MUNICIPALITY OF IQALUIT

WATER TREATMENT PLANT

PRE-DESIGN BRIEF

Prepared for:

City of Iqaluit
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Earth Tech Project No. 49,745

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The existing water treatment plant has a useful output of 1,050 m^3/day, which is currently less than the average day demand of 1,200 m^3/day, and the throughput of the existing plant is limited by the capacity of the filters.

The water treatment plant has been in need of an upgrade for several years and this continues to be the case. The excellent of the raw water quality of the Lake Geraldine source has permitted the City of Iqaluit to overcome the operational shortfall of the water treatment plant in its current configuration, but not without risk.

Based on a current design year of 2022, available population demands and projections, the upgraded water treatment plant will need to have a production capability of approximately 9,500 m^3/day in the design year. For the most part, this can be achieved utilizing the existing infrastructure, along with new, increased capacity filters, contained in a new building addition above an existing clearwell. However, shortfalls exist in both the existing intake and the recharge capability of Lake Geraldine, as described below.

The best information available at the time of this report, from OMM/Trow indicates, that the Lake Geraldine reservoir has an annual recharge volume of approximately 586,000 m^3. Based on this information, there may be a potential raw water shortfall situation in the year 2005. Consequently, additional raw water may need to be transferred into the Lake Geraldine reservoir from an alternate raw water source. This transfer has potential treatment process implications, however , we feel these are limited, given that the alternate raw water being transferred to Lake Geraldine should be of similar quality. This needs to be investigated and confirmed.

The existing raw water intake from the Lake Geraldine dam structure to the water treatment plant is approaching the end of its useful life. Based on current population and demand projections, the intake may be unable to meet the maximum day requirements of 4,000 m^3/day in approximately the year 2001/02. It is recommended that the intake be replaced with a 355 mm high density polyethylene insulated pipe, complete with a matched tempering system, similar to that currently in use. At the time of this report, NTPC has no desire to use Lake Geraldine as a cooling water source, as was done in the past.

This intake will have a projected lifespan well in excess of 20 years. Exactly how long this intake would be able to serve the City's requirements is difficult to determine, given the population projections and the demand data are not available beyond 20 years.

Potable water storage capacity is barely sufficient, and at projected population and demand increases within a couple of years additional equalization and emergency storage will need to be added. Additional storage volumes required, for disinfection, can be mitigated by increasing the chlorine dosage rate however, equalization and emergency storage are functions of demand. At present, the available on-hand potable water storage capacity is equal to slightly less than two (2) average day's demand. It is recommended that the City of Iqaluit start the funding process for adding a second cell to the main potable water storage reservoir.

Estimated total cost, including engineering and contingencies for the water treatment plant upgrade only, not including an additional cell to the potable water storage reservoir, is approximately $4,000,000.00. An aggressive schedule may permit the design during the winter of 2002, with construction starting after the 2002 sea lift, and final completion in approximately July 2003.

It is anticipated, that during construction a temporary water treatment plant will need to be provided as the old facility will need to be taken off line to permit the required modifications.

1.0 INTRODUCTION

1.1 Background

Earth Tech (Canada) Inc.'s, formerly Reid Crowther and Partners Ltd., involvement in the water treatment plant (WTP) upgrade began in 1989 when Reid Crowther conducted an assessment of present and future water demands for the City of Iqaluit. In 1991 the first draft of the Planning Brief was released, but the project was put on hold pending a decision on the funding level for fire storage. Following the fire storage decision funding, the City requested the planning brief be updated and the third draft was released in January 1994.

The third draft of the planning brief recommended changes to both the water treatment plant and the treated water storage. Subsequent to the planning brief the City elected to proceed with treated water storage expansion, and construction was completed in the fall of 1996.

In January 1998, the City again retained Reid Crowther to review the recommendations of the portion of the 1994 planning brief dealing with the water treatment plant upgrade and to prepare a predesign report. The 1998 predesign report was to consider the impact of Nunavut, the impact of which was specifically excluded from the earlier Planning Briefs. Shortly after the release of the 1998 Predesign report, this project was put on hold. In the spring, the City of Iqaluit contracted Earth Tech (Canada) Inc. to update the 1998 report to reflect current population projections and demand data. This report details the current predesign phase of the project.

Existing Facilities

The City is supplied with raw water from Lake Geraldine. The source is generally of good quality, for those parameters recorded. Water from the lake flows by gravity through a 360m long 250mm diameter ductile iron pipeline to the treatment plant. The intake line is poorly insulated with a blown glass material and is protected from freezing by a tempered water recirculation system. The treatment plant has a maximum design output of 1,296 m^3/day and a useful output of 1,050 m^3/day. The throughput of the treatment plant, in its present state, is limited by the capacity of the filters.

The treatment plant is comprised of the following major components:

- the Lake Geraldine dam structure and valve chamber,
- the raw water intake pipeline and tempering system (upgraded in 1999);
- the treatment plant inlet flow control valve,
- the prechlorination and pH control (through lime contact) system,
- the settling tanks,
- the filtration system,
- the flouridation system,
- the backwash system,
- and the main treated water storage reservoir.

The original plant construction had a post filtration ozonation system but this has been abandoned and has yet to be removed.

Treated water is stored in two (2) clearwells in the WTP with a combined capacity of 575 m^3 and in the main storage reservoir which holds 2,280 m^3. From the main reservoir the treated water flows by gravity to the distribution system. The clearwells and the main reservoir operate as one large tank with the same free water surface height of 94.5 m, neglecting pipe losses between the tanks.

The existing plant may contain hazardous materials which will require disposal in a responsible manner:

- asbestos piping insulation was known to exist in the NTPC utilidor, but appears to have been removed;

- asbestos piping insulation may be present on existing plant piping; and

- given the age of the ozonation system, the standard of manufacture of transformers from that time period, and statements made by City employees, the transformer in the ozonation assembly may contain PCB's.

1.2 Water Demands And Quality

Historical Water Demands

Historical water demands and populations, for the period 1978 to 2000, as well as projected values to 2021 are summarized in Table 1.1 and Figure 1.1.

Future Water Demands

It can be seen that since 1987, the demand has been generally declining. This is considered to be attributable to the efforts of the City to reduce leakage and the use of bleeders. As much of this work is now completed, it is anticipated that the average day demands will start to increase again with population growth.

Historically, from 1989 to 1990 the peak demands were recorded to be approximately 2,400 – 2,500 m^3/day, giving a peak day factor of around 1.7, and this is expected to climb to 2.0 as additional system losses are eliminated. From the third draft of the

Planning Brief, the minimum night flows factor is approximately 0.1 and will adopted for this report.

For design purposes, a 20 year horizon has been assumed yielding a design year of 2021. The City provided population projections by another consultant's work, indicating some 11,300 persons and a 400 lpcd usage in 2021. This is an increase of nearly 20% over previous projections utilized.

The 1994 Planning Brief utilized a modified MACA formula to estimate average water demands. This formula was:

Q	=	$F \cdot P \cdot 225$ lpcd $[(-1.0) + 0.323\,(\ln P)]$ (for pop. 2,000 to 10,000)
Q	=	$F \cdot P \cdot 225$ lpcd $\cdot 2$ (for pop >10,000)
Where F	=	allowance for wastage, leaks etc. (1.15)
Where P	=	projected population

This modified MACA formula is based on the entire community being on "piped" water supply system. As a significant portion of Iqaluit's population is on trucked delivery, the MACA formula for piped systems would be expected to yield higher than actual demands.

Table 1.1 clearly shows that for 2000, the City operated at 245 lpcd based on a known population of 4,880. This translates into 55% of what the modified MACA formula would predict and 62% of the 400 lpcd. Table 1.1 and Figure 1.1 both show that, in previous years, the usage had been above both the MACA formula and the 400 lpcd, but a trend towards the actual demand being less than calculated values, has existed for the past ten years.

The 400 lpcd is, we believe, a conservative design demand; provided the piped system is well maintained, bleeders are controlled and, over the design life of the plant, some of the trucked service is converted to piped usage (i.e., such as Lower Base). For comparison, Yellowknife is currently averaging between 425 and 450 lpcd.

Table 1.1 – Historical and Projected Populations and Demands

Year	Pop.	Average Demand	Average Demand	Modified MACA Formula Ave. Day	Actual vs MACA Formula	Municipal 400 lpcd Ave. Day	Actual vs Municipal 400 lpcd
		m^3/day	lpcd	m^3/day	%	m^3/day	%
1978	2,490	932	374	983	95%	996	94%
1979	2,454	1,103	449	966	114%	982	112%
1980	2,475	1,269	513	976	130%	990	128%
1981	2,490	1,145	460	983	116%	996	115%
1982	2,558			1,016		1,023	
1983	2,626			1,048		1,050	
1984	2,694	1,350	501	1,081	125%	1,078	125%
1985	2,763	1,593	577	1,115	143%	1,105	144%
1986	2,947	1,629	553	1,205	135%	1,179	138%
1987	3,057	1,382	452	1,259	110%	1,223	113%
1988	3,171	1,363	430	1,316	104%	1,268	107%
1989	3,298	1,469	445	1,380	106%	1,319	111%
1990	3,419	1,334	390	1,440	93%	1,368	98%
1991	3,552	1,209	340	1,508	80%	1,421	85%
1992	3,694	1,239	335	1,580	78%	1,478	84%
1993	3,842	1,329	346	1,656	80%	1,537	86%
1994	3,996	1,253	314	1,736	72%	1,598	78%
1995	4,156	1,101	265	1,819	61%	1,662	66%
1996	4,220	1,292	306	1,852	70%	1,688	77%
1997	4,376	1,202	275	1,934	62%	1,750	69%
1998	4,538			2,019	0%	1,815	0%
1999	4,706	975	207	2,108	46%	1,882	52%
2000	4,880	1,201	246	2,201	55%	1,952	62%
2001	5,510			2,541		2,204	
2002	5,711			2,651		2,284	
2003	5,920			2,766		2,368	
2004	6,137			2,886		2,455	
2005	6,361			3,010		2,544	
2006	6,594			3,140		2,638	
2007	6,835			3,275		2,734	
2008	7,085			3,416		2,834	
2009	7,344			3,563		2,938	

Year	Pop.	Average Demand	Average Demand	Modified MACA Formula Ave. Day	Actual vs MACA Formula	Municipal 400 lpcd Ave. Day	Actual vs Municipal 400 lpcd
		m^3/day	lpcd	m^3/day	%	m^3/day	%
2010	7,612			3,716		3,045	
2011	7,891			3,876		3,156	
2012	8,179			4,042		3,272	
2013	8,478			4,215		3,391	
2014	8,788			4,396		3,515	
2015	9,110			4,584		3,644	
2016	9,443			4,780		3,777	
2017	9,788			4,984		3,915	
2018	10,146			5,251		4,058	
2019	10,517			5,443		4,207	
2020	10,901			5,641		4,360	
2021	11,300			5,848		4,520	

City Population		GNWT Bureau of Statistics - 1997	
Projections	8500	Projections Shown as Boxed ie:	5,203

Municipal Projections Based on Average Demand (Trucked & Piped) of 400 lpcd	**Demand**	=F*P*225 lpcd*{ -1.0+.323(ln P)} for 2,000 - 10,000 =F*P*225 lpcd*2.0 for > 10,000
	Where	**F =** Allowance for Leaks = 1.15 **P =** Population

NOTES: 1. MACA Formula assumes all piped users, actual is a mix if trucks & pipes

Table 1.2 summarizes the Water Treatment Plant outputs recorded monthly from 1993 to 2000. Further, it presents minimum, maximum and average daily outputs based on that monthly data.

The data does not suggest any clear trends as they vary significantly from year to year. In addition, during the construction of the water reservoir in 1996, the flow meter was replaced. The old one was suspected of being miscalibrated, but to what extent, was never satisfactorily resolved.

Table 1.2 – Iqaluit Water Plant Output Records

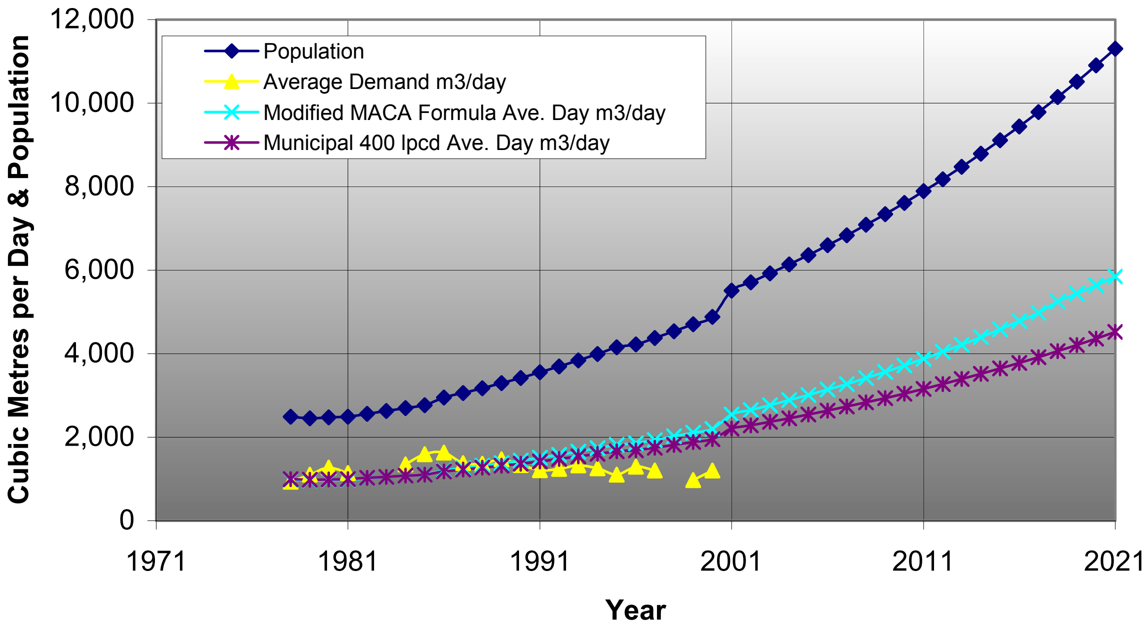
	1993 cu. m.	1994 cu. m.	1995 cu. m.	1996 cu. m.	1997 cu. m.	1999 cu. m.	2000 cu. m.
January	41,413	41,078	33,581	35,341	42,359	24,538	26,234
February	36,601	41,439	31,807	35,349	34,651	26,892	27,779
March	41,166	46,313	31,333	39,760	35,090	26,738	26,534
April	40,813	44,309	33,039	41,605	33,007	29,881	24,919
May	43,101	45,077	35,477	38,794	30,173	38,706	26,646
June	40,639	28,657	33,181	36,596	37,481	40,993	46,658
July	39,285	33,713	31,414	35,090	34,907	38,748	45,826
August	40,101	35,344	34,821	36,381	38,539	36,968	44,164
September	39,778	36,709	29,208	49,284	38,618	33,443	44,363
October	40,807	36,753	37,491	39,971	42,887	24,647	44,962
November	38,952	34,683	34,604	41,687		15,180	42,694
December	42,315	33,251	35,855	41,768	35,052	19,171	37,472
Total Yearly	484,971	457,326	401,811	471,626	402,764	355,905	438,251
Based on Month Data							
Min. Cu.m./Day	1,307	955	974	1,132	973	506	831
Max. Cu.m./Day	1,390	1,494	1,209	1,643	1,383	1,366	1,555
Based on Year Data							
Ave. Cu.m/Day	1,329	1,253	1,101	1,292	1,103	975	1,201
Low as % of Ave.	91%	75%	87%	89%	90%	51%	68%
High as % of Ave	107%	122%	112%	125%	128%	138%	128%

Legend for Table 1.2: Shaded indicates minimum value for the year.
Outline indicates maximum value for the year.

