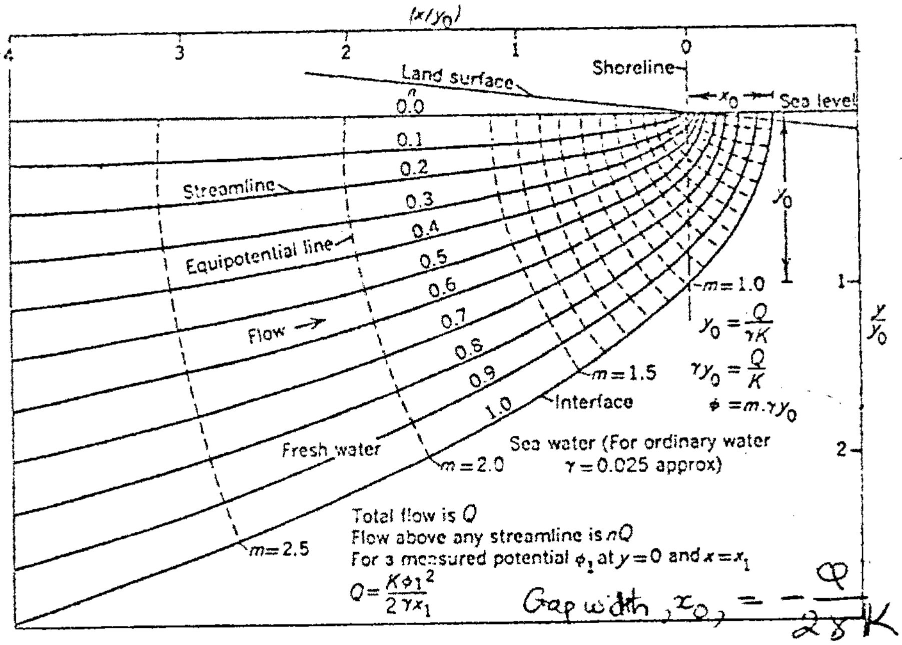


Figure 3-3: Flow pattern near a beach.

x = distance horizontally landward from shoreline, ft
y = distance vertically downward from sea level, feet
Q = freshwater flow per unit length of shoreline, ft²/sec
K = permeability of freshwater strata, ft/sec
γ = specific gravity of salt water minus freshwater

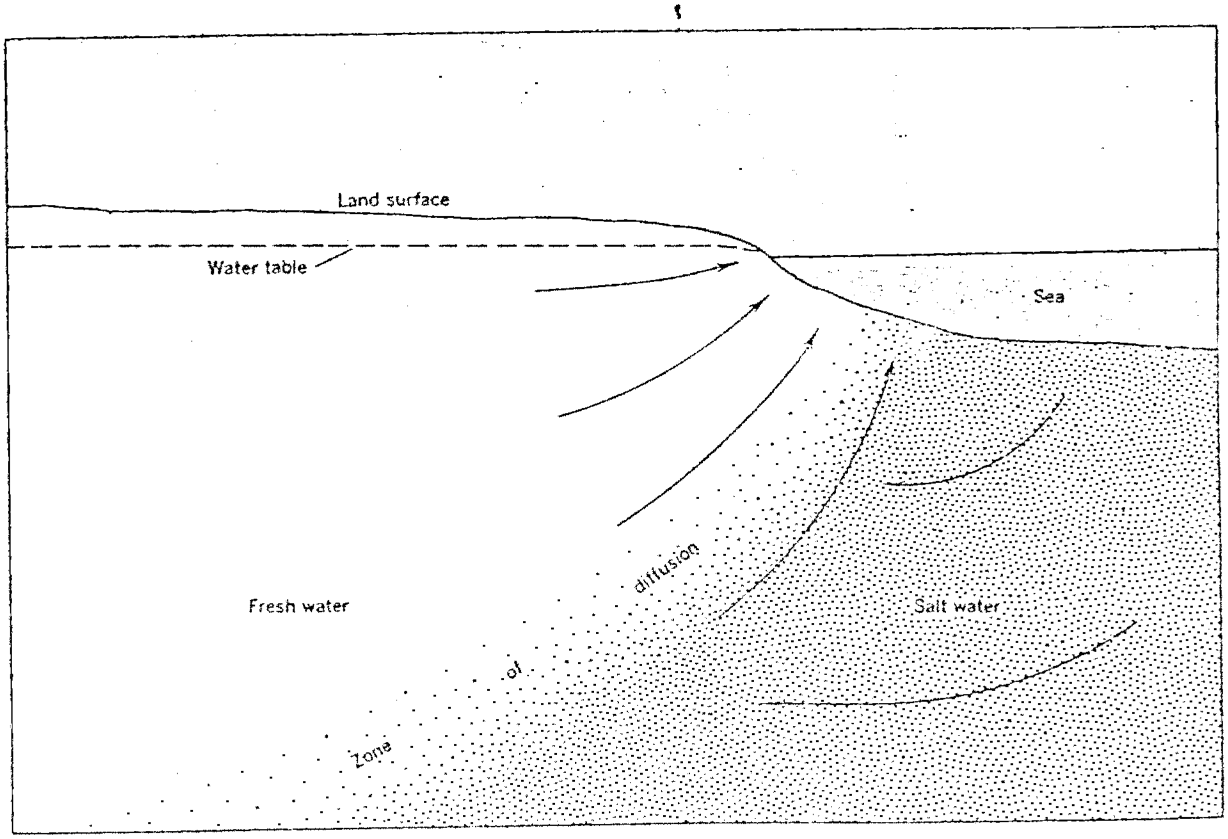


Figure 3-4: Circulation of salt water from the sea to the zone of diffusion and back to the sea.

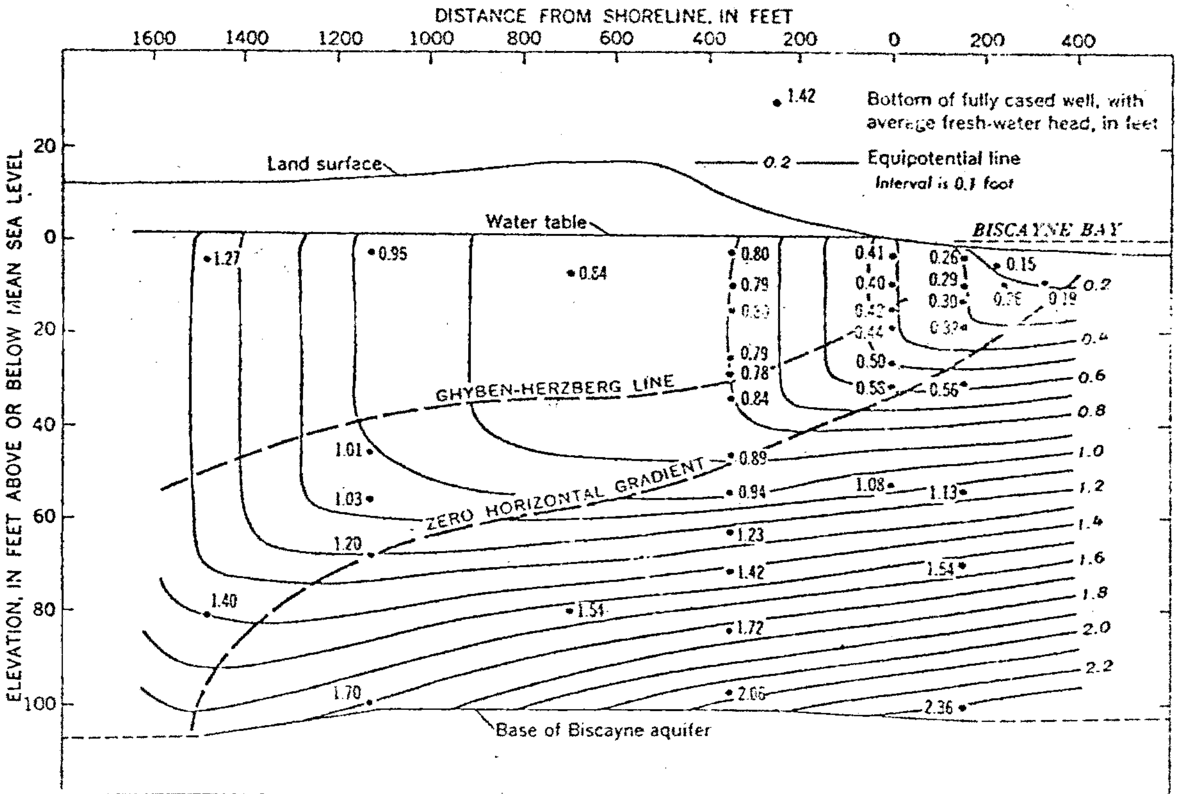


Figure 3-5a: Cross section through the Cutler area, near Miami, Fla., showing lines of equal fresh-water potential for a low-head condition; average for September 18, 1958.

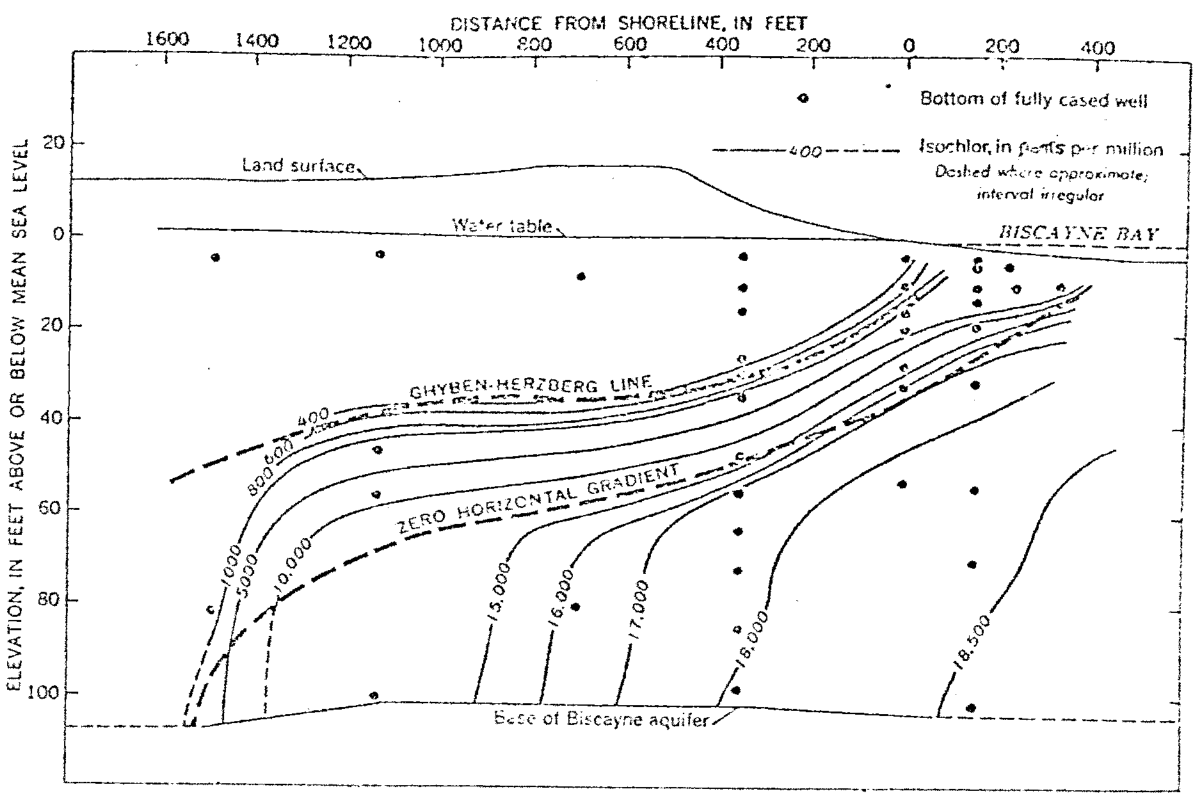


Figure 3-5b: Cross section through the Cutler area, near Miami, Fla., showing the position of the line of zero horizontal gradient within the zone of diffusion, September 18, 1958.