It should be noted that the maximum cu.m/day in the year 2000 of 1,555 m^3/day, equates to 319 lpcd, assuming a year 2000 population of 4,880 persons. Again, demonstrating 400 lpcd is a conservative design demand value.

Further, in 1997 during the construction of the new water storage reservoir, the plant output/system supply flow meter was changed out. At the time, the original flow meter output was suspect, and consequently long term trend analysis of plant output is difficult, and should be used with caution.

Figure 1.1 – Iqaluit Projected Water Consumption



Design Water Demands

Maximum Day and Peak Hour Factors of 2.0 and 3.0 respectively, are deemed appropriate and agree with current MACA standards. Consequently, the 2021 design flows are estimated to be:

Average Day	4,520 m^3/day
Maximum Day	9,040 m^3/day
Peak Hour	13,560 m^3/day

Table 1.3 outlines the previous and current design parameters.

Table 1.3 - Comparison of Design Parameters

	1994 Planning Brief	1998 Predesign Brief	2001 Predesign Brief	1998vs2001 %Chg
Design Year	2013	2017	2021	-
Design Population	5,626	8,500	11,300	33%
Design Peak Hour Demand	5,214 m3/day	8,456 m3/day	13,560 m3/day	60%
Design Average Day Demand	2,607 m3/day	3,400 m3/day	4,520 m3/day	33%

The marked percentage increases in design demands, arising from the new population projections has significant design and cost implications, ranging from insufficient storage volumes to replacement of the raw water intake.

Fire Flows

As advised by the City for the 1994 planning brief, a fire flow of 6,000 l/min for two (2) hours, or 720 cu.m., has been utilized for this report.

System Deficiencies

The 2000 average City demand for potable water, from plant records, is approximately 1,201m^3/day, which for the readers' information, is equal to the 1997 average demand of 1,202 m^3/day.

At present, the maximum rated plant output is 1,296 m^3. The treatment plant output will need to be increased to meet future demand, and in fact the plant is already being pushed over its design limits. The 2000 average demand of 1,201m^3 exceeds the recommended plant output of 1,050 m^3 by 15%.

The following will be addressed by the predesign report:

- the new chlorine room will be assumed to meet applicable code or functional requirements,
- additional pH correction is required to reduce corrosivity,
- removal of taste and odours during spring breakup have not been adequately documented, but if required, and will likely utilize powdered activated carbon, (PAC),
- alum clarification to enhance colour and turbidity removal, and ensure removal of parasites,
- many mechanical and electrical components have reached the end of their useful life and should be replaced,
- update chemical feed systems to take advantage of current technology, and
- improve plant safety.

2.0 WATER SUPPLY

2.1 General

This section describes the new water supply proposed for the plant.

In general, the proposed systems are in accordance with the conclusions and recommendations given in the 1998 Planning Brief. As a result of the startup meeting held in Iqaluit on August 23, 2001, changes to the population and demand projections have been made. These changes have implications.

2.2 Design Capacity Water Treatment Plant

It was agreed at the startup meeting that the design life of the upgraded plant should be 20 years, that is, until the year 2022.

Revised population and treated water demand estimates for the extended design period are presented in the Introduction. Based on these figures, the revised average day demand estimate at the end of the 20-year design period is 4,520 m^3/day. Assuming the same peak day factor of 2.0 recommended in the previous design brief, the net capacity of the water treatment plant would be 4,520 x 2 = 9,040 m^3/day. Allowing an additional 5% for in-plant uses such as filter backwashing, the gross plant capacity would be 9,040 x 1.05= 9,492 m^3/day. Rounding this up gives a value of 9,500 m^3/day, which is what we recommend as the nominal gross water treatment plant design capacity.

Water Supply System Capacity

At the time of this report, the City of Iqaluit had engaged OMM/Trow to review the recharge capabilities of the Lake Geraldine watershed.

OMM/Trow's initial findings revealed an annual recharge volume of 586,000 m^3 should be available. According to OMM/Trow this will result in a potential raw water shortfall situation around the year 2005, based on current demand and population projections. This is a major concern, and warrants careful consideration.

A review of the original drawings of the Lake Geraldine dam structure revealed the following:

- Inlet Elevation = 101.3 m or 332.3 feet (provided by OMM/Trow)
- Design Minimum Operating Level = 103.82m or 340.6 ft
- Current Maximum Operating Level = 109.33m or 358.7 ft (provided by OMM/Trow)

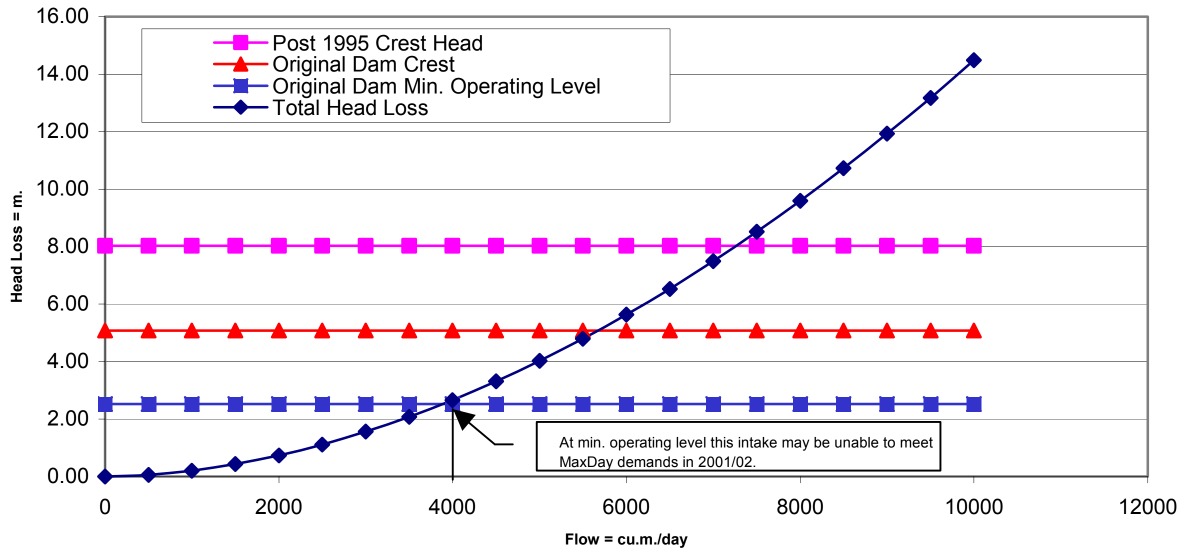
Using the above elevation data, the following heads were established.

- Minimum Operating Head = 103.82 – 101. 3 m = 2.52 m
- Current Maximum Operating Head = 109.33 – 101.3 m = 8.03 m
- Original Maximum Operating Head = 106.38 – 101.3 = 5.08 m

Figure 1.2 shows the flow of the existing 250 mm diameter intake related to the head available in Lake Geraldine.

Note that at a minimum head conditions, of approximately 2.52 m, the system curve intersects the available head at 4,000m^3/day. In maximum day conditions (equal to two times average day) this demand flow rate is predicted to occur in the year 2001/02, based on 400lpcd. This derived from Table 1.1 looking for an Average Day Demand of approximately 2,000 m^3/day, which corresponds to the years 2001 to 2002. Using the MACA derived formula, the intake should have been over capacity in the year 1998.

Figure 1.2 – Iqaluit Intake – Existing 250 mm ø



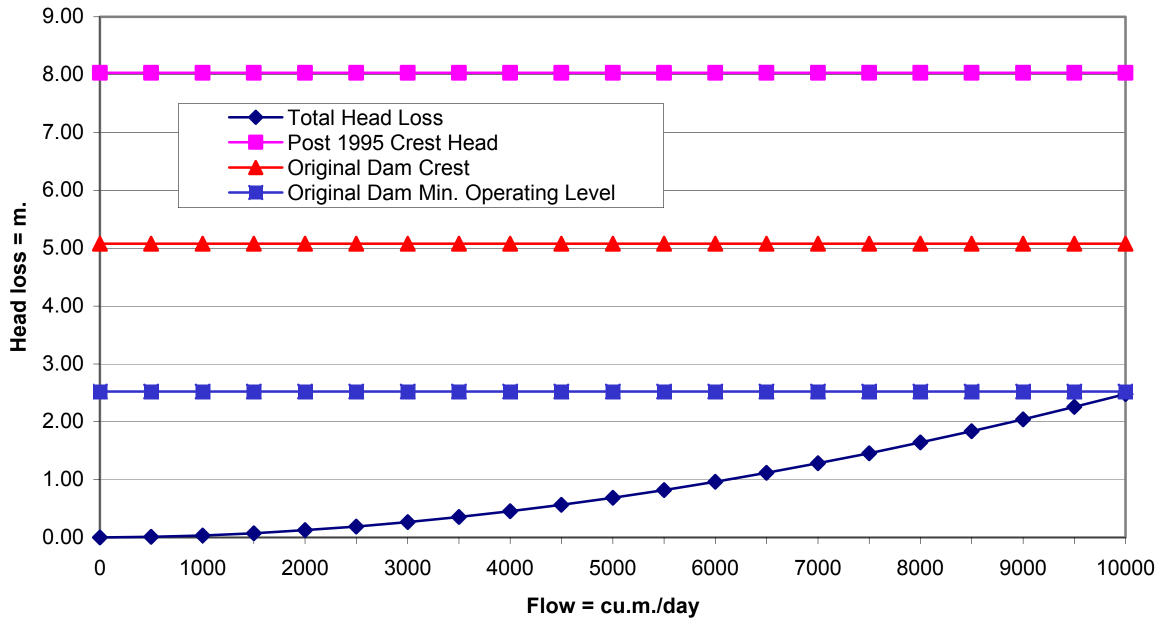
Observation of the level of Lake Geraldine in 1996/97 placed the elevation of the water surface at 109.3 m or a head of 8 m. At this elevation, the intake capacity is approximately 7,800 m^3/day, from the chart, based on assumptions made with respect to roughness coefficients due to the age of the intake.

However, based on the revised population provided to Earth Tech, by the City of Iqaluit, for this assignment, 7,800 m^3/day will no longer be sufficient in the design year. The design year raw water intake capacity is approaching 10,000 m^3/day, as developed in Section 2.2, Page 11.

Using the existing 250 mm intake, and assuming a C value of 80 to reflect the age of the steel pipe, Figure 1.2 shows the system curve of the existing intake and reveals the required head to deliver 9,500 m^3/day is approximately 14 m, or nearly twice the existing head. The topography of the Lake Geraldine reservoir will not support a dam height change of this magnitude. Other means to increase flow rates under existing available head conditions, needs to be investigated.

Figure 1.3 shows the system curve of a new 300 diameter HDPE intake operating under the existing Lake Geraldine dam head conditions, and clearly it is well matched for this application, as the required 9,500 m^3/day flow rate can be reached at 2.52 m of head; or at the minimum operating level of the reservoir.

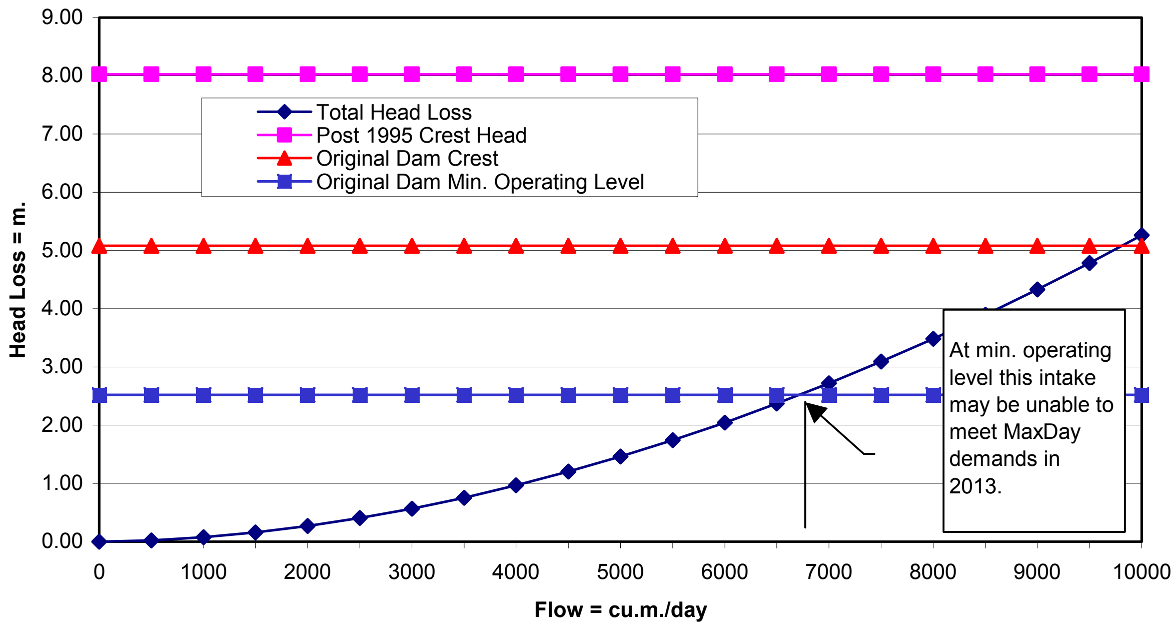
Figure 1.3 – Iqaluit Intake New 300 mm ø



One option is to upgrade the intake by twinning a new one, twinned with the existing. This would provide redundancy in the raw water intake components while increasing the capacity. This option adds complexity and risk due to the different heat loss characteristics of the two intake assemblies. They would need to be individually monitored and controlled for freeze protection. Further, the additional heat requirements for tempering two intakes may not be available from the existing boilers. Consequently, we do not recommend this option.

Figure 1.4 shows the system curve of a new 250 mm diameter HDPE intake under the existing Lake Geraldine dam head conditions, and it is slightly undersized, to deliver 10,000 m^3/day, at reduced water depths. This sized intake will be unable to meet max day demand flow rates, under minimum operating levels, in approximately the year 2013.

Figure 1.4 – Iqaluit Intake New 250 mm



Based on the above analysis and in conjunction with the City's population projection's and 400 lpcd demand rate, the existing intake could be replaced with a new 300 mm diameter HDPE intake, insulated with a minimum of 75 mm urethane insulation and encased in a UV stable sheathing.

However, we recommend a 355 mm over the 300 mm diameter option, because the intake should have a design life well in excess of the WTP's twenty (20) years, and the upset cost to provide the larger intake is minimal.

Water Tempering

In 1992/93 the treatment plant was retrofitted with a new boiler room addition as part of the City's high temperature system abandonment. In 1999 Reid Crowther was retained to modify the raw water intake tempering line and valve chamber in response to operational changes occurring with NTPC raw water cooling system.

Essentially, NTPC required continuous raw water for cooling their gensets and this doubled as a freeze protection mechanism for a large portion of the raw water intake. The decommissioning of this cooling system necessitated the extension of the 50 mm diameter copper tempering line approximately 260 m to the dam valve chamber.

This revised length posed significant head loss considerations in a 50 mm diameter line, and consequently it was replaced over the entire 360 m length with salvaged 150 mm diameter insulated HDPE from NTPC's recirculation line.

This work was completed during the fall of 1998, including replacement and addition of new valves in the system, and abandonment of old components.

Lake Geraldine Dam Structure

The City of Iqaluit should, in the not to distant future, undertake an investigation of the portion of the intake through the dam structure. Based on the condition of the valves and fittings removed during the 1999 valve chamber work, it reasonable to expect the embedded portion of the intake to be badly corroded. To the best of our knowledge, this has not been investigated to date. This portion may well become a limiting factor in intake throughput. The investigation, let alone the remedial work, will be difficult and require divers. A visual assessment from the intake side of the dam, will provide a reasonably accurate assessment of the degree of iron nodule formation and other flow restrictions such as debris, siltation, etc.

3.0 WATER TREATMENT PROCESS

3.1 General

This section describes the basic water treatment processes and equipment proposed for the plant.

In general, the proposed process treatment is in accordance with the conclusions and recommendations given in the 1998 planning brief. As a result of the predesign meeting held in Iqaluit on August 23, 2001, various changes from the planning brief recommendations have been made. Significant changes to the 1998 report arise predominantly from the marked increase in design year population projections, as highlighted in Section 2.

3.2 Design Capacity of the Water Treatment Plant

It was agreed at the start-up meeting that the design life of the upgraded plant should be 20 years, that is, until the year 2021.

Revised population and treated water demand estimates for the extended design period along with the water treatment plant design capacity are presented in Section 2. Based on these figures, the revised demand estimate at the end of the 20-year design period is 4,520 m^3/day; the net capacity of the water treatment plant would be 9,040 m^3/day and, allowing an additional 5% for in-plant uses such as filter backwashing, the gross plant capacity would be 9,492 m^3/day. Rounding this up gives a value of 9,500 m^3/day, which is what we recommend as the nominal gross water treatment plant design capacity.

3.3 Process Description

The 1994 design brief discusses the various water treatment process options in some detail. The main conclusions were that the quality of the raw water drawn from Lake Geraldine was generally very good, but some treatment was required to ensure the treated water meets Canadian Drinking Water Quality Guidelines. The areas that needed to be addressed were:

Turbidity:	Goal of 1 NTU to ensure removal of parasites
Colour:	Generally not a concern, but may be an occasional problem. Goal of 5 TCU.
Taste and odour:	May be a problem during spring breakup.
Corrosivity:	Raw water is aggressive, and pH correction will be required.

The main recommended process changes from the 1998 design brief which are still applicable were as follows:

1. Increase the capacity of the plant by constructing four new filters, extending the existing building structure to house them, and installing new backwash pumps.

2. Convert the existing sedimentation tanks into grit removal and flocculation units.

3. Provide additional backwash waste storage.

4. Provide facilities for dosing alum (or other coagulant) upstream of the flocculation chamber. Install coagulant mixing facilities.

5. Provide powdered activated carbon dosing facilities, if required. Design to allow for future inclusion into process train.

6. Replace the existing lime handling system with a caustic soda system.

7. Provide a PLC-based control system and desktop computer, to automate certain plant functions and provide data logging capability.

In general (with the exception of the design plant capacity) little has changed since the 1994 and 1998 planning briefs were issued. The following process description is, therefore, based on the planning brief recommendations, with some changes where appropriate to accommodate the increased plant capacity, recent discussions, and the various other considerations discussed below. Process and Instrumentation Diagrams (P&IDs A and B) of the revised process flow are presented at the end of Section 3, Page 41.

The recommended process upgrades are subject to change, following the laboratory water treatability evaluations planned as part of the final design process.

During our site inspection in February 1998, it was determined that although most of the concrete structure was in good condition, essentially none of the existing process equipment predating recent upgrades to the chlorine room and pumps, (such as piping, pumps, and valves) were worth salvaging and reusing. We assume this still to be a valid statement and the only existing structures and recently introduced equipment will be reused, to the extent possible.

Raw Water Supply

Raw water enters the plant through an existing 250 mm main and a flow control valve. The existing flow control valve will be replaced with a new unit, capable of accurately controlling flows from present minimum demands of about 1,000 m^3/day up to about 10,000 m^3/day, slightly above the plant design raw water flow.

pH Adjustment

The Iqaluit raw water is aggressive (corrosive) and will be made more so by the use of coagulants such as alum. In addition, for most common coagulants to even react with the water to form a floc, some increase in its alkalinity is often required - in other words, some alkaline chemical, such as lime or caustic soda, might have to be added. Whichever coagulant is selected, some pH adjustment will still likely be required.

The existing plant includes a limestone contactor, in which the raw water flows through a series of baffled chambers. This system could possibly be retained, but it has the serious disadvantage that there is no control over the amount of limestone dissolved and the resulting pH, which could vary considerably depending on factors such as raw water flow and temperature. Of the various chemicals can be used for pH adjustment, lime is one of the most common, because it is inexpensive and relatively safe to handle. Unfortunately, the equipment needed for handling powdered lime is expensive to buy and maintain, and its operation is messy and labour-intensive.

A more conventional pH adjustment system, with the capability of accurately controlling dosages, would be required whether the limestone contactor is retained or not. Accordingly, we recommend that the existing limestone contactor be abandoned.

An alternative to lime, and one that is becoming increasingly common, is caustic soda. This chemical is supplied as a solution in plastic drums, and is simply pumped from the drum to the process by a small chemical metering pump. The equipment is therefore much simpler and cheaper than would be required for lime, and operation of the system is much easier. Caustic soda is more expensive than lime, but less is needed so operating costs usually turn out to be similar.

We recommend the use of caustic soda at Iqaluit. We also recommend that two dosing points be provided: one at the front end of the plant in the event that pH adjustment is required before coagulation and flocculation (which may not always be required according to the treatability study undertaken by Earth Tech in 1998) and the second dosing point downstream of the coagulation, if the pH needs to be adjusted or re-adjusted to reduce corrosiveness.

Coagulation

The purpose of coagulation is to cause the joining together of particles in the raw water that are so small they would otherwise pass through the filters. This is done by adding a chemical (such as alum) that reacts with the water to neutralize electrical charges on the particles and also to form precipitates called *floc*. The floc entraps smaller particles, and both are subsequently separated from the water in the filters. In most plants, there are two intermediate processes between coagulation and filtration: *flocculation*, in which the coagulated water is gently mixed to allow the floc to grow in size; and *sedimentation*, in which the floc is separated from the water by gravity settling in a large tank. In the case of Iqaluit, the raw water is of excellent quality and very little coagulant is needed. The resulting small quantities of floc can be removed in the filters, without the need for a sedimentation stage. This process, which is commonly used with high-quality raw water supplies, is known as *direct filtration*.

Various coagulants are available. The most commonly-used one, aluminum sulfate or alum, is available in dry (powdered) form in bags, or as a solution in drums (commonly called “liquid alum”). Similar considerations to those discussed in Section 3.2 pH Adjustment, apply. Tentatively, we recommend the use of liquid alum at Iqaluit.

Other coagulants, such as PASS (polyaluminum silicate sulfate) or PACl (polyaluminum chloride) may well prove to be better than alum for use at Iqaluit. These coagulants are more expensive than alum, but lower dosages are needed and they do not depress the pH as much as alum. These and other coagulants will be evaluated during the treatability tests.

Flocculation

As already mentioned, flocculation facilities are usually provided to allow the floc particles to grow to a size that can easily be removed in the filters. This is done by gently stirring the flocculated water for periods of (typically) about 15 to 45 minutes. Usually, small amounts of *polymer* (flocculant aid) are added to increase the floc size and strength, and the speed of floc growth.

We also recommend the installation of polymer dosing facilities. An appropriate polymer can greatly increase the effectiveness and flexibility of the pretreatment processes, and improve the quality of the treated water, particularly with low-turbidity, cold raw water supplies. The most effective polymers for this type of application are usually available only in powdered form. However, the quantities involved are very small (dosages of less than 1 mg/L) so the handling and feeding facilities are not usually a problem. Generally, these consist essentially of a plastic mixing tank, a small motor-driven mixer, and a chemical dosing pump. Polymers are usually added to the second flocculator stage in this type of application.

Filtration

Additional filter capacity will be required for the upgraded plant flows. In the 1998 planning brief, the recommended course of action was to abandon the existing filters and construct four new units in an extension to the existing building. This recommendation is still basically valid with some exceptions due to increases in design flow rates and subsequent equipment sizing as discussed in Section 3.4.

Taste and Odour Control – if required/added in the future

Occasional taste and odour problems at water treatment plants can usually be best controlled by the use of powdered activated carbon (PAC). This is a finely-divided form of carbon which has been specially treated to adsorb taste- and odour-producing compounds from the water. It is simply added to the water at some stage in the process sequence, and removed in a later stage. At Iqaluit, the most appropriate location for the addition of PAC would be the inlet to the flocculators. It would then be removed in the filters. PAC is supplied in bags, and is mixed with water before use to form a slurry which is then pumped to the process. PAC is somewhat messy to handle, but since its use at Iqaluit will be relatively infrequent, this is not considered to be a major problem. The PAC feeding system would consist basically of a plastic drum, a motor-driven mixer, and a chemical dosing pump.

Disinfection

As discussed in the 1998 planning brief, we recommend installation of a chlorine residual analyzer and control system to automatically monitor the chlorine residual in the treated water. This type of system ensures that the desired residual is maintained at all times, thus ensuring proper disinfection. It also reduces the operator's workload to some degree.

3.4 Process Equipment Selection And Sizing

This section describes the process equipment and sizing of the major tanks. As much of the existing facility as possible has been reused to minimize capital cost investments. In sizing the equipment and tanks, two scenarios were investigated to determine the impact on tank sizing and upgrade timing. Plant capacities, calculated as maximum day demand plus in-plant water needs for a consumption rate of 400 lpcd, was considered, as follows:

- A consumption rate of **400** lpcd and **5**% in-plant water needs: 9,500 m3/day

Basically, at the lower flow rate, the time frame of upgrades could be delayed by around five years. However, upgrades to the entire system were required under either scenario with only slightly different tank sizing requirements. Therefore, as stated in Section 2, this discussion will be based on the consumption rate of 400 lpcd.

Flash Mixer

Raw water presently enters the water treatment plant and flows through a baffled chamber, where lime is added for pH adjustment. As discussed in the previous section, the use of the lime contactor for pH adjustment will be abandoned in favour of caustic soda addition.

However, the new water treatment process requires a first tank in which chemicals such as caustic soda, alum and polymers are mixed vigorously with the incoming raw water to ensure a homogeneous solution. The detention time required in the flash mixer is usually under one minute.