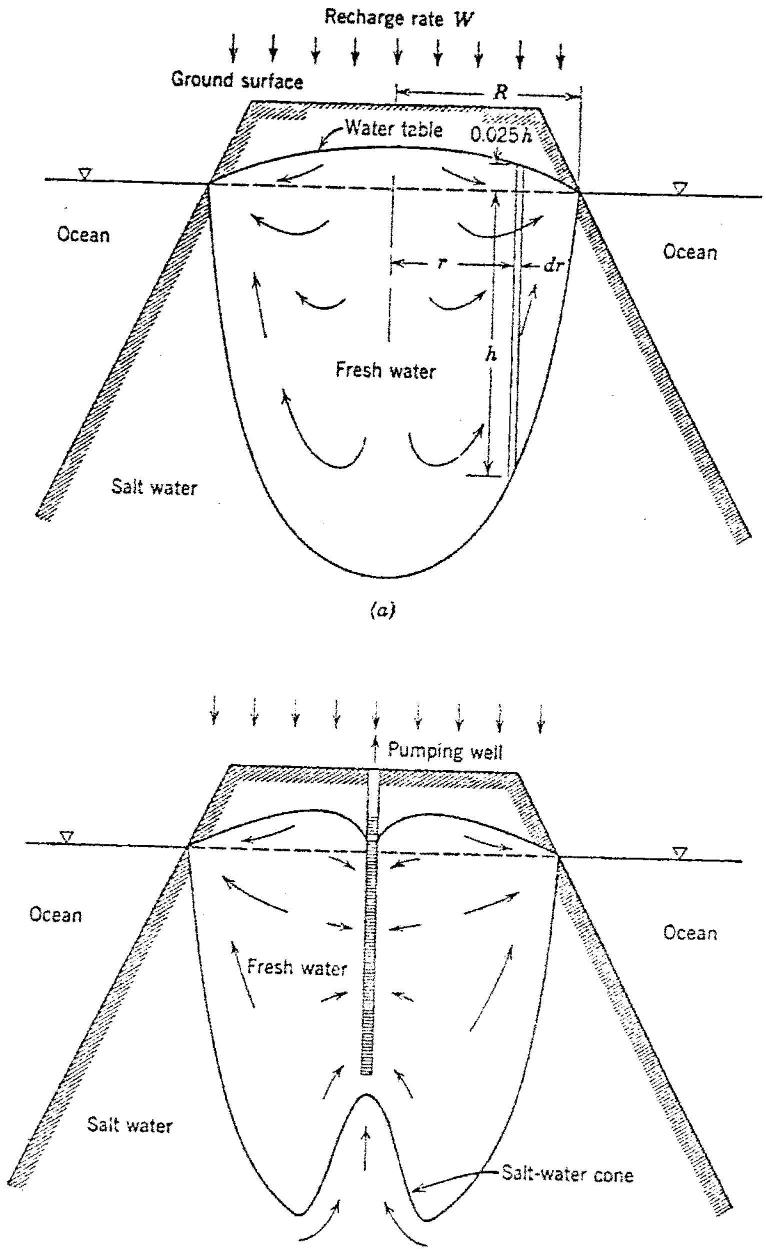


Figure 3-6: Fresh-water lens in an oceanic island under (a) natural conditions and (b) with a pumping well.

APPENDIX 4

TEST DRILLING, UPPER LAWRENCETOWN

#L1

Depth: 200 feet
Diameter: nominal 5 inch
Casing: 133 feet of 5 inch casing, cemented in
Static waterlevel: 6 inches above ground
Water at: 6-10 ft., 33-34 ft., 37-39 ft. (cased off) and 179-180 ft.
Estimated yield: about 0.5 gpm
Log: 0-10 sandy, bouldery till and weathered bedrock
10-37 weathered bedrock
37-49 light to medium gray quartzite
49-140 gray quartzite mainly slaty quartzite interbeds and quartz veins minor
140-150 mainly dark gray slate and quartz stringers
150-175 predominantly gray quartzite
175-180 gray slate and quartz stringers
180-200 greenish to blue-greenish gray quartzite with minor quartz and calcite stringers

#L2

Depth: 75 feet
Diameter: 6 inches
Casing: 22 feet
Static water level: 4-10 feet
Water at: 16-17 feet (cased off)
Fractures at: 33,40,45,56
Estimated yield: 0.5 gallons per hour
Log: 0-5 clay and boulders
5-12 weathered bedrock
12-75 greyish quartzite mainly; minor slaty quartzite and quartz stringers

#L3

Depth: 174 feet
Diameter: 6 inches
Casing: 145 feet
Static water level: 4 feet
Major fracture zone and water at: 152-155 feet
Estimated yield: 20-30 igpm
Log: 0-5 sand and gravel
5-12 boulders and broken bedrock
12-45 gray quartzite
45-102 greenish gray quartzite and quartz stringers
102-120 same as 45-102, but increase in slaty interbeds
120-152 greenish gray quartzite
152-165 dark gray and black slate and quartz veins
165-174 greenish gray quartzite and slaty quartzite, minor quartz stringers

#L4

Depth: 75 feet
Diameter: 6 inches
Casing: 21 feet
Static water level: 5 feet
Fractures: 18 ft., 32 ft., 39 ft.
Estimated yield: less than 0.5 gallons per hour
Log: 0-8 sandy till and boulders
8-15 weathered bedrock
15-75 [illegible]

15-75 gray quartzite and minor slaty stringers?

APPENDIX 5

Aquifer Test Data

L-1

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: M. Rushton M Rodgers MEASURED BY: Mike Rushton

LOCATION OF PROJECT ______ WELL LOCATION: Lawrencetown #1

STATUS Pumping 16GPM R ______ DATE 23/8/77 PAGE ______

(pumping or observation wells) (distance from pumping well in feet and direction)

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q — discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
23/8/77		0			11.1				
		1			15.8	4.7			
		2			18.7	7.6			
		3			18.5	7.4			
		4			20.1	9.0			
		5			19.8	8.7			
		6			19.8	8.7			
		7			20.7	9.6			
		8			21.1	10.0			
		9			21.1	10.0			
		10			22.1	11.0			
		15			24.7	13.6			
		20			27.7	16.6			
		25			29.7	18.6			
		30			31.1	20.0			Temp 12°C Cond 100
		40			34.1	23.0			
		50			36.6	25.5			
		60			38.8	27.7			
		75			40.6	29.5			
		90			42.5	31.4			
		100			43.5	32.4			

H_2O Levels [illegible]
of 23/8/77,
L1 - 1.35'
L2 - 4.6'
3 - 2.0

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: MEASURED BY:

LOCATION OF PROJECT WELL LOCATION: Lawrencetown #1 1GPM

STATUS Recovery (pumping or observation well) R (distance from pumping well in feet and direction) DATE PAGE

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q — discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
23/3/7?		0			43.9	t/t'			
		1			37.7	101			
		2			34.7	51			
		3			32.0	34			
		4			28.7	26			
		5			26.5	21			
		6			26.1	17.6			
		7			25.5	15			
		8			24.8	13.5			
		9			24.3	12			
		10			23.3	11			
		15			21.5	8			
		20			19.3	6			
		25			16.7	5			
		30			15.8	4			
		40			12.3	3.5			
		50			10.0	3			

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY M. Rushton M. Rodgers MEASURED BY: Mike Rushton

LOCATION OF PROJECT ______ WELL LOCATION: Lawrencetown Well #1

STATUS Pumping 2GPM R ______
(pumping or observation well) (distance from pumping well in feet and direction)

DATE 23/8/77 PAGE ______

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q — discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
	0					95			2 GPM
	1					10.5			
	2					23.1			
	3					26.2			
	4					27.8			
	5					29.8			
	6					30.9			
	7					33.5			
	8					35.9			
	9					36.6			
	10					38.3			12°C Cond 1050
	15					46.9			
	20					52.6			
	25					57.0			
	30					65.0			
	40					73.6			
	50					83.5			
	60					93.5			
	75					105.5			
	90					110.			
	100					116.2			Temp 10°C

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ________ MEASURED BY: ________

LOCATION OF PROJECT ________ WELL LOCATION: Well #1 Pumping 2 GPM

STATUS Recovery (pumping or observation well) ________ R ________ (distance from pumping well in feet and direction) DATE ________ PAGE ________

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
23/5/77		0			116.2	t/t			
		1			100.9	101			
		2			[illegible]	51			
		3			96.3	34			
		4			[illegible]	26			
		5			[illegible]	21			
		6			[illegible]	17.6			
		7			[illegible]	15			
		8			[illegible]	13.5			
		9			[illegible]	12			
		10			[illegible]	11			
		15			[illegible]	8			
		20			[illegible]	6			
		25			[illegible]	5			
		30			[illegible]	4			
		40			[illegible]	3.5			
		50			[illegible]	3			
		60			[illegible]	2.6			

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: M. Rushton, M. Rodgas MEASURED BY: M. Rushton

LOCATION OF PROJECT Lawrencetown WELL LOCATION: Lawrencetown Well # 2

STATUS Pumping 4 GPM
(pumping or observation well)

R
(distance from pumping well in feet and direction)

DATE PAGE

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q — discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
24/7/77		0			1.5				4 GPM
		1			5.4				
		2			13.8				
		3			17.9				
		4			22.8				
		5			25.0				
		6			33.4				
		7			37.8				
		8			42.9				
		9			47.4				
		10			51.7				Temp 10°C Cond. [illegible]
		15			71.2				
		20			89.8				
		25			108.8				
		30			122.6				
		40			147.0				
		45			152				Pump Level
									Note H2O levels [illegible]
									24/8/77
									L 1 - 1.5'
									L 2 - 4.7'
									L 3 - 6.15

L 4 - 6.6

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ____________ MEASURED BY: ____________

LOCATION OF PROJECT ____________ WELL LOCATION: Lawrencetown Well #1 4/26/79

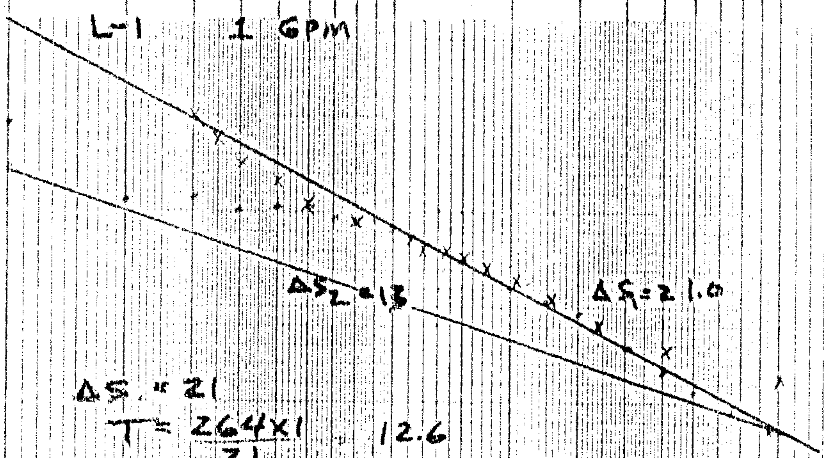
STATUS Recovery ____________ (pumping or observation well)

R = ____________ (distance from pumping well in feet and direction) DATE ______ PAGE ______

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet s/t	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0		152					
		1		149.5	45				
		2		147.5	23.5				
		3		147	16				
		4		145.6	12.2				
		5		143.8	10				
		6		142.2	8.5				
		7		140.6	7.4				
		8		139.2	6.6				
		9		137.7	6				
		10		135.9	5.5				
		15		125.5	4				
		20		115.5	3.2				
		25		106.2	2.8				
		30		97.1	2.5				
		40		82	2.1				
		50		69.4	1.9				
		60		58.3	1.75				
		70	75	44.1	1.6				
		105	90	34.3					
		120	105	26.6					
		150	120	19.9	1.3				
			150	15.5					