The lime contactor will provide ample detention time for use as a flash mixer. The available volume in the contact tank is approximately 13.8 m^3. The retention time for the plant capacity design flow is presented in Table 3.1.

Table 3.1. Retention Time in the Flash Mixer

Plant Capacity (m^3/day)	Retention Time (min)
9500	2.1

Flocculation Tanks

As already mentioned, flocculation facilities are usually provided to allow the floc particles to grow to a size that can easily be removed in the filters. This is done by gently stirring the flocculated water for periods of (typically) about 15 to 45 minutes. Based on treatability tests, it is recommended that 40 minutes be provided for this system, unless treatability analysis shows otherwise.

A flocculation chamber is provided in the existing plant, but its detention time of less than 5 minutes at the new plant design flow, is much too low for effective flocculation. Three options to increase retention times are discussed below.

Option 1 – Flocculation Chamber and Settling Tanks

The 1998 planning brief suggested using the existing flocculation chamber and converting both existing settling tanks for use as flocculation tanks since the settling tanks will no longer be required. The total volume of these three tanks is 185 m^3 and could provide a detention time of approximately 40 minutes based on 1998 design flows. However, with the further increase in design flows, 185 m^3 will not provide sufficient detention time at the design flows. Refer to Table 3.2. Based on the volume provided by these three tanks, a maximum flow of 6,652 m^3/day could be accommodated to achieve a retention time of 40 minutes. A plant flow of 6,652 m^3/day would correspond to an average day demand of 3,168 m^3/day, would be reached in 2011 at a design demand of 400 lpcd, based on current population projections for the design year.

In summary, the existing flocculation chamber and settling tanks could be used to provide a flocculation system for the new water treatment plant, but they will be insufficient in size, around the year 2011. Additional flocculation tank volume would then be required. There are advantages to providing the total design flocculation tank volume during the initial construction, while the temporary water treatment plant is on site, rather than going through two (2) plant shutdowns and construction programs.

Option 2 – Flocculation Chamber, Settling Tanks and Surge Tanks

In order to make use of the existing tankage, the existing settling tank floors could be removed and the volume of the existing surge tank added to the settlings tanks. The total tank volume provided by the flocculation chamber (25 m^3), the two settling tanks (80 m^3 each) and use of the surge tank (33 m^3 x two sections), i.e. 250 m^3, will provide retention times as shown in Table 3.2 below.

However, there are several major disadvantages to this option:

- The floor of the existing surge tank might only be designed to carry a load of 1.6 m of water. Although it would be feasible to reinforce this floor, this would be a difficult and onerous task.

- The height of the flocculating tank column would be approximately 7.3 m. Typical flocculators are designed at a height of 3.0 m to 3.7 m, half the height. This height might be problematic for the design of flocculator equipment.

- The floor of the existing surge tank is sloped which would also require some upgrades.

In summary, although the retention time provided by this option would be adequate, there are several potential process and structural disadvantages to this option.

Option 3 – Flocculation Chamber, Settling Tanks and Filter Tanks

A third alternative would be to reuse the existing flocculation tanks, settling tanks and filters. The total tank volume provided by the flocculation chamber (25 m^3), the two settling tanks (80 m^3 each) and the two filters (60 m^3), i.e. 305 m^3, will provide retention times as shown in Table 3.2 below.

The major advantage of Option 3 is that by converting the filters for reuse as part of the flocculating tanks, the capacity of one flocculating train, i.e. (25 m^3+160 m^3+120 $m^{3)}$/2= 152.5 m^3, would provide sufficient retention time to meet a plant capacity of 5,940 m^3/day or an average day demand of approximately 2,828 m^3/day. It is projected that this demand will be reached in 2008. Thus, the plant could be run with only one train in operation, if needed, for several years. Under option 2, the volume of one train, 138 m^3, would provide sufficient retention time to meet a plant capacity of 4,968 m^3/day, which is equivalent to an average day demand of approximately 2,366 m^3/day projected to be reached in 2003.

Table 3.2. Retention Time in the Flash Mixers

	Option 1: Floc Chamber +Settling Tanks		**Option 2: Floc Chamber, Settling+ Surge Tanks**		**Option 3: Floc Chamber, Settling Tanks + Filters**	
Plant Capacity (m^3/day)	Available Volume m^3	Retention Time (min)	Available Volume m^3	Retention Time (min)	Available Volume m^3	Retention Time (min)
9,500	185	28.0	250	37.9	305	46.2

Summary

For all of the options described above, the conversion of the existing flocculator and settling tanks would be relatively simple. The existing flocculator chamber would need some modifications to the baffles. The existing settling tanks would require the addition of motor-driven mixers and a baffle wall spanning the middle section of the tank to minimize short circuiting.

If Option 2 is used, the settling tank floor would be removed; a partition would be required through the middle of the surge tank and the surge tank floor would be leveled and strengthened.

If Option 3 is considered, in addition to providing a baffle wall and mixers in the settling tanks, the existing outlet channel would be removed and a direct overflow provided between the existing settling tanks and filters. Modifications to the filter area would be made to provide a symmetrical extension of the existing settling tanks. The existing filters and filtered water storage area directly below the filters would be used as part of the flocculation train thus the tank level in the filters would be slightly lower than the settling tank floor. Flow to the existing filtered water storage area not directly below the existing filters would be cut off. Modifications to the filter area would include: removal of the inlet channel; removal of the filter structure including tanks and underdrain support system; removal of the contact column inner walls; extension of the south filter wall; addition of a dividing wall between the filter areas. At lower flows, both trains can be operated in parallel and initially, only one train is required.

Recommendation

Earth Tech recommends Option 3, as it provides the most operating flexibility and ensures that retention times for future design flows can be met. We also recommend making all the necessary structural modifications at the times of the existing upgrade, while the plant is off-line.

Filtration Tanks

Design Considerations

Several design criteria used in the sizing of the filtration system will impact not only the filter area required to achieve proper filtration but also the number of filters, the size of the backwash pumps, and the storage volume required for backwash and filter to waste waters. These key design criteria include the flow rate, filter loading rate, the backwash loading rate, backwash duration, air scour rate. The impact of these design values on the equipment sizing was reviewed.

A summary of the typical and proposed design criteria is presented in Table 3.3.

Table 3.3. Filter Loading Rates

Design Parameter	Unit	Typical Range	Design Value
Filter Loading Rate	m/hr	8-12	8.5
Backwash Loading Rate	m/hr	35-50	50
Backwash Duration	min	5-20	12
Air Scour Rate	$m^3/m^2/hr$	18-27	25

Some of the conventions used when sizing the equipment are as follows:

- Filters should be designed with one filter out of service as this situation occurs regularly, i.e. when a filter is being backwashed.

- Backwash storage systems should be designed to provide sufficient storage for two filter backwashes.

In addition, these filters will be designed with extra freeboard to allow a future change from conventional filters to deep bed filters in the event that regulations become more strict.

Filter Sizing

It is proposed to house the new filters in the area above Clearwell No.1 by extending the existing north and west wall of the building. This area is slightly greater than the building extension area originally envisaged. The additional area would be approximately 6.8 m x 13.4 m for a total of 91.2 m^2. Allowing for approximately 3 m in front of the filter for the pipe gallery, the available area for the filters, including tank walls, would be approximately 51 m^2.

The increased plant design capacity now requires a significantly larger total filter area than discussed in either the 1994 or 1998 planning briefs. The 1998 planning brief proposed a filter loading rate of 7.5 m/h (a rate considered slightly conservative). The advantage of using a more conservative filter loading rate is that the filter run times are often longer, thereby increasing the time between backwashes. However, larger filter areas require greater backwash supply capability and backwash waste storage facilities. At a design flow of 9,500 m^3/day, a loading rate of 7.5 m/h would require a total filter area of 53 m^2, more than the available area, and a backwash volume for two filters of 264 m^3.

Therefore, we recommend increasing the filtration rate to 8.5 m/h thereby reducing the total required filter area to 47 m^2. Assuming four filters are used, the backwash volume for two filters would be 232 m^3. The backwash storage volume could be further reduced by increasing the number of filters from four to five. As the total filter area required remains constant at a given filtration rate assuming five filters are used, the backwash volume for two filters would be 186 m^3 at a plant flow rate of 9,500 m^3/day and filtration rate of 8.5 m/h. However, smaller filters require a greater number of valves and controls. For a system the size of Iqaluit, it is more practical to allow for greater storage than increasing the number of filters.

In summary, we propose the following:

1. We recommend the installation of four filters to be operated at a loading rate of 8.5 m/h. Therefore, when one filter is out of operation, the loading rate will be 11 m/h which is still in the typical range of filter loading rates.

2. Filters dimension will be approximately 4 m long x 2.5 m wide x 4 m high each. We recommend the prefabricated aluminum type of filter construction, available from several suppliers.

3. We recommend the installation of two new vertical turbine backwash supply pumps to meet the new backwash rates. Vertical turbine pumps have many advantages, including self-priming capability, minimal floor space requirements and ease of maintenance.

Backwash and Filter to Waste Storage

Backwashing produces large amounts of waste water for a relatively short time, and because the capacity of the sanitary sewage system is limited, some form of surge or holding tank must be provided. Usually, provisions are made to store the wastes from at least two filter backwashes. Two backwashes of the proposed filters, at a rate of 580 m^3/h for 12 minutes, would result in a backwash flow rate of 9,700 l/min and waste volume of 116 m^3 x 2 filters = 232 m^3.

The 1998 planning brief proposed to reuse the existing surge tank (55 m^3), and convert the existing filtered water tank to function as a supplementary surge tank. Based on the current design flows, it is now necessary to incorporate part of the filters and filter storage area into the flocculation system. Thus, the available remaining filter storage volume is approximately 78 m^3. The total available backwash waste storage capacity would then be 133 m^3, which is inadequate for two filter backwashes.

Consideration must be given to providing additional storage. A list of tanks potentially available for use as backwash water storage and their respective volumes is presented in Table 3.4. In addition, a new storage tank could be built adjacent to the new filter area, along either the north or west building walls. Consideration was given to locating the tanks either along the east or south wall but the existing access to the chlorine room, fuel tank and utilidors prevented these options from being given further consideration.

Table 3.4 Backwash Water Storage Options

Description	Tank Volumes (m^3)
Surge Tank – existing	55
Filter Storage Tank – existing	78
Clearwell No.1 – existing	247
New Storage Tank	As Required
Total Volume Required	*232*

The most feasible options are discussed in the following paragraph along with advantages and disadvantages for each option.

Option 1 – Existing Surge and Filter Storage Tanks and New Tankage

Filter backwash storage could be provided by reusing the existing surge and filter storage tanks. In addition, a new tank would be required to compensate for deficient storage volumes. The additional storage would be located either along the north or west wall of the proposed building extension for the filters.

The advantage of this option is that the existing treated water storage capacity remains intact while minimizing capital cost investment. The disadvantage to this option is that filter backwash storage would be divided between three tanks, thereby requiring greater control when collecting backwash water in the tanks or discharging the water into the sewage collection system.

Collection of the backwash water in the three tanks would require either manual or automatic control to ensure that the tanks do not overflow or are not surcharged as the tank volumes and maximum fill elevations would vary greatly. In addition, the discharge of the backwash water into the sewage collection system would also require either manual or automatic control to ensure timely emptying of the tanks and to ensure that sufficient backwash storage space is available if another backwash is initiated.

Manual control would require operator time and attention to monitor tank levels and open and close valves as required. This time could be better served doing other duties. Automatic control would require the addition of addition tank level sensors, automatic valves and programming to coordinate tank filling and emptying.

Option 2 – Existing Clearwell No. 1

The existing Clearwell No. 1 would provide ample storage capacity for two successive backwashes.

The main advantage of using the existing clearwell is capital costs are minimal. Only some piping and concrete modifications would be required. However, the major disadvantages would be that on-site filtered water storage would be cut in half and the total treated water storage, which is already marginally deficient, as discussed in the next section would be reduced even further. Decreasing on-site storage would mean that more control over the backwashing would be required. To ensure that backwash pumps do not run dry, a filter backwash lockout would have to be provided whereby a minimum delay time between two consecutive backwashes would be required.

Option 3 – New Storage Tank

A third option would be to build a new storage tank sized to accommodate the flow from two consecutive backwashes. The advantage of this option is that the existing treated water storage can remain intact and control of backwash water collection in the tank and discharge into the sewage collection system is greatly simplified. However, this is the most costly of the four options considered.

Option 4 – Limit Backwashing

The fourth option would be to only provide sufficient storage for one filter backwash. The volume of wastewater produced during one filter backwash is approximately 116 m^3. This volume could be accommodated by using the existing surge tank and the filter storage tank. To ensure that a second backwash is not initiated until the storage tanks are empty, a filter backwash lockout would have to be provided whereby a minimum delay time between two consecutive backwashes would be required. This option would require more control than providing a completely new tank however capital costs would be much lower. Piping modifications and additional level sensors and controls would be required.

Recommendation

Considering the quality of the raw water, which is considered very good, and the present backwashing frequency of one filter per day, it is recommended to pursue Option 4: limit the filter backwashing and allow initiation of a filter backwash only when the storage tanks are empty. It is not expected that the filters would require more than one backwash per day and it is possible that this frequency might even be reduced. Although this option does not provide sufficient storage for two backwashers, additional storage could be provided at a later date if this configuration becomes inadequate.

3.5 Chemical Dosing

Upgrades have been made to the chlorination room, which is located in its own room. It is assumed that no code upgrades are required to this system, but some changes may be required to accommodate the proposed process changes.

It is proposed to located the other chemicals on the reservoir floor in the area along the west wall. New door would be provided along this wall to facilitate delivery of chemicals.