10min 100

L-1 1 GPM



$\Delta S_1 = .21$

$T = \frac{264 \times 1}{.21}$ 12.6

$Q = \frac{12.6 \times 155}{1848}$ 1 igpm

$\Delta S_2 = .13$

$T = 20.3$

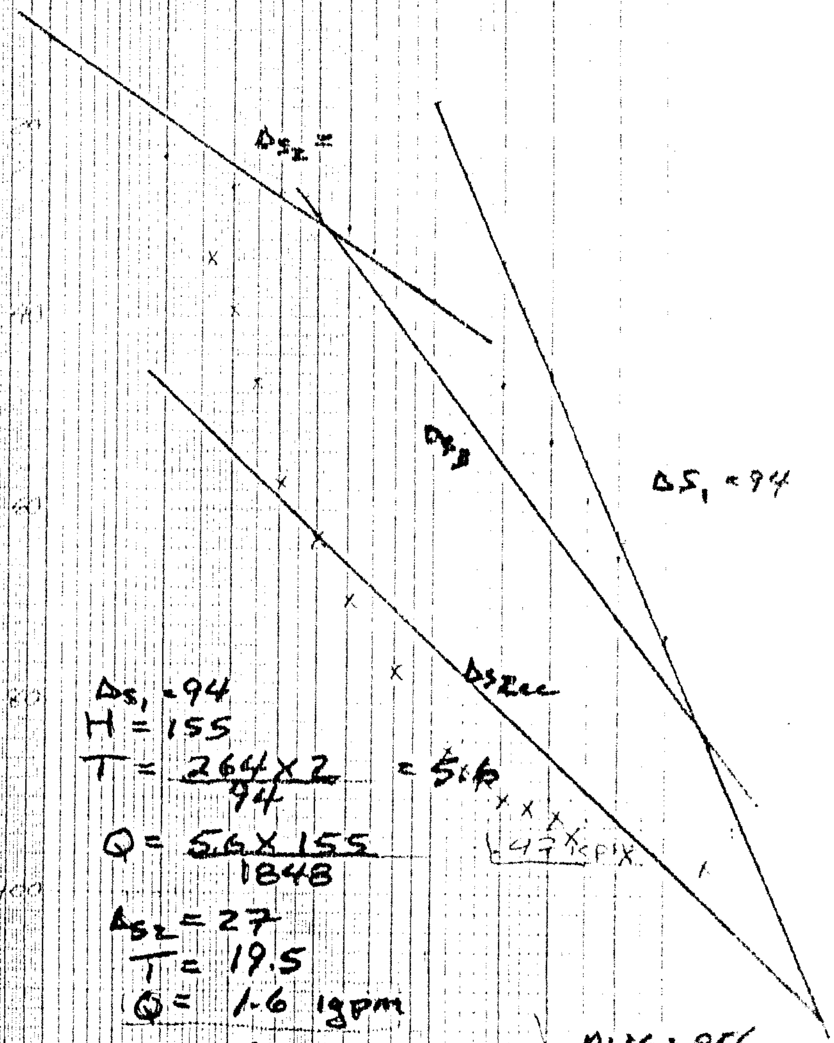
$Q = 1.7$ igpm

$\Delta S_{Recovery}$ same as ΔS_1

AUG. 1.35

10min 100

L-1 2 GPM



$\Delta S_1 = .94$

$H = 155$

$T = \frac{264 \times 2}{.94} = 5.6$

$Q = \frac{5.6 \times 155}{1848}$

$\Delta S_2 = .27$

$T = 19.5$

$Q = 1.6$ igpm

$\Delta S_3 = .54$

$T = 9.7$

$\boxed{Q = .8 \text{ igpm}}$

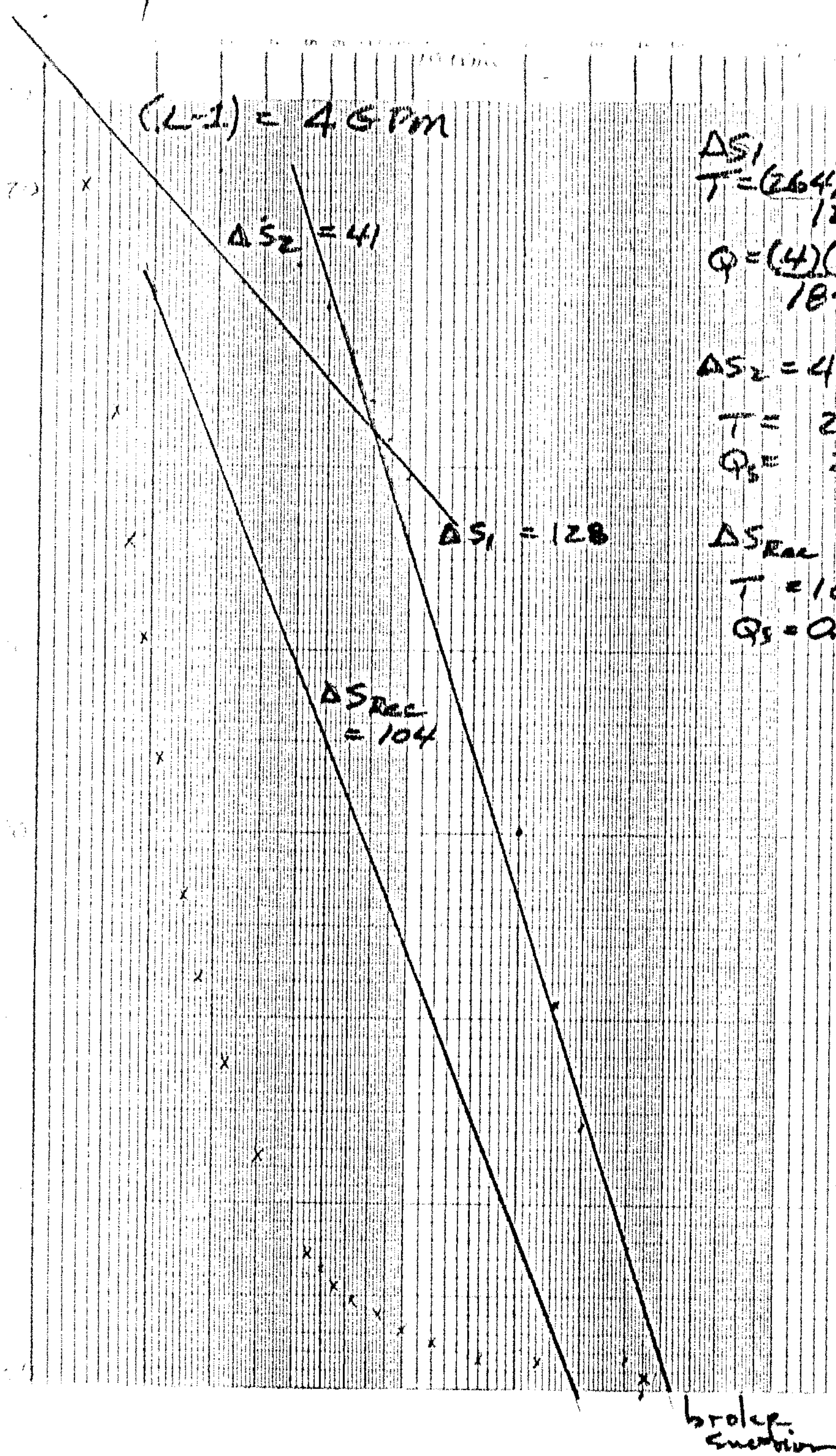
$\Delta S_{Rec} = .38$

$T = 13.9$

$Q_R = 1.2$ igpm

AUG. .956

.47 EPIX



$\frac{\Delta S_1}{T} = \frac{(264)(.48)}{128} = 4$

$Q = \frac{(4)(155)}{1848} = 0.331$ gpm

> AVG = 1.2

$\Delta S_2 = 41$

$T = 25.75$

$Q_s = 2.1$

$\Delta S_{Rec} = 104$

$T = 10$

$Q_s = .085$

Summary #1

AVG of ~~of~~ Q values 1.16 gpm

1 Well 1 GPM

Short term pumping Well #1
is probably good for 1 gpm
Safe yield over 20 yr period 0.75 gpm

APPENDIX 6

Aquifer Test Data

L-2

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: [illegible] M Rodgers MEASURED BY: Mike Rushton

LOCATION OF PROJECT ______ WELL LOCATION: Lawrencetown #2

STATUS Pumping 1 GPM (pumping or observation well) R = ______ (distance from pumping well in feet and direction) DATE 30/8/77 PAGE 1

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
30/8/77		0			1.8				
		1			6.8				
		2			10.2				
		3			14.4				
		4			16.3				
		5			17.0				
		6			17.5				
		7			18.1				
		8			19.0				
		9			19.7				
		10			20.3				Temp 12°C Cond [illegible]
		15			24.4				
		20			28.7				
		25			33.6				
		30			38.4				Temp 13°C Cond 495 [illegible]
		40			41.5				
		50			53.5				
		60 55			57.0				Temp 13°C Cond [illegible]
		~~75~~							Broke Suction
		~~90~~							
		~~100~~							

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT – WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: M Rushton M Rodgers **MEASURED BY:** Mike Rushton

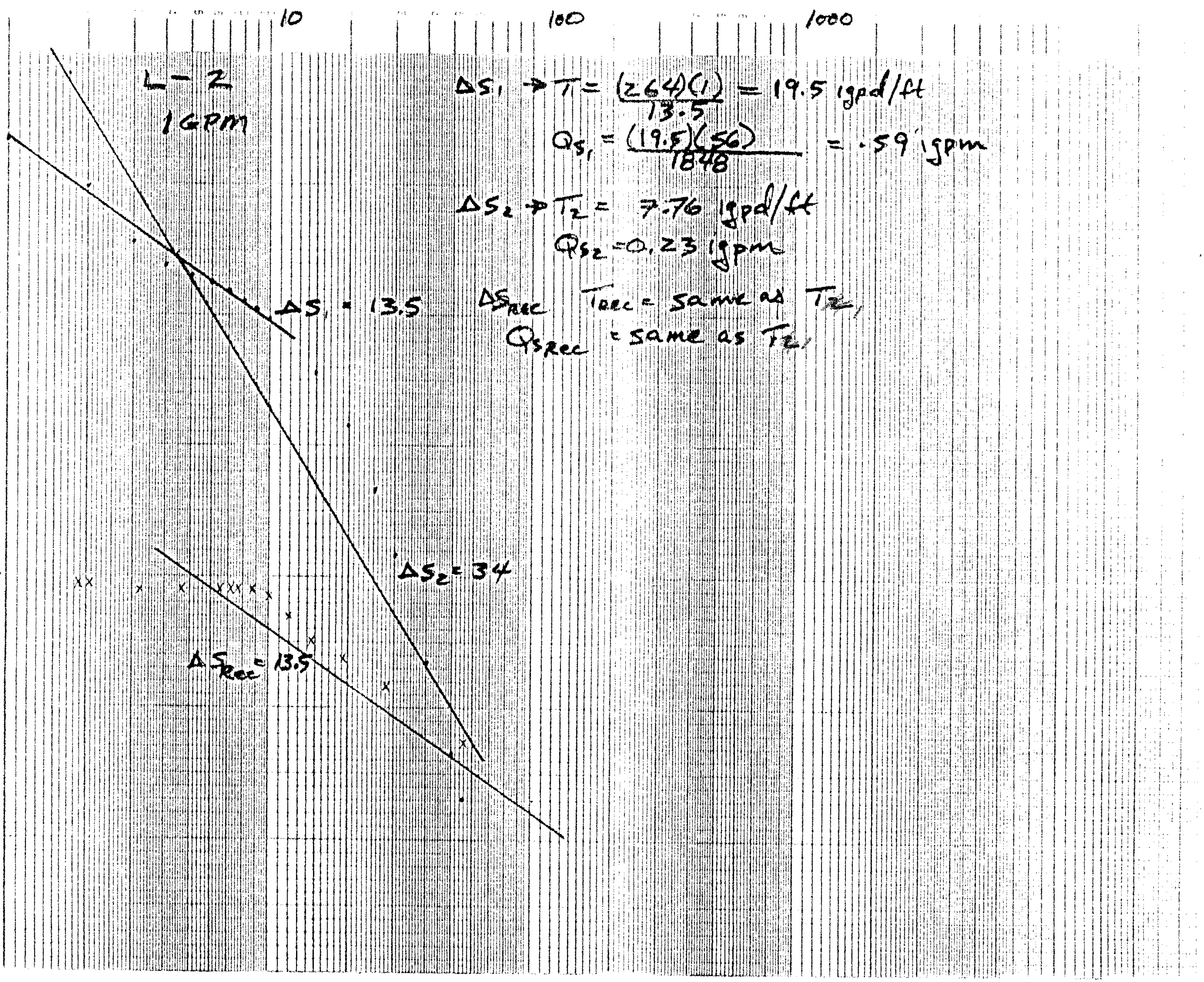
LOCATION OF PROJECT ____________ **WELL LOCATION:** Lawrencetown #2

STATUS Recovery 1 GPM
(pumping or observation well)

R = ____________
(distance from pumping well in feet and direction)

DATE 30/8/77 **PAGE** 2

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			57.0				
		1			52.4				
		2			48.5				
		3			46.8				
		4			45.0				
		5			43.0				
		6			41.8				Constant head tank
		7			41.4				drained back in well
		8			41.4				
		9			41.4				
		10			41.4				
		15			41.6				
		20							
		25			41.6				
		30							
		40							
		50			40.5				
		60			40.6				



$\Delta S_1 \rightarrow T = \frac{(264)(1)}{13.5} = 19.5 \text{ igpd/ft}$

$Q_{S_1} = \frac{(19.5)(56)}{1848} = .59 \text{ igpm}$

$\Delta S_2 \rightarrow T_2 = 7.76 \text{ igpd/ft}$

$Q_{S_2} = 0.23 \text{ igpm}$

ΔS_{rec} T_{rec} = same as T_2

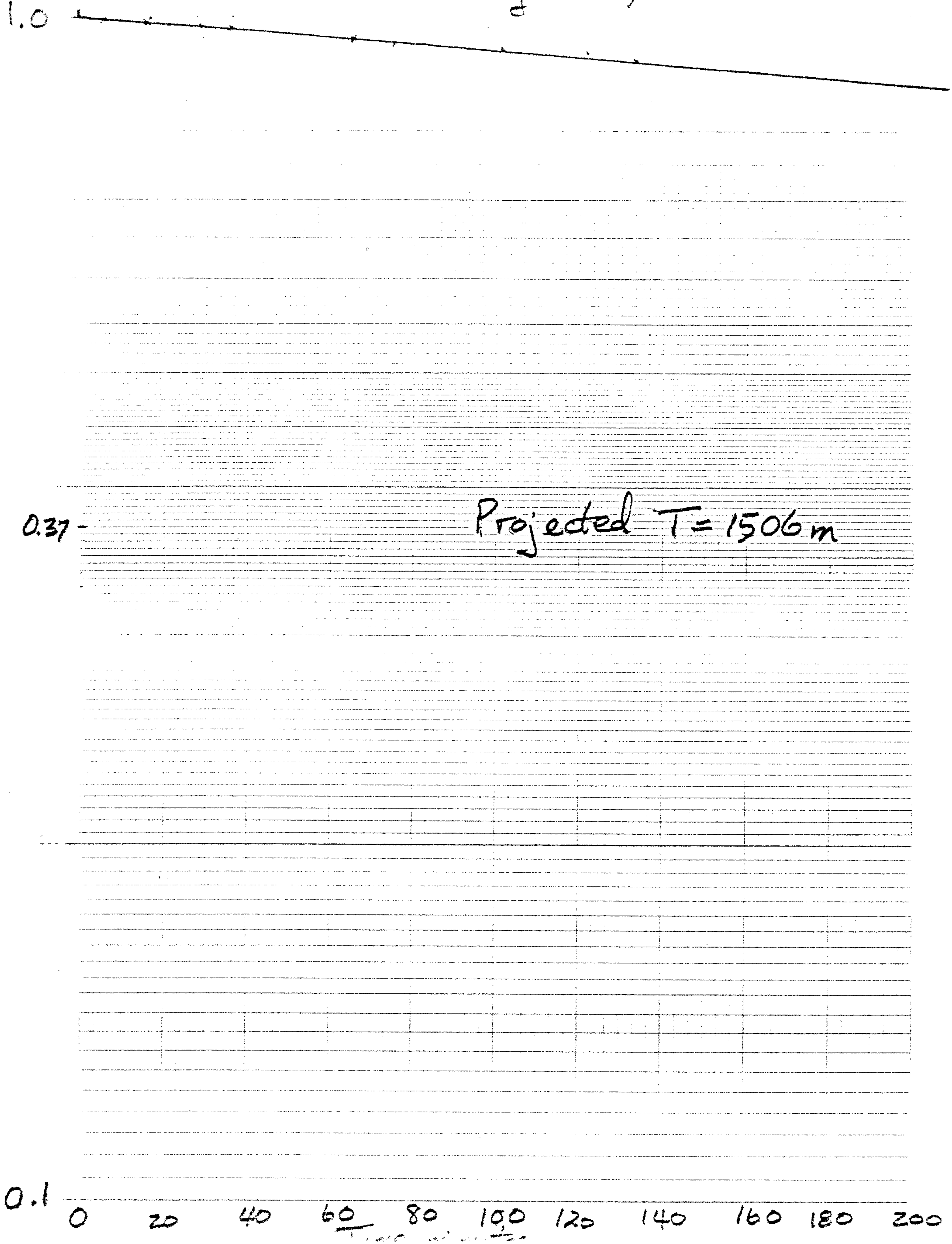
$Q_{S_{rec}}$ = same as T_2

Jan 16/79 SINGLE-WELL OR SINGLE-PIEZOMETER RESPONSE TEST DATA

PIEZOMETER L-2	Ho 3.29	DATE	MEASURED BY

TIME mins	DEPTH TO WATER	H	H/Ho
12:50 0⁻	3.42	3.29	1.00
12:50 0⁺	0.13	3.29	1.00
12:57 7	0.13	3.29	1.00
1:06 16	0.15	3.27	0.99
1:16 26	0.18	3.24	0.985
1:19 29	0.18	3.24	0.985
1:26 36	0.19	3.23	0.98
1:56 66	0.25	3.17	0.96
2:06 76	0.28	3.14	0.95
2:33 103	0.33	3.09	0.94
2:52 122	0.37	3.05	0.93
3:03 133	0.39	3.03	0.92
Jan 17 5:30pm 1720	2.33	1.09	0.33

Falling Head, L-2.



APPENDIX 7

Aquifer Test Data

L-3

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: M. Rushton W Rodgers MEASURED BY: M. Rushton

LOCATION OF PROJECT Lawrencetown WELL LOCATION:

STATUS Pumping 16Pm Well #3 (pumping or observation well) R = (distance from pumping well in feet and direction) DATE PAGE

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			6.7				
		1			8.0				
		2			8.5				
		3			7.8				
		4			7.6				
		5			7.6				
		6			7.6				
		7			7.5				
		8			7.6				
		9			7.5				
		10			7.6				Temp 12°C Cond 325 umho
		15			7.3				
		20			7.3				
		25			7.5				
		30			7.4				Temp 11°C Cond 390 umho
		40			7.5				
		50			7.5				
		60			7.6				
		75			7.6				
		90			7.6				
		100			7.6				

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ______ MEASURED BY: ______

LOCATION OF PROJECT ______ WELL LOCATION: ______

STATUS Recovery Well #3 1GPM
(pumping or observation well)

R = ______ DATE ______ PAGE ______
(distance from pumping well in feet and direction)

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			7.5				
		1			7.0				
		2			7.0				
		3			7.0				
		4			7.0				
		5			7.0				

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ______ MEASURED BY: ______

LOCATION OF PROJECT ______ WELL LOCATION: Lawrencetown Well #3

STATUS Pumping 20GPM (pumping or observation well) R = ______ (distance from pumping well in feet and direction) DATE ______ PAGE ______

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			6.8				
		1			85				
		2			7.8				
		3			7.7				
		4			8.2				
		5			8.1				
		6			8.5				
		7			8.2				
		8			8.2				
		9			8.1				
		10			8.3				Temp 11°C Cond 325 umhos
		15			8.6				
		20			8.7				
		25			9.0				
		30			9.0				
		40			9.5				
		50			9.7				
		60			9.4				
		75			9.7				Temp 11°C Cond 330 umhos
		90			10.0				

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: [illegible] MEASURED BY: ______

LOCATION OF PROJECT ______ WELL LOCATION: ______

STATUS Recovery 26pm
(pumping or observation well)

R = ______ (distance from pumping well in feet and direction) DATE ______ PAGE ______

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at		Depth to water in feet	Draw-down ~~in feet~~ t/t'	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
			Meas. Point	Water level					
		0			10.				
		1			9.1	91			
		2			9.0	46			
		3			8.9	31			
		4			8.8	23			
		5			8.6	19			
		6			8.5	16			
		7			8.5	14			
		8			8.4	12			
		9			8.6	11			
		10			8.5	10			

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ____________ MEASURED BY: ____________

LOCATION OF PROJECT ____________ WELL LOCATION: Lawrencetown Well # 3

STATUS Pumping 40 GPM
(pumping or observation well)

R = ____________
(distance from pumping well in feet and direction)

DATE ______ PAGE ______

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			5.8				
		1			7.5				
		2			7.0				
		3			7.8				
		4			8.0				
		5			8.0				
		6			7.5				
		7			8.3				
		8			8.4				
		9			8.3				
		10			8.2				Temp 10°C Cond 340 umho
		15			9.2				
		20			9.6				
		25			9.7				
		30			9.9				
		40			10.6				
		50			11.1				
		60			11.5				Temp 10°C Cond 335 umho
		75			12.1				
		90			12.6				
		100			12.6				

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ____________ MEASURED BY: ____________

LOCATION OF PROJECT ____________ WELL LOCATION: Lawrencetown

STATUS Recovery Well # 3 416[illegible] R = ____________ DATE ______ PAGE ______
(pumping or observation well) (distance from pumping well in feet and direction)

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at		Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
			Meas. Point	Water level					
						t/t'			
		0			12.6				
		1			11.1	101			
		2			11.0	51			
		3			11.0	34			
		4			10.7	26			
		5			10.7	21			
		6			10.4	17			
		7			10.4	15			
		8			10.3	13			
		9			10.1	12			
		10			10.1	11			
		15			9.4	7.7			
		20			9.3	6			
		25			8.9	5			
		30			8.7	4.3			

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ____________ MEASURED BY: ____________

LOCATION OF PROJECT ____________ WELL LOCATION: Lawrencetown

STATUS Pumping 8GPM (pumping or observation well) R = ____________ (distance from pumping well in feet and direction) DATE ______ PAGE ______

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			8.7				L-1 = 1.15
		1			9.5	0.8			L-2 = 4.55
		2			11.5	2.8			4-4 - 6.6
		3			12.2	3.5			
		4			12.5	3.6			
		5			13.1	4.4			
		6			13.6	4.9			
		7			13.7	5.0			
		8			14.0	5.3			
		9			14.1	5.4			
		10			14.4	5.7			Temp 9°C Cond 345 umho
		15			15.1	6.4			(L-1) - 4.35
		20			15.1	6.4			(L-2) - no change
		25			16.8	8.1			(L-4) - " "
		30			17.0	8.3			Temp 9°C Cond 350 umho.
		40			18.6	9.9			(L-1) - 8.1
		50			19.9	11.2			(L-1) - 9.35
		60			21.0	12.3			(L-1) - 10.4 (L-4) 6.75
		75			21.8	13.1			(L-1) - 11.4 (L-4) - 6.8
		90			22.9	14.2			(L-1) - 12.7
		100			23.4	14.7			

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT - WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: ____________ MEASURED BY: ____________

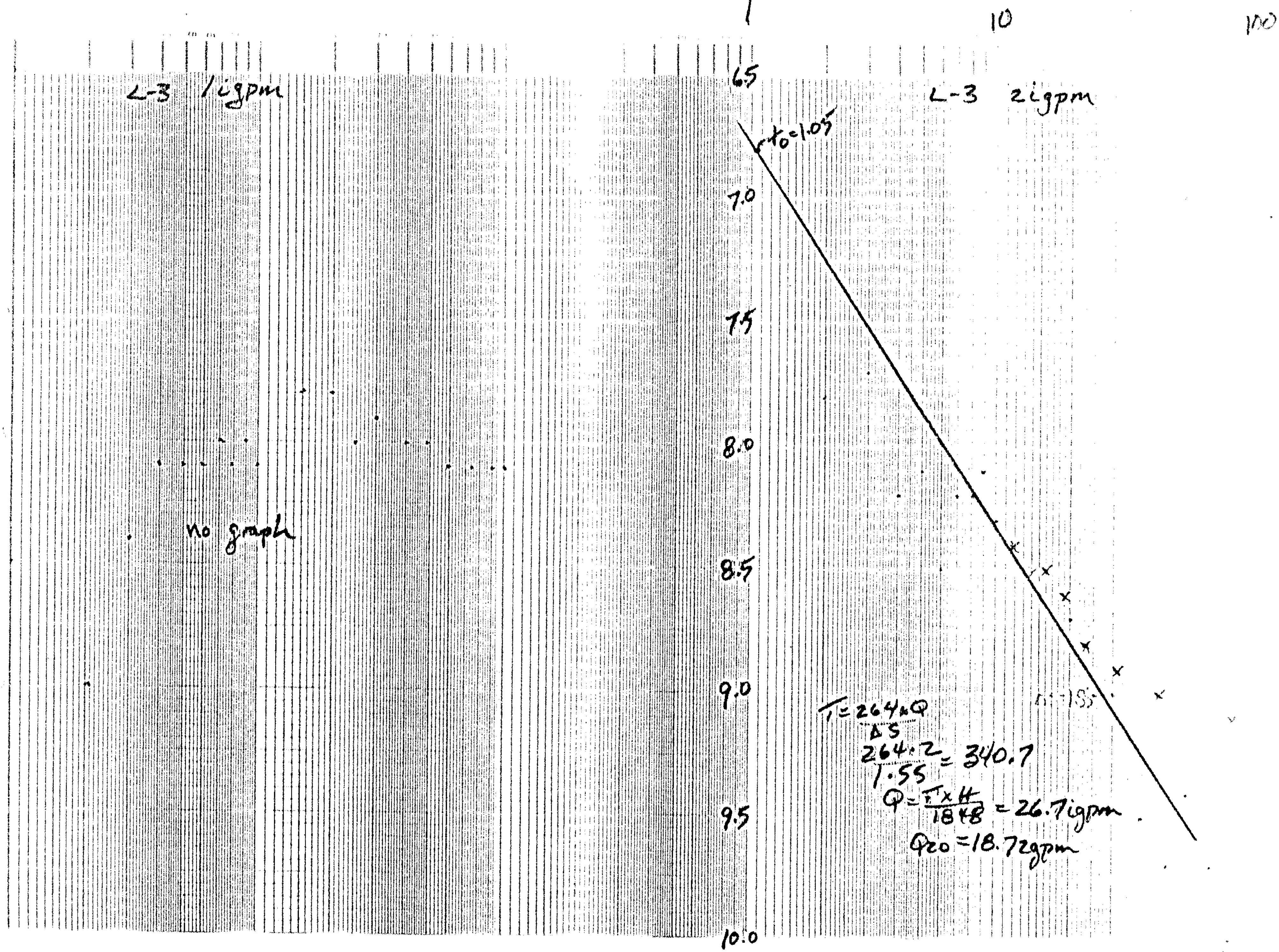
LOCATION OF PROJECT ____________ WELL LOCATION: Lawrencetown