3.6 Treated Water Storage

Treated water storage is important for providing sufficient disinfection and to balance the needs of the community with respect to fire storage, equalization storage and emergency storage. Treated water storage requirements were calculated in the 1995 Design Brief according to the empirical formula presented below. The disinfection criterium has been added in this report. Therefore, the required treated water storage, S, can be calculated as follows:

S = (A + B + C +E) or D (whichever is greater)

Where:

S = Total storage requirement m^3

A= Fire Storage m^3

B = Equalization storage (25% of projected maximum day demand) m^3

C = Emergency storage (30% of projected average day demand) m^3

E = In-plant needs m^3

D = Disinfection contact time (T_{10}) storage to meet the CT requirements m^3

Disinfection Storage

Ensuring proper disinfection is achieved via sufficient contact time as well as ensuring that the disinfectant residual will last until the water has reached the consumer. Disinfection calculations have been based on achieving1.0-log *Giardia* and 3.0-log virus removal through disinfection contact time in the treated storage reservoirs. A ratio of T_{10}/T of 0.3 was attributed to the retention in the clearwell and main storage tank since there is no baffling within the tanks but there is interconnecting piping between the clearwells and the main reservoir. It was assumed that the pH of the water was 7.5 and as a conservative estimate, that the water temperature was 0.5°C. Based on these assumptions, the disinfection storage requirements for the system were calculated for existing and design flows.

The required CT values for viruses is 9. The required CT value for Giardia and a chlorine residual of 0.5 mg/L and 0.8 mg/L is 79 and 84 respectively. A summary of the required and available storage at existing plant capacity and future design flows is presented in Table 3.5.

Table 3.5 Required Disinfection Storage

	Flow (m^3/day)	Residual Chlorine (mg/L)	Required Storage (m^3)	Available Storage (m^3)
Existing Plant Capacity	1,296	0.5	504	2,285
Existing Plant Capacity	1,296	1.0	252	2,285
Design Flow	9,500	0.5	3694	2,285
Design Flow	9,500	1.0	1847	2,285

From Table 3.5, it can be seen that at the existing plant capacity, there is sufficient storage in the system for disinfection. However, at the future design capacity, there is insufficient storage if the chlorine residual is 0.5 mg/l. However, increasing the chlorine residual will decrease the required disinfection storage.

Acceptable CT values can be achieved through short contact times at higher concentrations. The existing storage reservoir's useful disinfection storage life can be extended beyond the year 2022, by increasing the residual chlorine from 0.5 to 1.0 mg/l.

Fire Storage

Generally, fire flow requirements have four characteristics: flow, duration, residual pressure, and looping. Recommended fire flows are related to the type and density of development within a specific area.

The required duration of fire flow is dependent upon the required fire flow. The required duration can be from as little as 1.0 hour for fire flows of 2,000 l/min or less to as much as 9.5 hours for fire flows of 40,000 l/min and over. In smaller systems or in the design of new reservoirs, the duration is an important factor because it will determine the amount of reservoir storage dedicated for fire flow.

As advised by the City for the 1994 planning brief, a fire flow of 6,000 l/min for 2 hours, or 720 m^3, has been utilized for this report. The plant will be capable of producing nearly 6,300 l/min, but that does not remove the need for fire storage, as the maxday demand still needs to be met during a fire event.

Equalization Storage

Equalization storage is generally defined as 25 percent of the maximum day demand. The calculation of this storage requirement is for a system where the water treatment plant is capable of satisfying only the maximum day demand, as is the case in Iqaluit. The equalization storage volumes that are required based on the existing and future maximum day demands, are: 2,402 m^3/day x 0.25 = 600 m^3 and 9,040 m^3/day x 0.25 = 2,260 m^3, respectfully.

The equalization storage component will allow the water treatment plant to operate at a steady rate over a given period of time and compensate for fluctuating demands throughout this period.

Emergency Storage

Emergency storage is generally defined as 30 percent of the average daily demands. The emergency storage volumes that are required based on the existing and future average day demands of 1,201 m^3/d x 0.30 = 360m^3 and 4,520 m^3/day x 0.30 = 1,356 m^3, respectfully.

In-Plant Water Needs

In addition to meeting system demands, the WTP must also provide sufficient water for in-plant water needs. This includes water for backwashing and plant service water including chemical requirements. Sufficient water storage must be available at the WTP to meet in-plant needs. In-plant water needs typically range between five and ten percent of the plant capacity, depending on the age and efficiency of the system, the type of backwash system and the number of filter backwashes required per day.

Presently, the filters are backwashed daily. As air scouring is not part of the existing filter operation, which usually results in higher backwash volumes, it is assumed that in-plant water needs correspond to 10 % of the plant capacity. This is approximately 105 m^3/day.

In the future, since the raw water source is very good, it is not expected that the backwashing frequency would increase beyond more than once per day. Therefore, for simplicity, it was assumed that in-plant water needs would be 5% of the plant capacity. Storage for in-plant needs would be approximately 475 m^3 at a plant capacity of 9,500 m^3/day. The in-plant storage volume is greater than the volume required for 2 backwashes, i.e. 233 m^3. The existing clearwell capacity of 575 m^3 exceeds the in-plant needs and therefore additional storage at the plant would not be required.

Summary

The recommended minimum treated water storage for the existing and future system demand are 2,385 m^3 and 4,811 m^3, respectively. The recommended treated water storage amounts above assume no increase in the recommended fire storage component of the storage equation. The storage available at the plant is 575m^3 which exceeds the present and future recommended storage requirements for in-plant needs. The total system storage, 2,875 m^3, just meets present requirements. However, the existing storage will likely no longer meet the storage requirements within a couple of years if the treated water demand increases as projected. The total storage volume should be increased within a couple of years. Table 5.9 summarizes the system storage requirements.

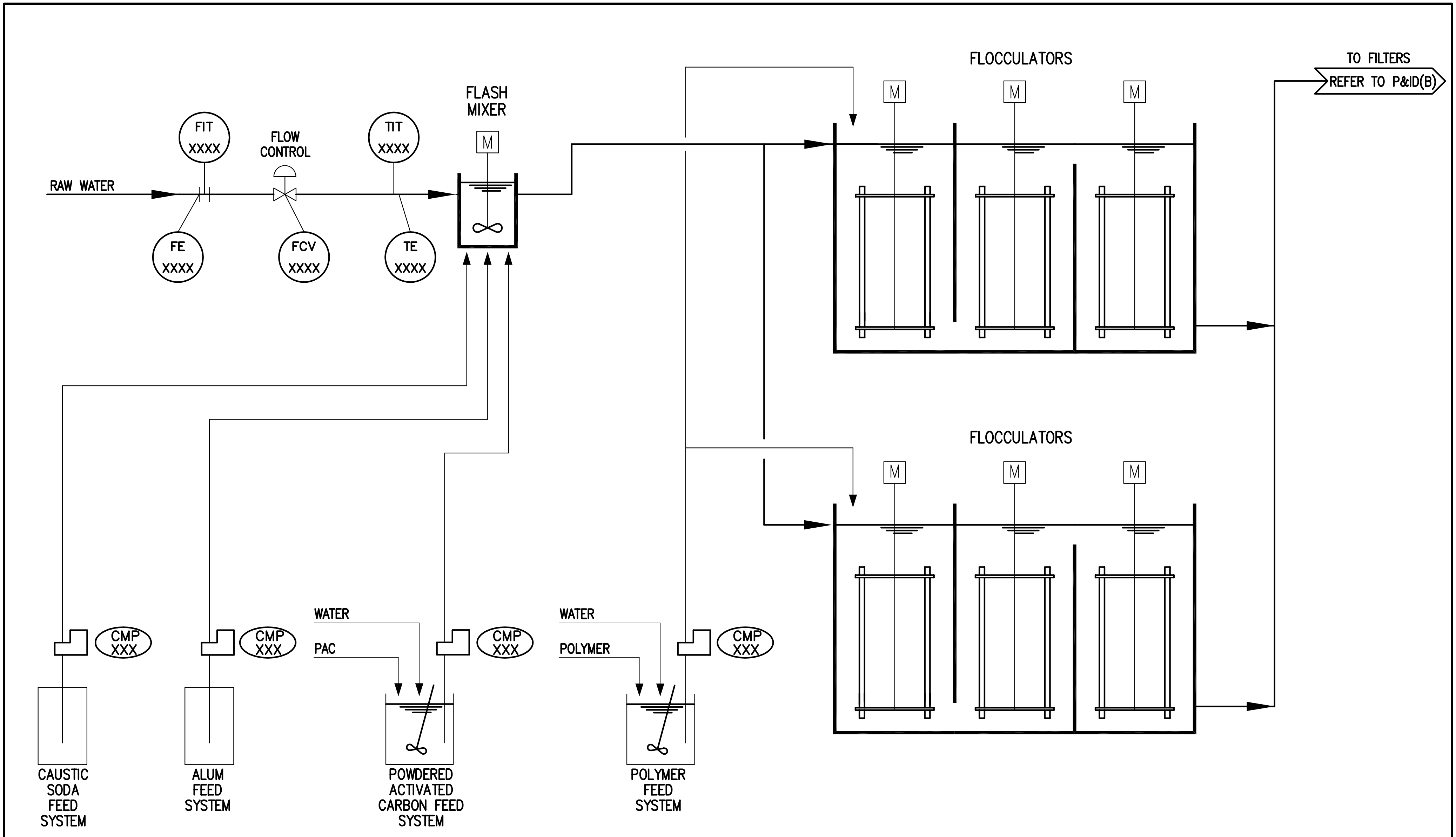
Table 3.6 – Reservoir Storage Requirements

Storage Requirements	Existing (m^3)	Future (m^3)
A – Fire	720	720
B – Equalization	1,200	2,260
C – Emergency	360	1,356
In-Plant Needs	105	475
Total Storage Required	2,385	4,811
Total Storage Available (Clearwell and Reservoir)	*2,875*	*2,875*

The existing storage meets disinfection storage requirements for both existing plant capacity. The existing storage can also meet disinfection storage requirements at plant design capacity of 9,500 m^3/day if the chlorine residual is increased to approximately 1.0 mg/L.

3.6 Process Sketches

- Piping and Instrument Diagrams (P and ID) - A
- Piping and Instrument Diagrams (P and ID) - B
- Programmable Logic Controller (PLC) Schematic



NO.	REVISION	DATE	INITIAL

IQALUIT WATER TREATMENT PLANT

MUNICIPALITY OF IQALUIT
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EARTH TECH

Earth Tech (Canada) Inc. Edmonton, Alberta 780 488 6800

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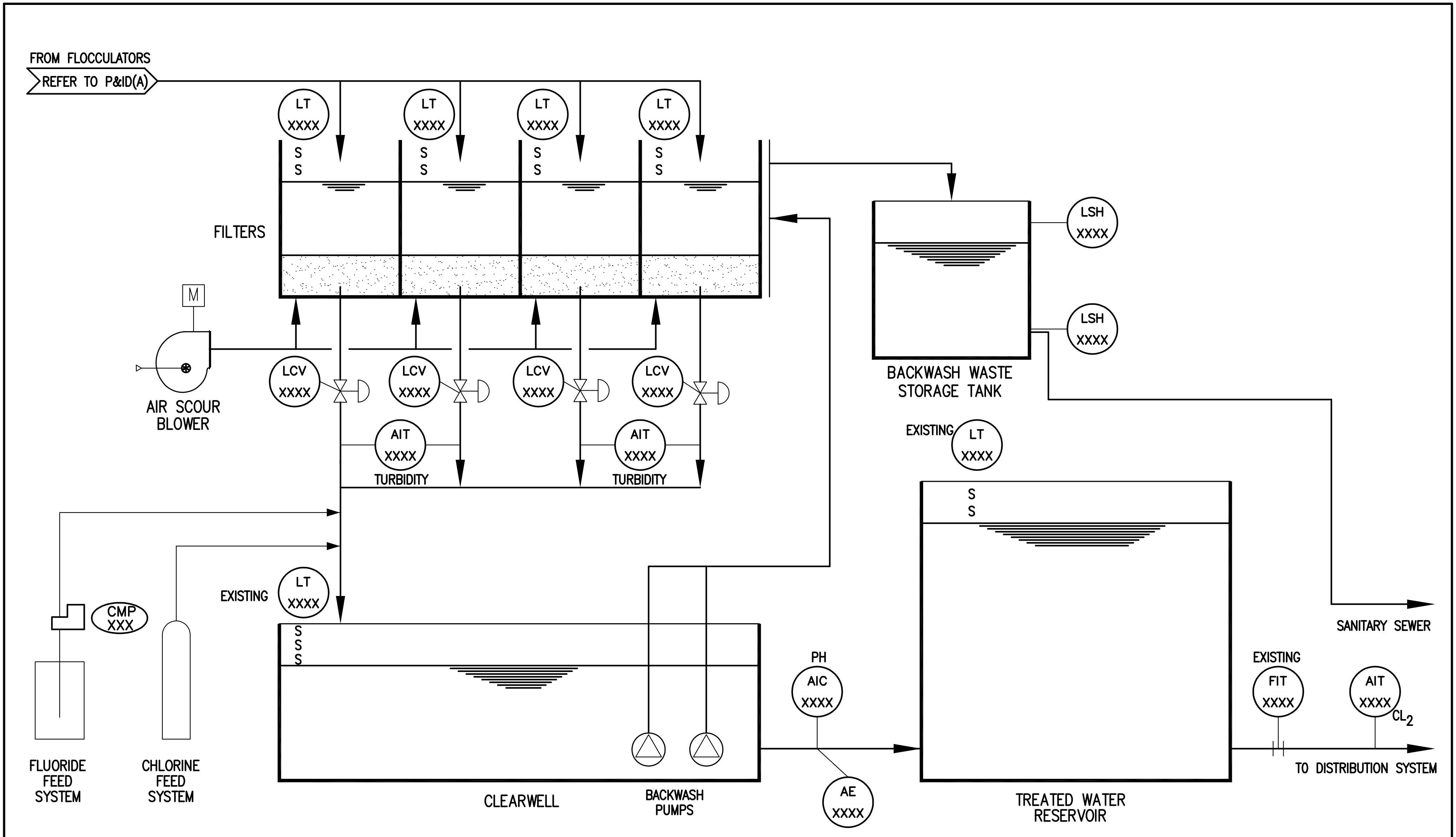

NO.	REVISION	DATE	INITIAL

IQALUIT WATER TREATMENT PLANT

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EARTH TECH

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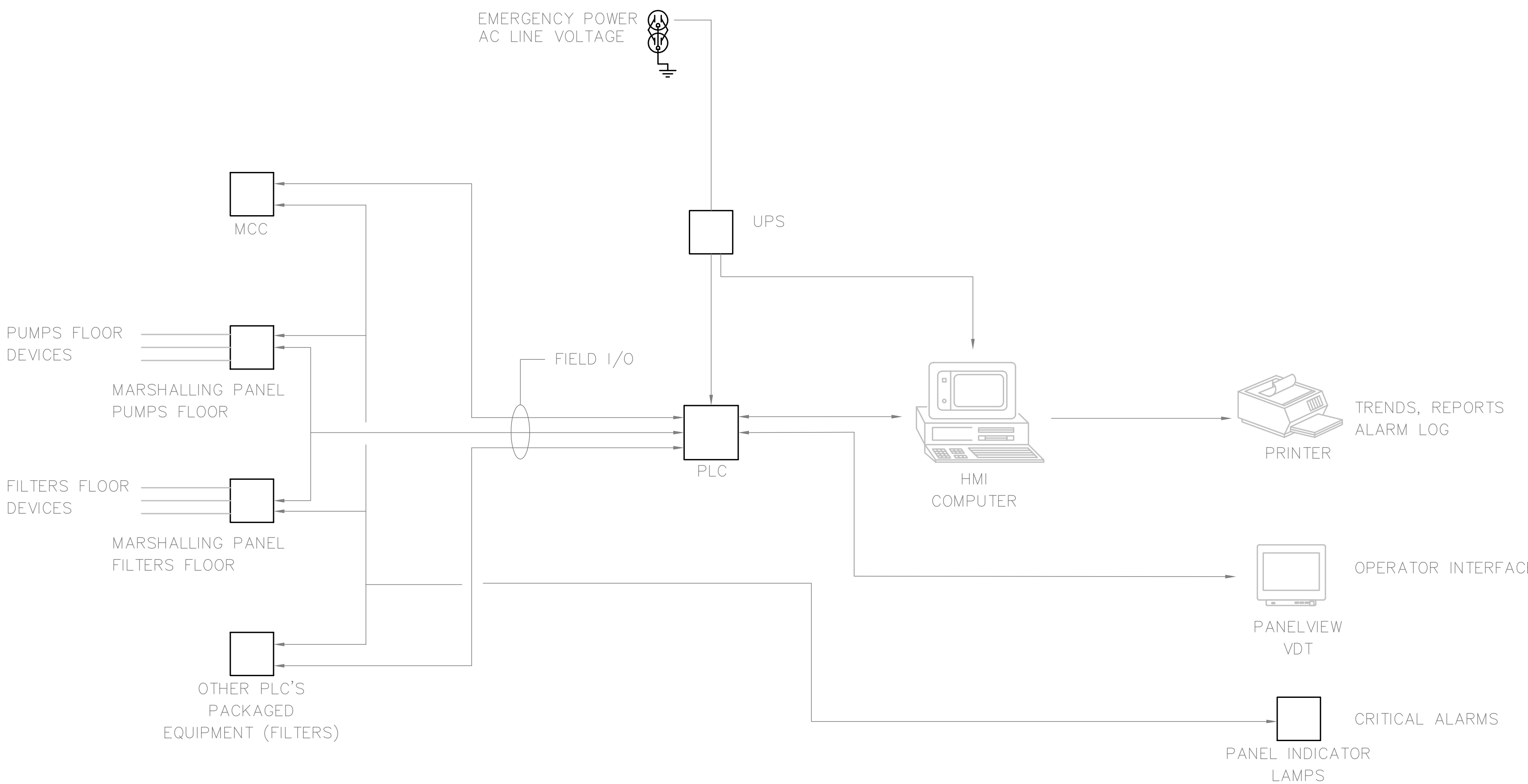

NO.	REVISION	DATE	INITIAL

IQALUIT WATER TREATMENT PLANT

MUNICIPALITY OF IQALUIT
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EARTH TECH

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4.0 ARCHITECTURAL/STRUCTURAL

4.1 Proposed Renovations

It is proposed to upgrade the existing water treatment plant utilizing as much of the existing plant as possible. Revisions to process flow pattern and storage requirements will necessitate the removal and additions of several concrete walls and baffles. Part of the upgrade program will also include the addition of four (4) new filters along with a filter gallery. These filters will be placed on the existing concrete structure forming the roof of the exterior clear water well.

The roof elevation of the clear water well matches the level of the existing pump floor at elevation 313'-0" (95.4 m). At present, approximately 2.0 metres of earth fill covers this roof structure at the exterior of the building.

The proposal would be to remove this fill and construct new 250mm thick concrete walls on two sides of the clearwell matching the existing well perimeter (see drawings following this section of the report).

A new timber superstructure would be constructed on top of the concrete walls. The new roof would connect to the existing concrete beams at the level of the existing roof, elevation 335'-0" (102.108 m).

At completion of roofing and waterproofing the wall, fill material would be replaced against the new wall structures.

The new wall structures would be designed for the earth fill pressure and the roof would be designed for snow loads applicable at this site (2.4 kPa or drift loads as prescribed by the NBC).

4.2 Existing Conditions

The existing water treatment plant structure was inspected by Jim Patrick of Reid Crowther on February 2, 3 and 4, 1998. Original engineering drawings of the plant were available for review. These drawings were dated April 1962, so the plant is presumed to be approximately 39 years old.

The existing plant is a cast-in-place concrete structure. Drawings show the required concrete strengths of 3000 psi with intermediate grade reinforcing steel (40 ksi).

It consists of three primary levels, with an exterior superstructure covering the upper level. The tank walls, upper levels and roof are cast-in-place concrete.

Construction consists of one way slabs spanning between concrete beams and column supports.

The building is supported by walls and pedestals bearing on bedrock. The reservoir floors are elevated over a crawlspace. The elevations of the reservoir floor is 300'-0" (91.440 m). The other primary levels are the pump floor and the filter floor.

A visual inspection of the entire structure was completed and it is considered to be in good condition. Neither materials testing nor other non-destructive testing was undertaken, however the structure was reviewed for cracking, movement or other distress. The well structures were full and thus the inside of the tanks could not be inspected.

The floor structures of the reservoirs could be inspected from the crawlspace, although a complete inspection was limited by the presence of free water in the crawlspace. This is from minor leaking, condensation and melt water from the exterior of the building. Nevertheless the foundation and floor structures are considered to be in good condition, and no significant cracking or sign of movement was noted. There is some minor cracking in the reservoir walls and we recommend a coating of Xypex on the inside of all wet surfaces during the plant upgrade.

The roof structure of the clearwell is a 10 inch (250 mm) thick one way slab spanning approximately 10'-6" from the exterior wall to an interior column and beam line, and then 10'-6" to the demising wall between the two clearwells.

The drawings show the exterior wall to be 18 inches thick with the centre beam (18" x 28") supported on two 18" x 14" interior columns within the clearwell. The drawings also showed the reinforcing steel detail, so the structure was checked for load capacity.

The structure has ample reserve capacity for new superimposed loads, and thus no reinforcing is necessary.

The building was not constructed in strict accordance with the engineering drawings. We believe it was intended that the insulated exterior wall be constructed on the demising line between the two wells, down to the elevation of the pump floor. Instead, the centre wall was constructed up to the level of the filter floor and the columns supporting the filter floor and roof superstructure were built as pilasters within this wall. This enabled the placement of about 2.0 metres of fill over the exterior clearwell roof and up against this wall.

4.3 New Construction

Filters

It is proposed that the new 250 mm thick walls be constructed with 30 MPa concrete and 400 MPa reinforcing steel. They will be placed directly on top of the two exterior clearwell walls and constructed to a height of approximately 3050 mm, matching the elevation of the filter floor. Above this would be 38x184 wood stud walls constructed to match the elevation of the existing adjacent roof.

The new concrete walls will be connected to the existing walls at the bottom and sides with epoxy set dowels. Reglets on each side of the wall will allow the installation of sealants. Although caulkable sealant bead materials are now available to simplify this task (Sikaswell), sealants exposed to turbulent water flows can erode from the joint over time.

These concrete walls will be waterproofed with a bituthene membrane carried 300 below the base of the wall. This will prevent groundwater penetration through the replaced fill. The top of this fill will be at approximate elevations 97.4 m. The exposed upper section of the wall would be covered with 50 mm styrofoam and treated plywood protection board up to the elevation of the filter floor. Exterior wood stud walls would be filled with 300 mm of batt insulation and covered with prefinished metal cladding to match existing construction.

The new roof structure over the filter floor would consist of pre-engineered 300 mm timber truss joists, spaced at 305 mm o.c. and sheeted with 20 mm plywood. They would be insulated with 200 mm of rigid roofing insulation (R40) and covered with plywood and a single ply torch applied membrane. The joists would span 6.4 m (21’-0”) between the new outside wall and the existing wall and would slope down to match the slope of the existing roof. The new wall structure would be designed to restrain the fill pressure and wind pressure against the new exterior walls and support snow loads. To accommodate additional load from the new roof structure, the existing concrete beam roof structure would be reinforced with steel columns, one per bay. The total area of new roof structure would be approximately 6.85 metres x 13.4 metres (91.8 m^2 or approximately 990 $ft.^2$).

Access from the existing pump floor into the newly created filter space would be provided by a new opening in the existing exterior wall. This would require an 1880 x 2100 mm opening cut into the existing concrete wall. This will not require any reinforcing of the wall.

Maintenance catwalks would be placed around the new filters at the elevation of the existing filter floor. To provide access to the catwalks and to facilitate connecting the new roof structure to the existing roof structure, the existing filter floor exterior wall panels would be removed.

Tank Configuration Changes

Due to increased design flows, the existing tanks will be modified slightly to accommodate process requirements. A number of walls and channels will be removed and new concrete walls will be added to take advantage of the existing concrete works. This work includes the removal of about 28.3 cu.m. of existing concrete and the addition of 22.3 cu.m. of concrete walls and baffles. Several small infill sections are expected, along with additional ports to allow re-routing of the flow.

4.4 Building Shell Retrofit

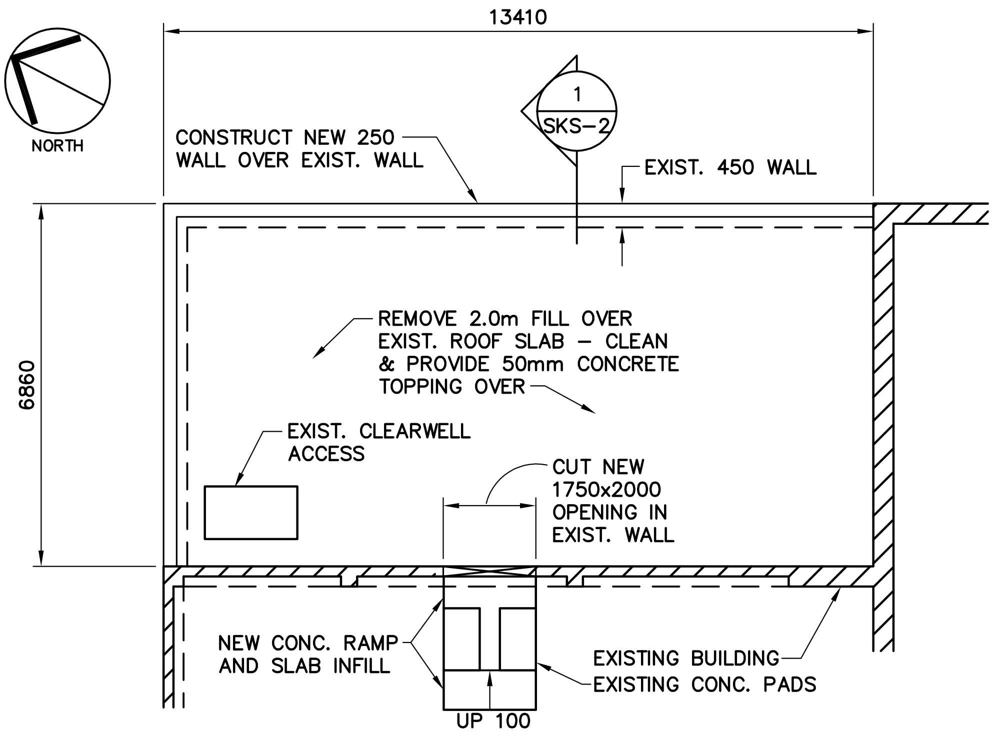
The existing building walls do not meet current energy efficiency requirements and we are recommending a complete envelope upgrade.

The existing building sandwich panel walls will be covered with a 10 mil vapour barrier secured with 19 mm x 89 mm vertical strapping at 600 o/c. 9.5 mm exterior sheathing and 100 mm Type II rigid insulation will follow the interior strapping. The final wall components will be an air barrier, followed by 19 mm x 89 mm horizontal strapping at 600 o/c, and finally, metal siding coloured to match the existing water reservoir. New flashings will be provided as required. This wall system will provide at least R-20. The R-value of the existing sandwich panels is not known at this time.

In addition to the upgraded wall system, we recommend all exterior doors and windows be replaced with units suitable for cold climate conditions.

The roof appears to have been retrofitted as the original drawing indicate a flat roof while the existing structure has a low slope peaked roof. The details of this retrofit are being investigated, and will dictate the need or lack there of, for another upgrade. Our cost estimates include for a new R-40 rigid insulation roofing system, with a new torch-on membrane.