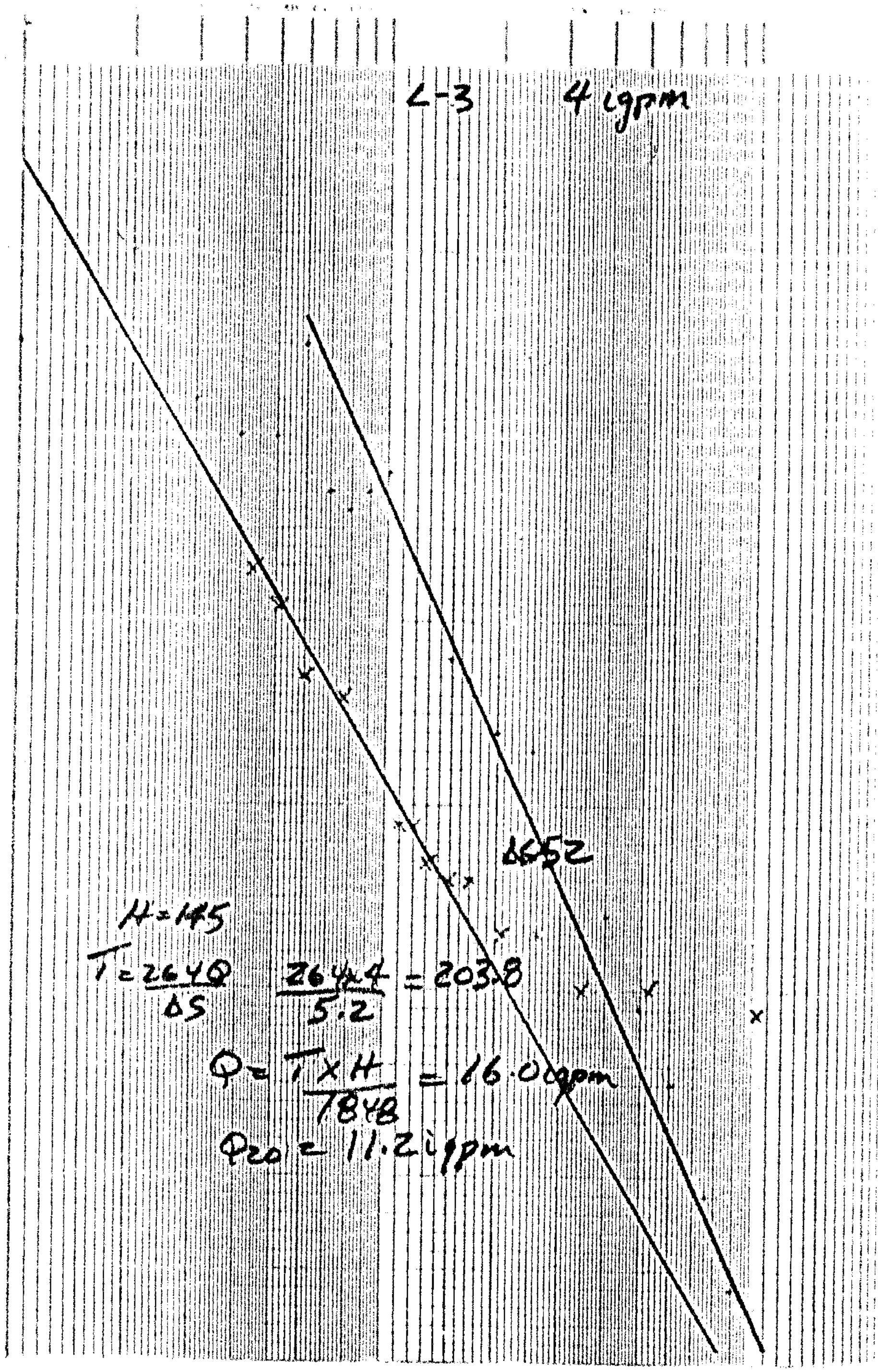
STATUS Recovery 80GPM
(pumping or observation well)

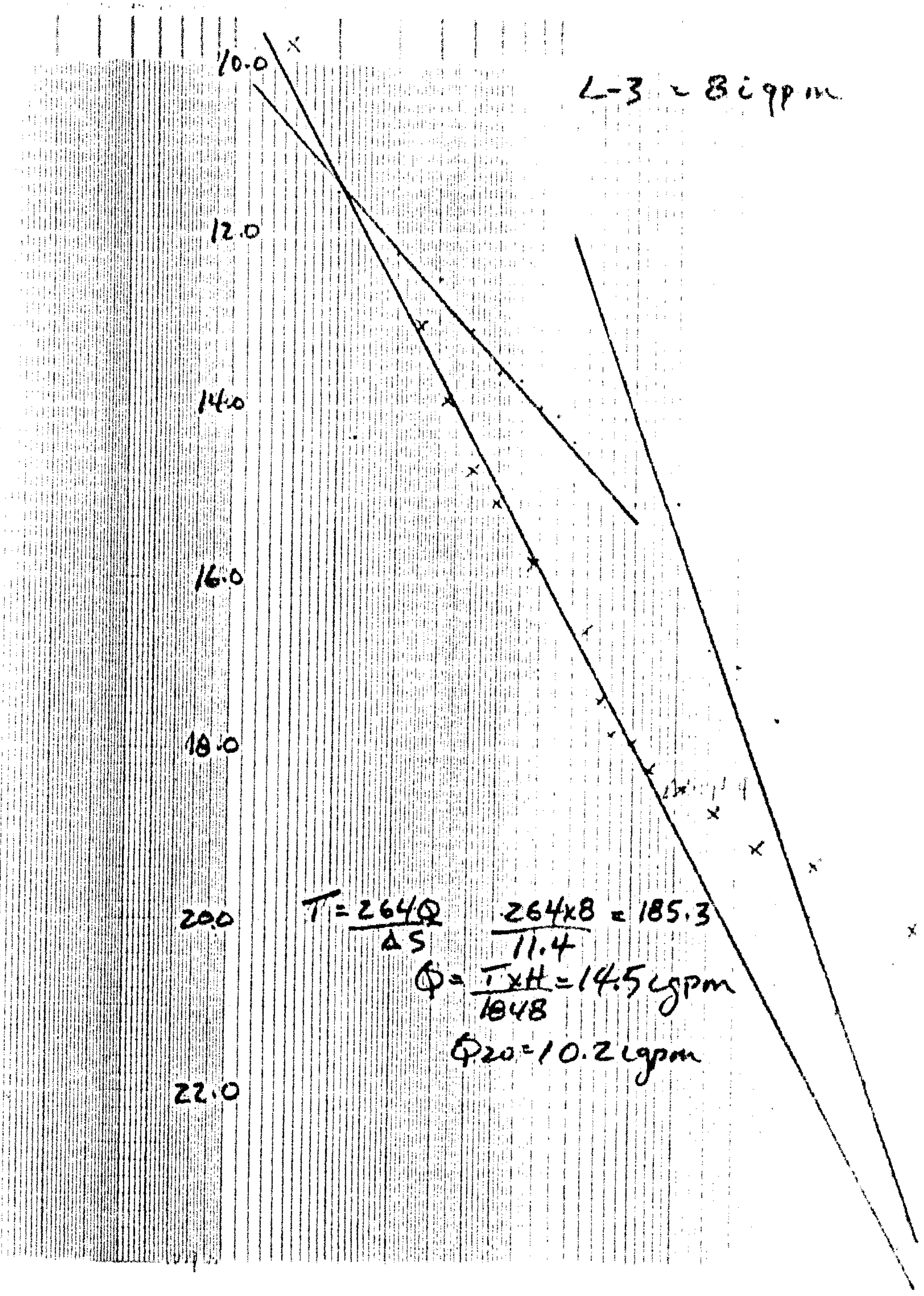
R = ____________
(distance from pumping well in feet and direction)

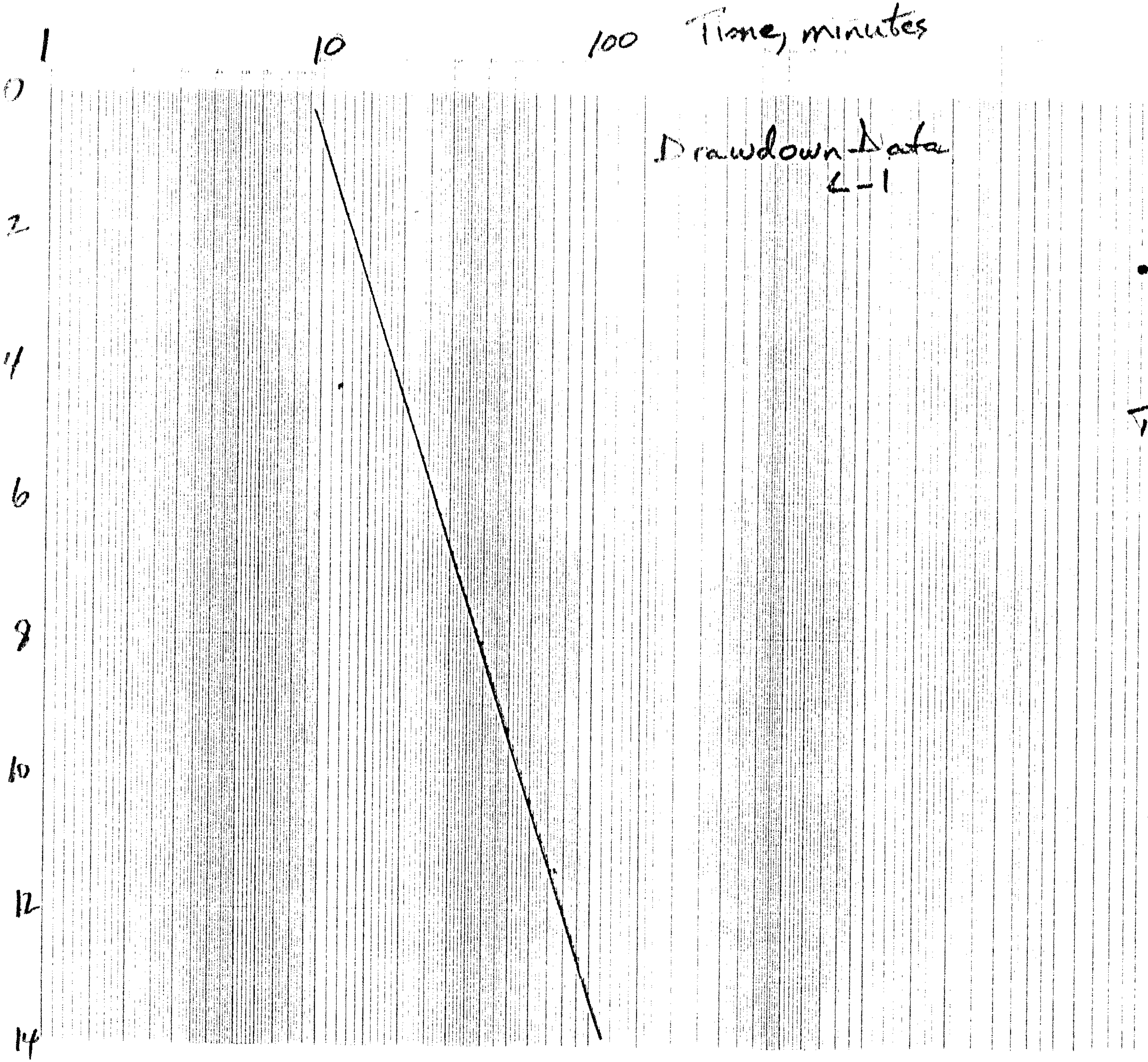
DATE ____________ PAGE ____________

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at: Meas. Point	Tape Reading at: Water level	Depth to water in feet	Draw-down in feet t/t'	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			23.4				
		1			20.0	101			
		2			19.3	51			
		3			19.1	34			
		4			18.7	26			
		5			18.4	21			
		6			18.2	17			
		7			17.9	15			
		8			17.8	13			
		9			17.4	12			
		10			16.6	11			
		15			15.8	7.7			
		20			15.1	6			
		25			14.2	5			
		30			13.9	4.3			
		40			13.0	3.5			
		210			9.8	1.5			