Typical wall and roof sections are presented in the Figures SK-2 and SK-3, at the end of this section.



PLAN AT PUMP FLOOR
ELEV. 313'–0" (95.400)
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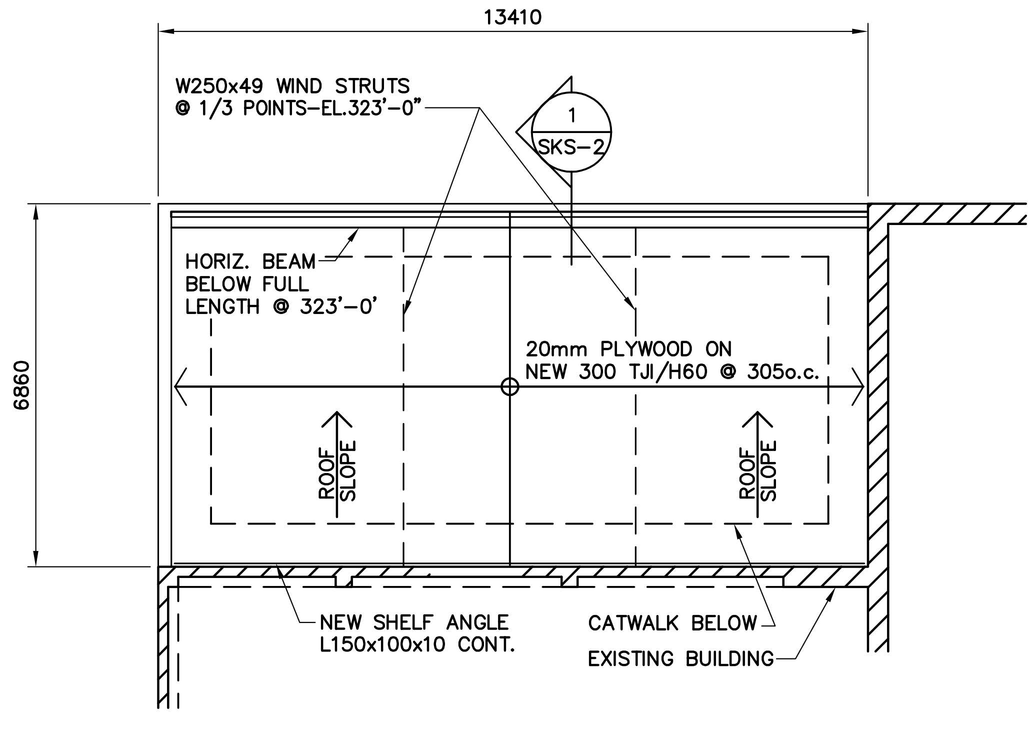

NEW ROOF PLAN
ELEV. 335'–0" (102.108) NOM.
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NO.	REVISION	DATE	INITIAL

IQALUIT WATER TREATMENT PLANT
FILTER ROOM ADDITION

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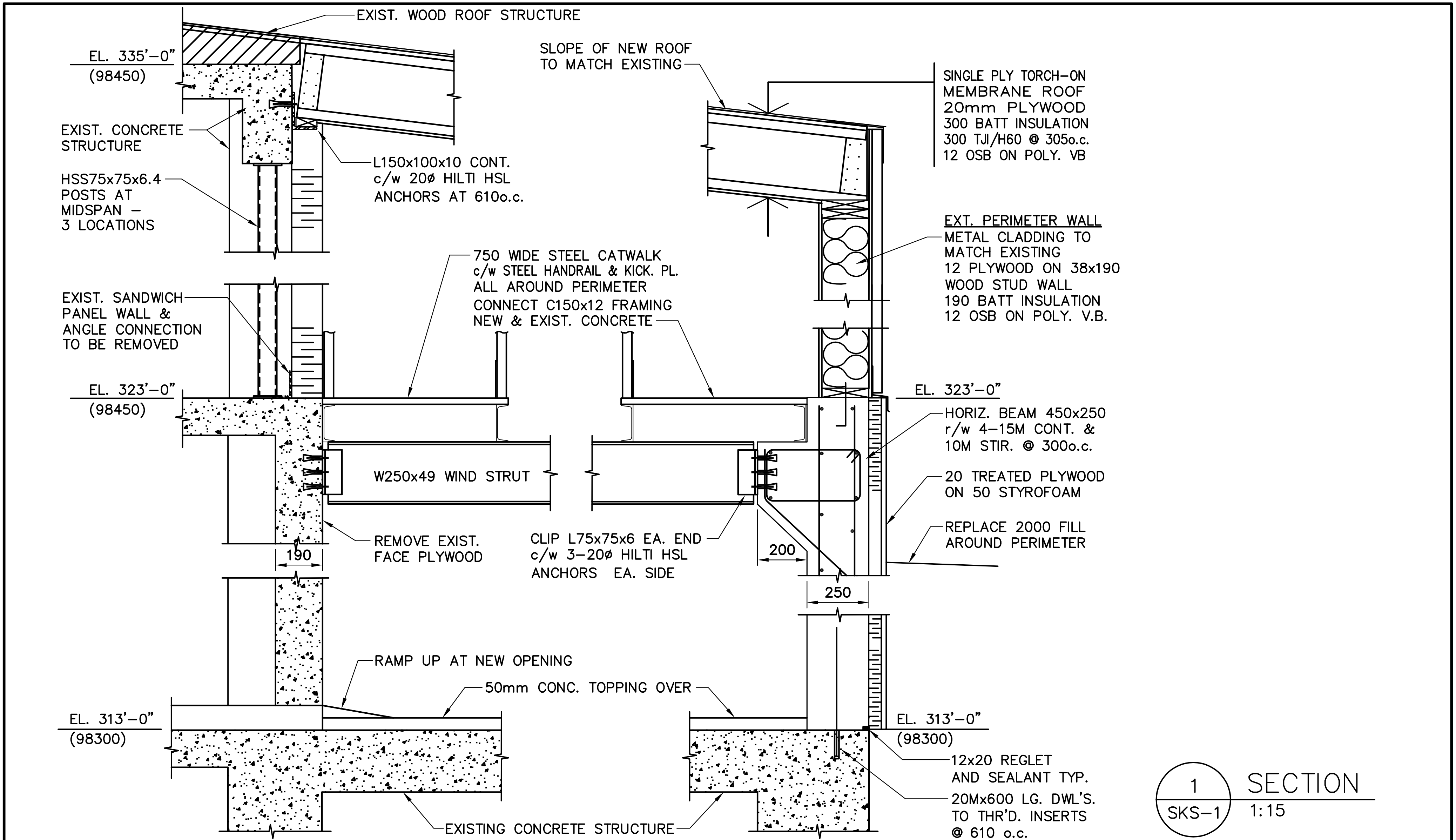

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IQALUIT WATER TREATMENT PLANT
FILTER ROOM ADDITION

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5.0 MECHANICAL

5.1 Introduction

The mechanical systems serving the Iqaluit Water Treatment Plant have, for the most part, reached the end of their useful economic service life. Three (3) notable exceptions are the two new glycol heating boilers installed in 1993 which remain in good repair, the recent upgrades to supply and backwash pumps, as well as the chlorine room upgrade. The new heating and ventilation system components installed will be expected to have a service life in excess of 20 years.

This report will address mechanical replacement strategies for heating, ventilation, safety and miscellaneous mechanical systems. The recommended components will meet all applicable codes, will meet the intended function of the facility, will be selected to meet the project budget while retaining the standards of quality and reliability required for this remote facility.

It is recommended that during the design phase, the possibility of utilizing heat recovery off the NTPC power house, be explored as a means to supplement the heating capacity of the existing boilers.

Codes And Standards

The following list of codes and standards have been utilized in the preparation of this report:

National Building Code 1995
National Plumbing Code 1995.
National Fire Code 1995.
CAN/CSA B-139 - Oil Burning Equipment Code.

Public Health Act - General Sanitation Regulations

Notes of the Water Treatment Plant Inspection performed by the Baffin Region Health Services Office in October of 1997.

Existing Services

The existing mechanical building services include two oil fired glycol heating boilers located in the Boiler Room, distribution pumping, glycol unit heaters and baseboard radiation throughout the facility and localized exhaust fans for ventilation. Poor ventilation has resulted in extensive corrosion of exposed mechanical components throughout the facility.

5.2 Heating

Reservoir Level

This level is heated by three existing glycol unit heaters located around the perimeter. All of these heaters and exposed piping are exhibiting corrosion. Coupling the observable corrosion to the age of the equipment, it is recommended that the equipment be replaced.

Three new glycol unit heaters will be provided with new insulated steel piping connected to the building glycol heating loop. The unit heaters will be provided with stainless steel components where possible for corrosion resistance.

Pump Floor Level

This floor area is presently served by four glycol unit heaters and one force flow heater in the stairwell. All of these mechanical items are beyond their useful service life and exhibit corrosion which is a result of the poor level of ventilation on this floor. Four new glycol unit heaters are proposed with one glycol forced flow heater in the stairwell all connected to the building glycol heating loop.

Filter Floor Level

This floor is presently heated by six unit heaters and one strip of radiation in the washroom area. All of these mechanical items are beyond their useful service life and exhibit corrosion which is a result of the poor level of ventilation on this floor.

Five new glycol unit heaters will be provided for the revised filter floor plan including the chemical storage room. Radiation will be provided for the office/laboratory room and the washroom.

New Filter Addition

This area will be a new area under this project. Heating requirements of this area will be served by two new glycol unit heaters connected to the building glycol heating loop. Unit heaters will be sized to maintain space temperature as per existing minimum building setpoints.

Reverse Return Hydronic Piping

Due to the corrosion condition of the original heating system distribution piping replacement is recommended. To maximize system efficiency and minimize the need for system balancing, the hydronic heating system will be designed as a reverse return (first supplied – last returned) system.

Valve Chamber at Dam

An electric unit heater on a thermostat will maintain the chamber above 0º C. This will assist in reducing the dangerous level of condensation observed on the existing non-vapour proof equipment.

5.3 Ventilation

Reservoir Level

This area is presently not provided with mechanical ventilation.

Useful ventilation by mechanical means cannot cost effectively be provided. Therefore, the mechanical equipment installed will have durable finishes to extend the useful service life in this environment.

Pump Floor Level

The inadequate ventilation on this floor has been noted in the Baffin Regional Health Services Report. This floor is not equipped with mechanical ventilation.

A new glycol heated make-up air unit and one exhaust fan will be provided for general ventilation of the filter and pump floor levels. This system will be sized to provide 6 continuous air changes per hour on the pump floor level.

Filter Floor Level

This floor is provided with one exhaust fan in the washroom and one natural ventilation opening in the exterior wall. The present system does not provide adequate ventilation for this floor.

The new glycol heated make-up air unit and central exhaust fan will provide 6 air changes per hour on a continuous basis to this floor including the new filter addition.

Chlorine Room

This room has been provided with a dedicated ventilation system under a previous contract. This system is currently programmed to automatically initiate evacuation mode of operation upon a high limit detection level. Current standard practice is to replace this automatic mode with a manually initiated evacuation mode to ensure safe chlorine releases. This modification to the existing system will be provided.

Chemical Storage Room

The Baffin Regional Health Service Report recommends this room be provided with identical forced mechanical ventilation systems as in the chlorine room. However, since this room will no longer contain chlorine storage, continuous ventilation of 6 air changes per hour only will be provided. Heating requirements will be served by a new glycol unit heater.

5.4 Safety Issues

Personal Protective Equipment

As noted in the Baffin Region Health Services report, the following items will be reviewed and upgraded to improve workplace safety:

- Standard Operating Procedures for safe use and handling of controlled products.
- Emergency procedures for protection of workers in the event of a controlled products spill.
- Instruction to workers on work and emergency procedures for controlled products.

- Chlorine detectors and alarms have been provided for the chemical storage room and the pump floor level under a previous contract. Annunciation has been provided both inside and outside the building according to previous project consultant.

- Responder "Level A" vapour protective clothing will be made available on site for chlorine emergencies.

- Four 'Self Contained Breathing Apparatus' (SCBA's) will be provided on site. Two will remain operational at all times while two can be out of the building for service.

Asbestos Removal

The existing NTPC tunnel piping is known to have been covered with asbestos fibre insulation but is believed to have been removed. Should any further asbestos be found at the site, it should be removed according to asbestos abatement standards and disposed of according to code and City regulations.

Existing original piping within the water plant may also have asbestos insulation which, again, should be removed and disposed of prior to new construction at the plant. This cost for both items is not included in the estimates and is not part of this contract.

5.5 Miscellaneous

Fuel Oil Day Tank Dike

The existing indoor fuel oil day tank is not equipped with a containment dike to meet code requirements. The tank is also mounted above floor level using a steel frame which does not meet the minimum fire code requirements. To meet code requirements, this frame would need to be replaced with a structure to maintain its integrity during a fire for a minimum of two hours.

Due to the age of the tank and the upgrades that are required, we would propose that the oil day tank is replaced with a ULC approved self-contained storage tank with a pumped supply and be mounted at floor level.

Chemical Storage Area Construction

A new chemical storage area will be provided on the west side of the main floor. The area will be upgraded to include fire rated drywall, fire dampers and fire stopping will be provided for mechanical penetrations. A new exterior door will be installed with a window for observation from the exterior. This window will be plexiglass over georgian (wired) glass to deter vandalism.

NTPC Domestic Water Service

The existing 80 mm water service supply and return to NTPC will be removed and replaced with a new standard municipal domestic water service.