1
10
100
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0
L-3 1igpm
no graph
L-3 2igpm
$t_0 = 1.05$
$T = \frac{264 \times Q}{\Delta s}$
$\frac{264 \cdot 2}{1.55} = 340.7$
$Q = \frac{T \times H}{1848} = 26.7$ igpm
$Q_{20} = 18.72$ gpm


L-3 4 igpm
2652
H=145
T=264Q/ΔS 264x4/5.2 = 203.8
Q= TxH/7848 = 16.0 igpm
Q20 = 11.2 igpm


L-3 ~ 8 igpm
10.0
12.0
14.0
16.0
18.0
20.0
22.0
T=264Q/ΔS 264x8/11.4 = 185.3
Q= TxH/1848 = 14.5 igpm
Q20 = 10.2 igpm



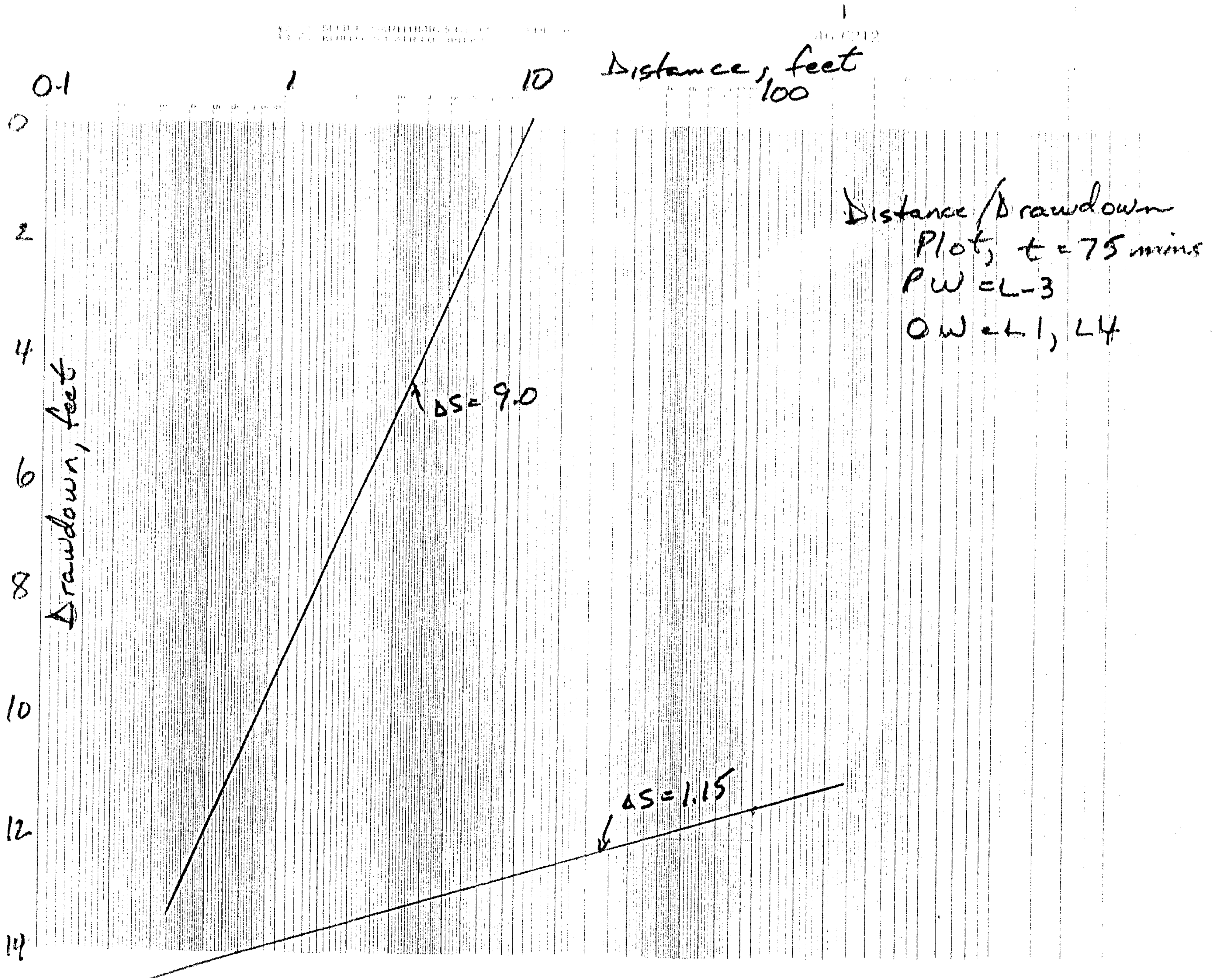
Pumping L-3

• = obs well L-1

$\Delta s = 13.2 - 0.35$
$= 12.85$

$T = \frac{264 \times 8}{12.85}$
$= 164.4$ gpd/ft

$L/s_0 = 1.15'$

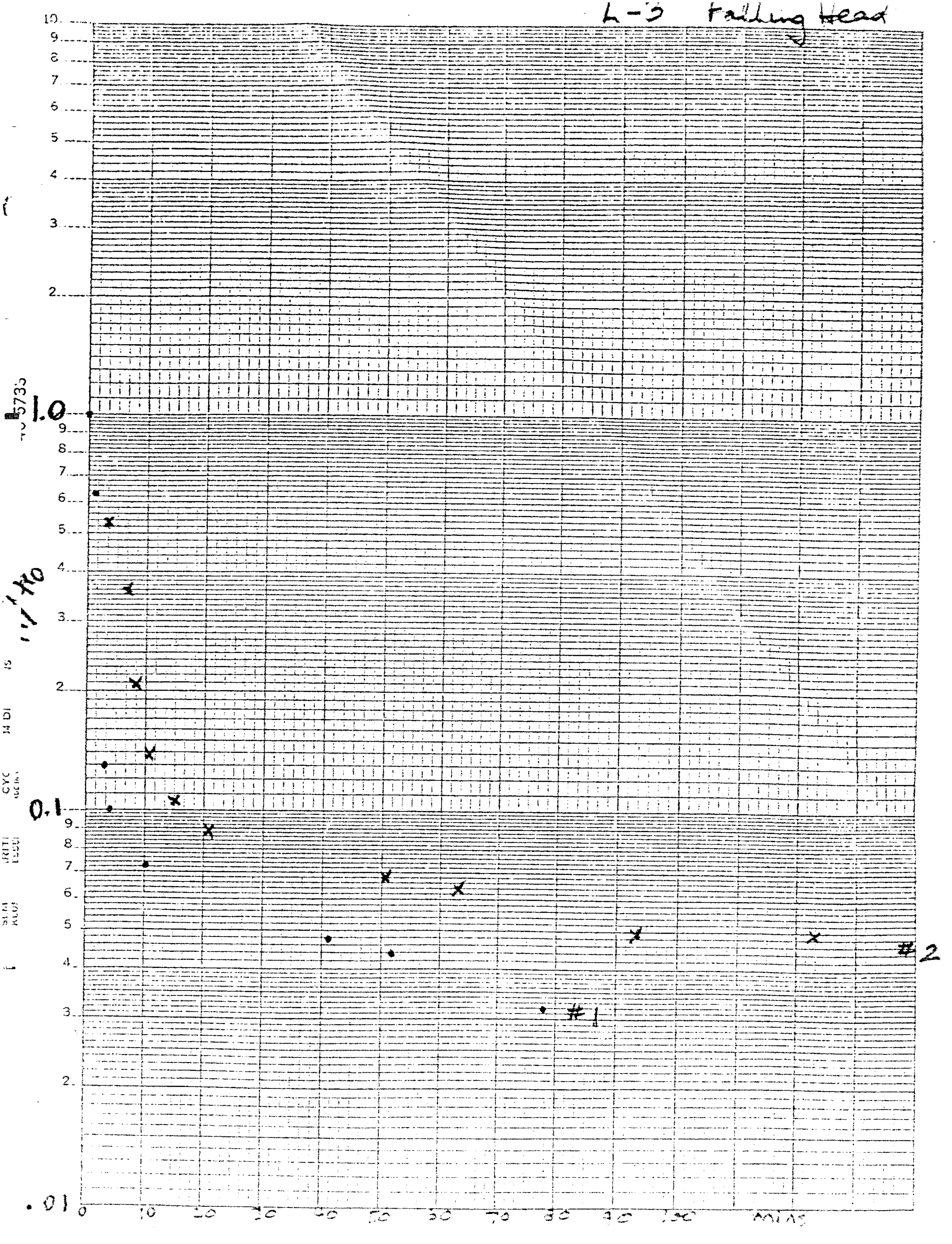


Distance, feet/100
0.1
1
10
Drawdown, feet
0
2
4
6
8
10
12
14
ΔS = 9.0
ΔS = 1.15
Distance/Drawdown Plot, t = 75 mins
PW = L-3
OW = L-1, L4

Jan 16/79 SINGLE-WELL OR SINGLE-PIEZOMETER RESPONSE TEST DATA Tacoma [illegible]

PIEZOMETER L-3	Ho 3.15	DATE	MEASURED BY

TIME	DEPTH TO WATER	H	H/Ho
TEST #1			
1:08 0−	3.48	3.15	1.00
1:09½ 0+	0.33	3.15	1.00
1:10 30s 0.5m	1.42	2.06	0.65
1:10½ 60s 1m	1.50	1.98	0.63
1:11 1.5m	2.90	0.52	0.17
1:11½ 2m	3.02	0.40	0.13
1:12 2.5m	3.13	0.35	0.11
1:12½ 3m	3.17	0.31	0.10
1:13 3.5m	3.18	0.30	0.10
1:14 4.5m	3.21	0.27	0.086
1:15 5.5m	3.21	0.27	0.086
1:20 10.5m	3.25	0.23	[illegible]
1:27 12.5m	[illegible]	[illegible]	[illegible]
1:56 41.5m	3.33	0.15	0.048
2:06 51.5m	3.34	0.14	0.044
2:33 78.5m	3.38	0.10	0.032
TEST #2	H_o =	3.40	
2:44 0−	3.40	3.40	1.00
2:44½ 0+	0.00	3.40	1.00
2:45 30s	1.58	1.82	0.54
2:45½ 60s	2.17	1.23	0.36
2:45¾ 75s	2.67	0.73	0.21
2:46 90s	2.83	0.57	0.17
2:46¼ 105s	2.92	0.48	0.14
2:46½ 120s	2.98	0.42	0.124
2:47 150s	3.04	[illegible]	0.106
2:47½ 180s	3.08	0.32	0.094
2:48 210s	3.10	0.30	0.088
2:49 270s	3.13	[illegible]	0.079
2:50 330s	[illegible]	[illegible]	[illegible]
[illegible]	3.17	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]		



L-3 Falling Head
1.0
0.1
.01
H/H0
#1
#2
0
10
MINS

APPENDIX 8

Aquifer Test Data

L-4

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT – WATER PLANNING & MANAGEMENT DIVISION

WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: Mike Rushton, M Ralzu MEASURED BY: Mike Rushton

LOCATION OF PROJECT Lawrencetown WELL LOCATION: Well #4

STATUS Pumping (pumping or observation well) R ___ (distance from pumping well in feet and direction) DATE 29/8/77 PAGE 1

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			148				
		1			181				
		2			186				
		3			195				
		4			20.2				
		5			21.1				
		6			220				
		7			226				
		8			233				
		9			241				
		10			249				Temp 18°C Cond ~~455~~ 355 umho
		15			291				
		20			329				
		25			36.8				
		30			39.9				Temp 18°C Cond ~~495~~ 345 umho
		40			47.7				
		50			52.0+				Probe caught in [illegible] connector

NOVA SCOTIA DEPARTMENT OF THE ENVIRONMENT – WATER PLANNING & MANAGEMENT DIVISION

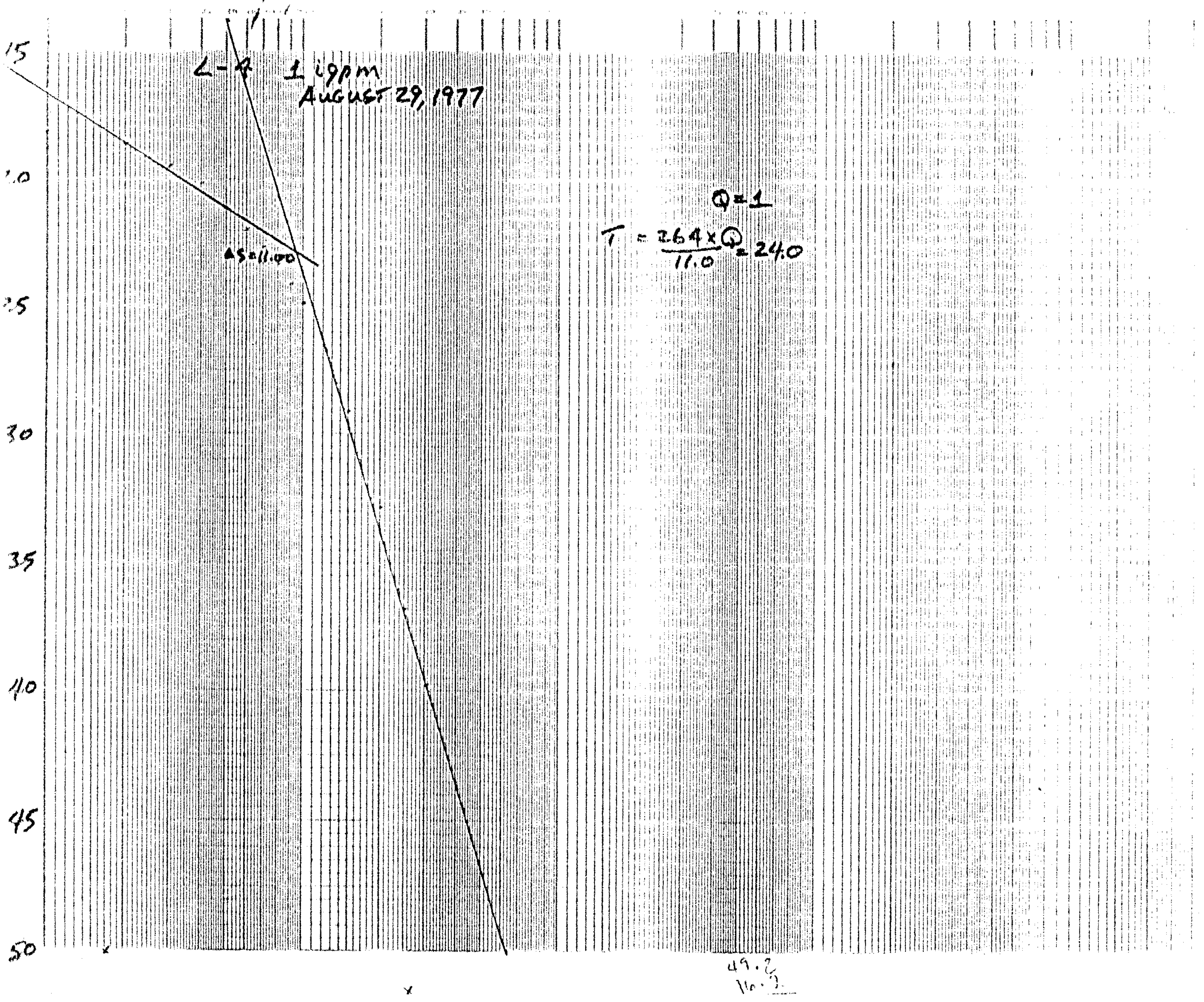
WATER LEVEL MEASUREMENTS (FIELD)

TEST CONDUCTED BY: M. Rushton M. Rodgers **MEASURED BY:** Mike Rushton

LOCATION OF PROJECT Lawrencetown **WELL LOCATION:** Well # 4

STATUS Recovery (pumping or observation well) **R** = (distance from pumping well in feet and direction) **DATE** 30/8/77 **PAGE**

Date	Time hrs. & mins.	Elapsed time in mins.	Tape Reading at Meas. Point	Tape Reading at Water level	Depth to water in feet	Draw-down in feet	Q = discharge gals/min		REMARKS (i.e. pump adjustments, water temp. static levels, etc.)
		0			52.1	t/t'			
		1.			52.1	51			
		2			52.9	26			
		20			52.6	3.6			
		25			52.5	3.0			
		30			52.4	2.7			
		40			52.1	2.2			
		50			51.9	2.0			
		60			51.3	1.8			
		75			50.3	1.7			



L-4 1 igpm
AUGUST 29, 1977
Q = 1
$T = \frac{264 \times Q}{11.0} = 24.0$
ΔS = 11.00
15
20
25
30
35
40
45
50
49.2
16.2

COLE HARBOUR (LOW TIDE)

Date sampled: 1 August, 1971

Station #	1	2	3	4	5
Sodium	-	-	-	-	-
Potassium	335	348	348	357	353
Manganese	-	-	-	-	-
Magnesium	-	-	-	-	-
Calcium	-	-	-	-	-
Iron	-	-	-	-	-
C.O.D.	56	52	38	58	72
Chloride	15,500	15,500	15,500	15,500	15,750
Sulfate	2,200	2,000	2,040	2,120	2,040
Ammonia Nitrogen	-	-	-	-	-
Nitrite Nitrogen	-	-	-	-	-
Nitrate Nitrogen	.205	.59	.5	.45	.435
Phosphate	.163	.237	0.1	0.2	.163

Date sampled: 1 August, 1971

Station#	1	2	3	4	5	6	7	8	9	10
Sodium	-	-	-	-	-	7.08	6.37	5.32	5.67	2.86
Potassium	362	371	362	344	348	0.2	0.2	0.1	0.06	0.06
Manganese	-	-	-	-		-	-	-	-	-
Magnesium	-	-	-	-		0.16	0.89	1.04	1.07	0.56
Calcium	-	-	-	-		5.00	2.58	2.76	1.61	1.00
Iron	-	-	-	-		-	-	-	-	-
C.O.D.	50	48	54	42	52	14	4.9	7.5	3.0	3.2
Chloride	16,000	17,000	17,000	17,000	17,000	20	12.5	12.5	7.5	7.5
Sulfate	2,080	2,160	2,200	2,360	2,240	12	9	5	3	5
Ammonia Nitrogen	-	-	-	-	-	.1	.1	.01	<.01	.125
Nitrite Nitrogen	-	-	-	-	-	.000	.004	.003	.000	.000
Nitrate Nitrogen	.54	.424	.475	.445	.42	1.25	.29	.65	.28	.20
Phosphate	0.237	<0.05	0.09	0.05	0.05	<0.05	0.05	<0.05	0.2	<0.05

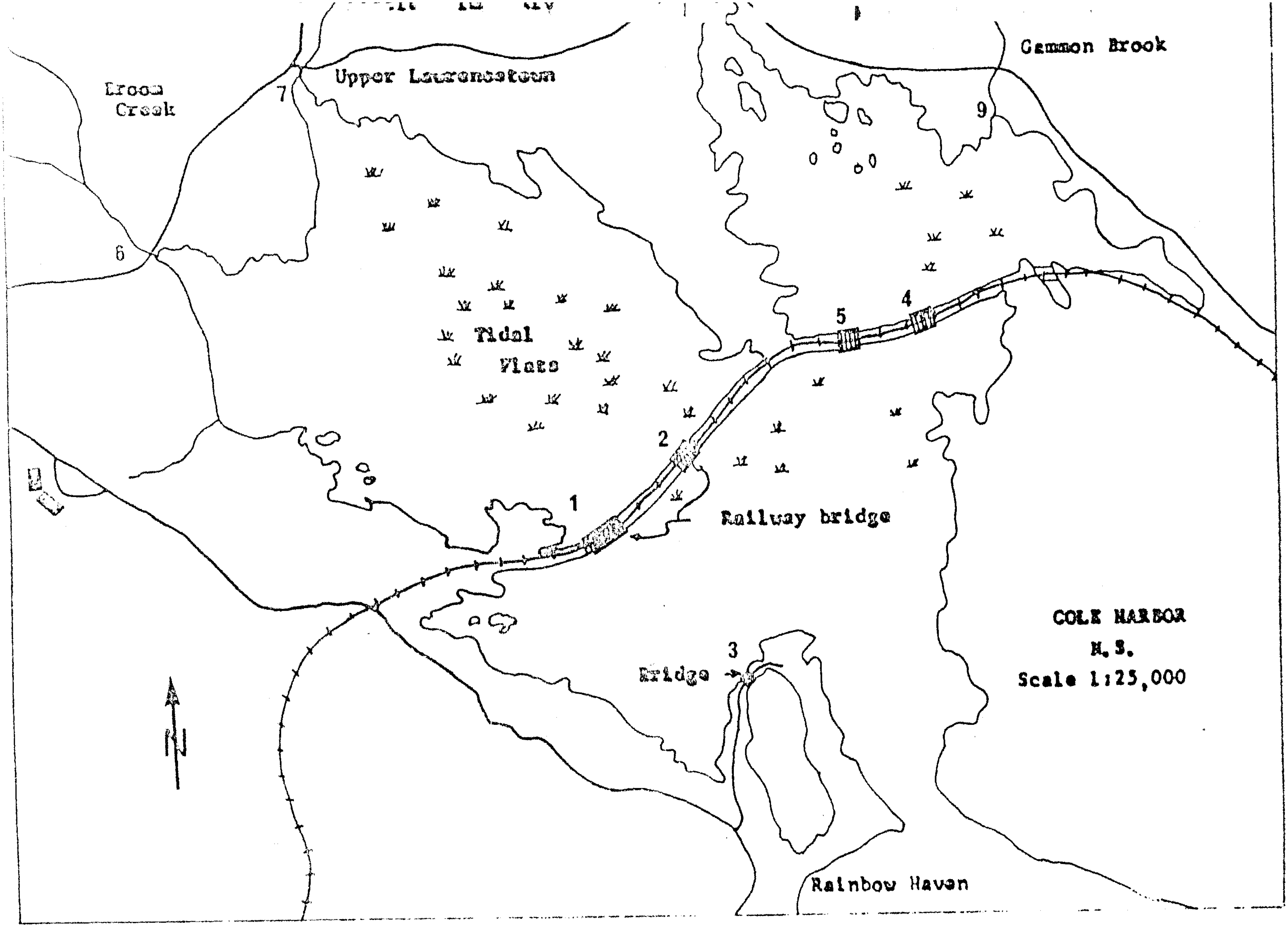

Gammon Brook
Upper Lawrencetown
Creek
7
6
9
Tidal
Flats
1
2
5
4
Railway bridge
3
Bridge
COLE HARBOR
N.S.
Scale 1:25,000
Rainbow Haven
N

TABLE I

Temperature Data,
Cole Harbor
August 1, 1971

Station	Station No.	Code	Time	Tide	Temperature °C.
Hospital sewer outlet	1	CH1		Low	22.5
R.R.Bridge West Arm	2	CH2		Low	22.0
Rainbow Haven	3	CH3	11:20	Low	
R.R.Bridge (far east)	4	CH4	12:10	Low	21.2
R.R.Bridge (east)	5	CH5	13:00	Low	21.4
Hospital sewer outlet	1	CH1	14:45	High	25 *
R.R.Bridge West Arm	2	CH2	15:35	High	17.7
Rainbow Haven	3	CH3	15:55	High	18.0
R.R.Bridge (far east)	4	CH4	16:20	High	18.6
R.R.Bridge (east)	5	CH5	16.45	High	18.1

* This measurement was taken at the very beginning of the high tide so that this station may not have been influenced by the influx of colder ocean water. The chloride measurements seem to show that only partial mixing has occurred.

will result in continuing development of salt marsh and eventually meadow in the upper part of the basin, as salt water access is progressively reduced due to sedimentation in tidal channels.

demand values shown in the table. Station number 7 includes materials brought down the Little Salmon River whose volume of approximately 47 x 10^6 Imperial gallons per day indicates that substantial quantities of nitrogen and phosphorus enter Cole Harbor from this source. It is to be noted that the additions to Little Salmon River occur between the outlet of Lake Major (sample 10 in this table) and the Upper Lawrencetown Bridge (sample 7).

Previous studies (see report to Core Organization, January 1971) have indicated that the principal contribution to the Salmon River comes from Cherry Brook, just north of the Number 7 highway. Sample 8 comes from the outlet of Robinson Brook, and sample 9 from the outlet of Gammon Brook on the eastern arm of Cole Harbor. Both of these sources have relatively small watersheds and show little effects of residential development at the present time.

Summary of Summer Survey Results In Cole Harbor

Tidal flushing studies in Cole Harbor during the summer confirm results reported by J. L. Warner in January of 1971. With approximately 10% replacement of water in the basin with each tide cycle, the residence time of materials introduced into both the western and eastern arm of Cole Harbor may be expected to exceed one week. Since tidal exchange in the upper basins of Cole Harbor is restricted by the railway causeway, it may be expected that rapid eutrophication

The average velocity of water moving into the eastern arm of Cole Harbor is .6 meters per second with a maximum value of .88 meters per second reached early in the rising tide cycle. The average velocity in the western arm of Cole Harbor is 1.16 meters per second with a maximum value of 1.5 meters per second, again achieved in the first quarter of the tide cycle.

After plotting current velocity versus time, areas under the curve were integrated to achieve an estimated total tidal flow volume for both arms of Cole Harbor. Approximately 140 x 10^6 cubic feet of water entered the western arm per tidal cycle and 97 x 10^6 cubic feet entered the eastern arm in each tide cycle.

Tables 2 and 3 detail water chemistry studies made on samples from each of the stations located in Cole Harbor. Stations 1 through 5 are marine stations, as indicated on the accompanying map (fig. 1). Normal sea water has a chloride concentration of about 17,200 parts per million of chloride and about 2400 parts per million of sulfate. Comparison of chloride values at the same stations indicate an approximate 10% dilution of the tidal water by fresh water coming in from streams in the vicinity of Cole Harbor.

Stations 6 through 10 define contributions from the fresh water sources entering Cole Harbor. Station number 6 is near the mouth of Broom Creek in the western arm of Cole Harbor. A small treatment plant located near the head of the Broom Creek watershed plus a few homes and trailers contribute to the nitrogen values and the chemical oxygen

SUMMER WATER CHEMISTRY AND FLUSHING TIME - COLE HARBOR, N.S.

A preliminary report on Cole Harbor was submitted to the core organization of the Metropolitan Area Planning Committee by the Task Group on Water Supply and Waste Disposal on 13 January, 1971, entitled, "Effects of Existing and Potential Sources of Pollution - Cole Harbor". To supplement the winter data presented in the report, five marine and five fresh water sample stations were occupied at Cole Harbor through one tide cycle on 1 August, 1971. Water chemistry samples were collected at low tide and at high tide. Current velocity was determined during a rising and a falling tide on the 24 August, 1971. The measurements were taken at the bridges on each arm of Cole Harbor. Location of the sampling stations is indicated on the accompanying map. Stations 2 and 5 were sites of tidal current measurements.

Temperature measurements in Cole Harbor for 1 August, 1971 are reported in Table 1.

Planimetric measurement of the high water shorelines indicate that there is 1.6 times as much surface area in the western arm of Cole Harbor as in the eastern arm. Current velocities were determined with a Weather Measure, model F581 current meter. Cross-sectional areas of the water beneath the bridges was integrated with current velocity measurement to determine the amount of water entering and leaving the two arms of Cole Harbor.

APPENDIX 9

Chemical Analyses, mg/l. Deeper Groundwater — Testholes & Other Drilled Wells

No.	Description	Na	K	Ca	Mg	Hardness	Alkalinity	SO_4	Cl	F	SiO_2	o-PO_4	NO_3+NO_2 as N	NH_3 as N	TDS	SS	Color (TCU)	Turbidity (JTU)	Conductivity (micromhos)	pH	other
1	L1 - sample during blow test	225	16	38	10	138	55	1.0	443	0.2	4.4	0.03	<0.1	0.1	860		15	45	1500	8.5	Tannin/lignin <0.
1	L1 - 5 mins during pump test	230	4.7	39	14	156	53	<1	440	0.1	3.3	0.05	<0.1	<0.1	821		10	1.3	1450	7.7	TOC 1.0 tannin/lignin 0.
1	L1 - 40 mins during pump test	235	4.9	37	14	151	42	<1	455	0.1	2.7	0.08	<0.1	0.1	826		10	1.4	1500	8.0	TOC 1.0 tannin/lignin 0.
3	L3 - sample during blow test	95	3.4	23	7.2	88	76	3.2	160	0.2	8.2	0.04	<0.1	<0.1	362		5	5.5	600	8.4	tannin/lignin 0.
3	L3 - 5 mins during pump test	80	2.4	23	7.8	90	88	5.0	130	0.1	8.2	0.05	<0.1	<0.1	276		10	1.7	600	7.8	TOC 1.0 tannin/lignin 0.
3	L3 - 60 mins during pump test	82	2.4	23	8.0	91	86	3.0	140	0.1	8.5	0.04	0.2	<0.1	338		5	0.9	575	7.8	TOC 2.0 tannin/lignin 0.
	Taylor - West Lawr.	20	1.9	92	7.9	263	155	31	94	<0.1	13	<0.02	<0.05	0.53	374	9.3	10	130	600	6.9	TOC 4.0 Humic acid 4.5
	Campbell - West Lawr.	42	3.8	66	7.8	198	72	21	155	<0.1	6.3	<0.02	2.4	0.10	390	3.0	35	15	650	6.5	
	Tibbet - Upper Lawr.	175	12	508	110	1730	64	70	1372	0.1	9.8	0.02	<0.1	<0.1			10	0.8	4212	6.9	
	Maynard - Jollimore			616	187	2320	43	166	2000				<0.1	0.4						6.9	
	Pollett - Bedford	77	2.1	90	11	270	34	35	250	0.1	8.0	<0.02	0.9	0.2	583		5	0.5	850	6.9	
	McFadden - Glenhaven	193	5.3	119	30	423	62	58	490	1.3	18	0.02	<0.1	<0.1	1046		5	2.5	1750	6.8	
	Humphries - Indian Pt	120	19	85	50	423	98	43	388	0.4	9.6	0.02	<0.1	<0.1	902		30	2.8	1200	7.3	
	Cook - Indian Pt	224	9.2	122	46	497	91	68	547	1.0	12	<0.02	<0.1	<0.1	1278		40	5.7	1945	7.4	
	Publicover - W. Dover	685	30	86	93	601	26	173	415	0.2	9.3	<0.02	4.0	<0.1	3060		15	1.2	4500	6.3	
	Stewart - Allen Hgts	92	18	348	211	1750	55	74	1350	0.4	13	0.09	<0.1	2.15	2614		15	130	4400	6.2	
	Findlay - Sambro	385	8.6	298	71	1040	32	180	1200	0.5	11	0.02	0.1	0.1	2410		20	70	3980	6.1	
	Wells - Allen Hgts	32	4.6	78	17	267	86	8.0	175	0.5	15	0.7	<0.05	1.07	409	15	15	30	700	7.6	
	McWhorter - Bedford	97	4.0	256	46	835	92	64	660	0.1	12	<0.02	<0.05	0.06	1218	[illegible]	40	0.35	2400	7.8	

Chemical Analyses, mg/l.

Surface Water and Shallow Groundwater.

No.	Description	Na	K	Ca	Mg	Hardness	Alkalinity	SO_4	Cl	F	SiO_2	o-PO_4	NO_3+NO_2 as N	NH_3 as N	TDS	SS	Color (TCU)	Turbidity (JTU)	Conductivity (micromhos)	pH	other
1	Little Salmon River	11	0.7	5.6	1.8	21	11	9	18	<0.1	2.2	<0.02	<0.1	<0.1	40		20	2.3	105	7.1	TOC 3.0 Temp 16.2°C Sept. 2/77
2	Estuary near test holes	8500	405	353	1098	5475	130	2250	15900	1.9	<1	0.06	<0.1	0.1	29810		20	3.8	48,000	7.1	TOC 3.5 Temp 18°C
3	Estuary off Park-Crest Drive	8500	420	378	1098	5600	140	2290	15900	2.0	<1	0.21	0.4	0.2	30940		15	1.8	50,000	6.8	TOC 5.5 Temp 17°C
4	Estuary off Lyla Drive	8950	410	366	1122	5590	135	2380	16350	2.0	<1	0.13	<0.1	0.1	36880		25	2.4	52,000	6.9	TOC 5.5 Temp 18°C
5	Estuary near Meadow Haven	8925	430	382	1170	5830	140	2460	16880	2.2	<1	0.19	<0.1	0.1	31640		15	0.7	50,000	8.0	TOC 3.0 Temp 16°C
6	Estuary near Brown Creek	8925	390	364	1122	5585	133	2350	16350	2.0	<1	0.18	0.11	0.1	30240		10	1.3	46,400	8.1	TOC 9.5 Temp 11°C
	Sea Water (Hem)	10,500	380	400	1350	6551	117	2700	19000	1.3	6.4	0.21	2.2							8.1	
	B Patterson dug well	25	1.5	11.2	1.0	32	22	12	41	<0.1	4.3	0.07	1.0	<0.1	139		40	0.6	200	6.4	Temp 44°F May 16/77
L1	L1-1 well depth 50', csg 6.	117	17	24	12	110	51	35	225	0.2	4.2	0.05	<0.1	<0.1	481		25	8.5	860	7.5	Tannin/lignin 0.5 mg/l
L2	L2-1(20')-Karm lake								165											8.2	
L2	L2 2(60')-Karm Lake	108	57	24	8.5	96	43	57	200	0.1	8.5	0.14	<0.1	<0.1	506		10	27	900	8.7	Temp 47.5°F Mar 17/77
L2	L2 5min during pumping	87	20	30	11	118	50	22	186	0.2	1.8	<0.02	<0.1	<0.1	406		10	26	750	7.6	TOC 3.5 Temp 12°C
L2	L2-30mins during pumping	95	19	31	12	126	53	24	206	0.2	2.3	<0.02	<0.1	<0.1	442		10	55	750	7.6	TOC 3.0 Temp 12°C
L3	L3-1 well depth 55', csg 10	17	8.4	20	3.5		44	23	29		7.4									8.1	Tannin/lignin 3.0 mg/l
L4	L4-1 (20')-Kandale								112											8.8	
L4	L4 2(60')-Kandale	70	21	11	5.0	49	39	6.0	125	0.1	8.6	0.18	<0.1	0.1	284		10	28	490	9.3	
L4	4-5min during pumping	67	11	22	7.6	87	70	6.8	125	0.1	6.8	0.17	<0.1	<0.1	258		15	0.8	530	7.9	TOC 2.0 tannin/lignin 0.5
L4	-4-20min during pumping	63	5.3	24	7.8	93	73	5.8	122	0.1	6.4	0.12	<0.1	<0.1	268		15	0.5	520	8.0	TOC 4.5 tannin/lignin 0.5

APPENDIX 10

Metal Scans, mg/l

[illegible]	Al	As	B	Ba	Be	Cd	Co	Cr	Cu	Fe	Mn	Ni	Pb	Sb	Se	Sn	V	Zn	Comments
1	<0.05	Tr	<0.02	0.005	Tr	Tr	<0.01	<0.01	Tr	0.11	0.02	<0.02	Tr	<0.05	<0.10	<0.03	<0.01	<0.01	
2	0.16	Tr	3.2	0.009	Tr	Tr	0.02	0.06	0.01	0.14	0.11	0.14	Tr	0.10	0.30	0.19	0.03	0.05	
3	0.12	Tr	3.4	0.008	Tr	Tr	0.02	0.06	0.02	0.09	0.05	0.14	Tr	0.10	0.30	0.18	0.03	0.08	
4	0.10	Tr	3.4	0.006	Tr	Tr	0.01	0.05	Tr	0.08	0.06	0.14	Tr	0.09	0.29	0.20	0.02	0.06	
5	0.11	0.02	3.5	0.005	Tr	Tr	0.02	0.06	Tr	0.09	0.04	0.14	Tr	0.10	0.30	0.19	0.03	0.04	
6	0.10	Tr	3.5	0.005	Tr	Tr	0.01	0.05	0.007	0.09	0.05	0.14	Tr	0.09	0.29	0.18	0.02	0.05	
[illegible]	0.01	0.003	4.6	0.03	Tr	Tr	Tr	Tr	Tr	0.01	0.002	0.002	Tr	Tr	0.004	0.003	0.002	0.01	
[illegible]		Tr							0.01	0.03	<0.01		Tr					0.11	
1-1(60')	0.23	0.01	0.04	0.78	Tr	0.02	0.03	0.01	0.04	0.3	0.9	<0.02	0.008	<0.05	<0.1	<0.03	<0.01	0.03	
1-2(60')		0.16							0.02	0.5	0.05		Tr					0.005	
2-5min	<0.05	Tr	0.05	0.02	Tr	Tr	<0.01	<0.01	Tr	0.01	0.15	<0.02	Tr	<0.05	<0.10	0.04	<0.01	<0.01	
2-60mins	<0.05	Tr	0.06	0.02	Tr	Tr	<0.01	<0.01	Tr	0.09	0.13	<0.02	Tr	<0.05	<0.10	<0.03	<0.01	0.009	
3-1(15')		0.03								0.2	0.2								
4-2(60')		0.16							0.02	0.5	<0.05		0.005					0.005	
4-5mins	<0.05	0.14	0.06	0.02	Tr	Tr	<0.01	<0.01	Tr	0.09	0.03	<0.02	Tr	<0.05	<0.10	<0.03	<0.01	<0.01	
1-20min	<0.05	0.10	0.05	0.01	Tr	Tr	<0.01	<0.01	0.04	0.22	0.03	<0.02	0.02	<0.05	<0.10	<0.03	<0.01	<0.01	