Removal of Previously Abandoned Equipment

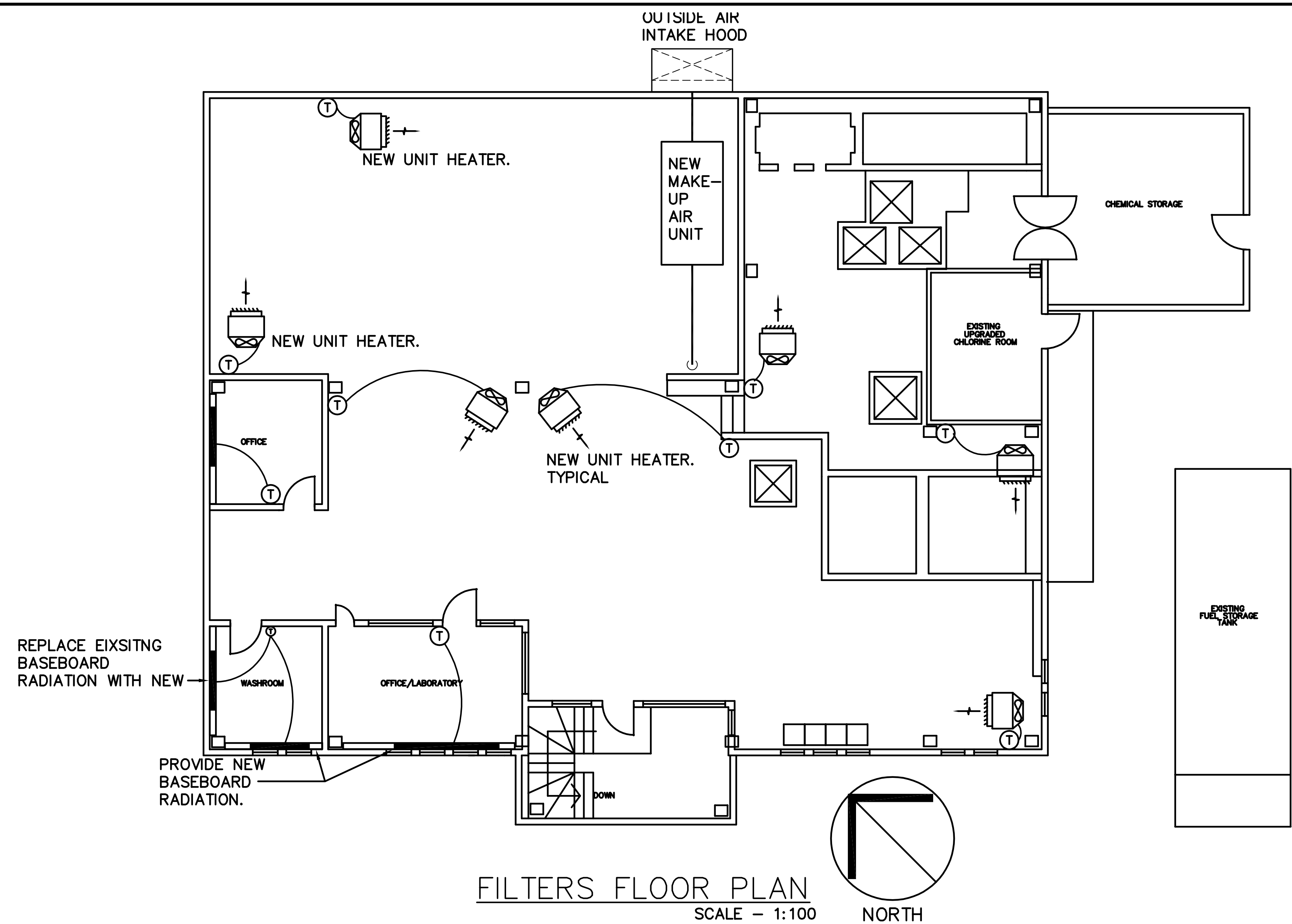
Various pumps, piping, compressors, and other equipment no longer in service or required in the treatment plant will be removed.

Raw Water Tempering

It is recommended that the existing raw water intake capacity be upgraded to meet the current and future demand requirements. It is therefore necessary to upgrade the existing tempering facility to include the additional intake capacity. In 1998, the existing tempering water installation was upgraded to include a 150mm line to the valve chamber. This provides the capacity for both the existing intake requirements as well as the new requirements. It may, however, be necessary to upgrade the pumping arrangement for the increased flow requirements. This will need to be determined during the final design phase using heat loss calculations.

5.6 Mechanical Sketches

- M-1 Filter Floor Plan
- M-2 Pump Floor Plan



FILTERS FLOOR PLAN

SCALE – 1:100

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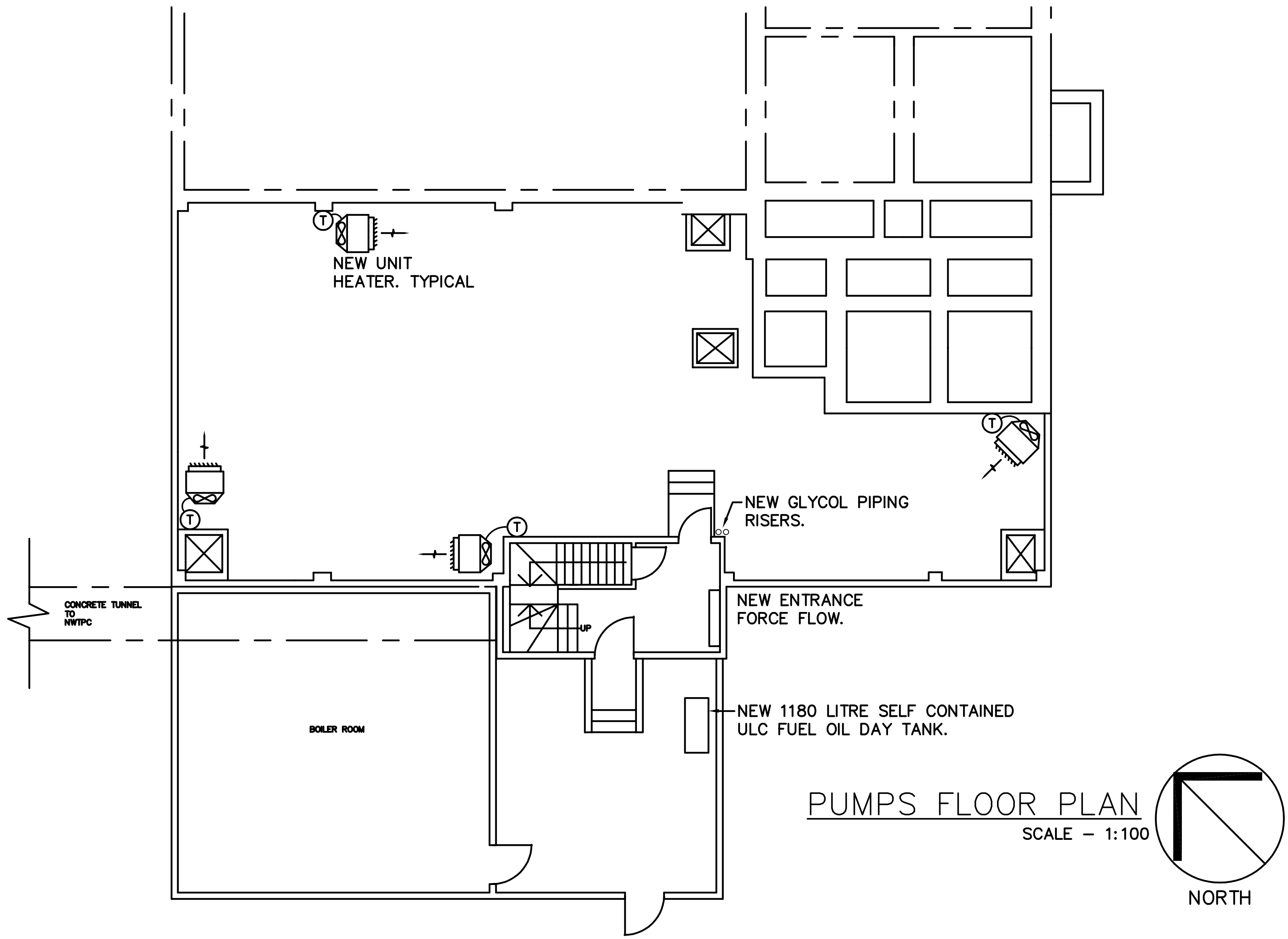

PUMPS FLOOR PLAN

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6.0 ELECTRICAL

6.1 General

The electrical systems and devices installed in the building have reached the end of their life expectancy. Components required to repair or upgrade existing equipment are non-existent or hard to come by. It is the intention that new components be installed to replace most if not all electrical systems and devices. New systems and devices installed will have a life expectancy of at least twenty-five years and will address all current code compliance issues. Where possible, existing conduit systems will be reused and new wire installed. Some conduits installed in the concrete slabs may have to be re-routed overhead due to the conduit having rusted away.

The purpose of this report is to outline the proposed electrical systems for the Iqaluit Water Treatment Plant.

The selection of these systems is based on following criteria:

National Building Code of Canada 1995
Canadian Electrical Code, CSA C22.1-98
CAN/ULC S524, Standard for the Installation of Fire Alarm Systems
CAN/ULC S537, Standard for the Verification Testing of Fire Alarm Systems

Local Authorities having Jurisdiction

The selected electrical systems will:

- Meet the functional and environmental requirements of the facility through effective lighting, power and communications.

- Meet the present capital budget

- Reduce operating costs by utilizing low maintenance energy efficient systems where practical within the present budget.

- Provide reliable system operation with technology backed by proven service agencies.

6.2 Site Services

At present, the power service for the Water Treatment Plant runs from the Nunavut Power Corporation Powerhouse through the tunnel and into the existing switchgear located on the pump floor. Nunavut Power Corporation has indicated that the present power service to the building should be taken out as it does not conform to current code requirements. The new power service would be provided overhead complete with a new pole mounted transformer bank. The building would also be fed with a new overhead telephone service dedicated to the Water Treatment Plant.

Discussions with Nunavut Power Corporation continue to indicate they will "contribute" this work and materials. The Nunavut Power Corporation has also indicated that during the design phase they will provide Earth Tech (Canada) Inc. with all technical requirements related to the installation of the new service including the metering equipment specifications. This will be finalized during the design phase and any work remaining outside of that provided by the utility, will be incorporated into the Tender Documents.

6.3 Power Distribution

The existing electrical service for the building is located on the pump floor. It is a 400 amp 600 volt Westinghouse CDP type. Due to the age of the CDP and NTPC requirements to refeed the building, a new MDP/MCC lineup is proposed to be installed on the filter floor.

The main electrical service will be sized at 600 amperes, 600 volts, 3 phase, 4 wire. To accommodate the expanded filtration requirements and future expansion requirements, spare capacity will be allowed for in the circuit breaker lineup.

The main power distribution will configured as follows (Refer to attached single line drawing and Distribution Equipment Elevation drawing):

- Utility provided 3 phase transformer.
- A main circuit breaker with adjustable solid state tripping.
- A normal power MDP utilizing thermal/magnetic trip molded case circuit breakers.

An adjacent MCC with normal and emergency powered sections. Each MCC cell will be comprised of motor starters, hand-off-auto switches, and motor run/off/trip, indicating lights.

Transformer cubicles built into the main distribution to eliminate conduit runs to separate transformers and to ease maintenance requirements.

A 120/208 panelboard to provide branch circuit wiring to the filter floor.

A new panelboard utilizing bolt-in circuit breakers will be provided on the pump floor replacing the two existing panels.

6.4 Emergency Power System

At this time, it is felt that the WTP would have an emergency power requirement. The emergency power system would be used to ensure that the plant control systems, building heating systems and the tempering boilers remain in operation at all times. The source of standby emergency power could come from one of two sources: the first possibility is a stand alone diesel generator; the second possibility is a connection to the standby power system within the Nunavut Power Corporation's adjacent powerhouse. The latter of these two options would be less costly, provided an access agreement could be negotiated with Nunavut Power Corporation.

6.5 Mechanical Equipment Connection

Existing motor control starters are comprised of a 600 volt disconnect and a 600 volt starter with two overloads. Current code requires that 3 phase motors be protected with three overloads (one per phase). It is proposed that the existing motor starters and disconnects be removed and all new motors .56 kW or larger be fed from starters housed in the proposed MDP/MCC lineup. Each motor would also be fitted with new isolation disconnects mounted next to the motors.

Power connections to mechanical equipment will be completed as follows:

Motors .373 kW and less — Manual motor starter. A relay will be provided if automatic operation is required.

Motors .56 kW or more — A magnetic motor starter installed in a Motor Control Centre (MCC) and isolation disconnects.

Packaged Units c/w integral Circuit breakers installed in the MCC controls/ motor starters.

The MCC will incorporate low voltage and single phase protection relays that will disconnect equipment under poor power conditions.

6.6 Branch Circuit Wiring

All conductors will be copper, RW-90 insulation, minimum size # 12 AWG. Wiring methods will be as follows:

Rigid Galvanized Conduit	Exposed conduit subject to mechanical injury.
EMT Conduit	General concealed and surface runs.
BX Cable	Horizontal single circuit runs in partitions; drops to luminaire from the conduit system.
Liquid Seal Conduit	Connections to mechanical equipment and transformers.
Teck Cable	Connections to mechanical equipment, panelboards, exterior devices.

Electrical devices installed in specific use areas will be recessed or protected with wire guards or lexan shields.

Generally, building utilization voltages will be as follows:

120 volts	Small motors .373 kW or less, duplex receptacles, lighting.
208 volts single phase	Specific supplied equipment.
208 volts three phase	Motors .56 kW and larger, specific supplied equipment.
600 volts	Motors .56 kW and larger specific supplied equipment.

6.7 Lighting

Existing lighting is 2 lamp florescent pendent mount fixtures. These fixtures utilize magnetic ballast's and F-40 T-12 lamps. Acrylic reflectors are broken and some are missing. It is proposed that new fixtures be installed with T-8 lamps and electronic ballast's. The new fixture will be totally enclosed and gasketed which will prohibit the entrance of environmental contaminants.

To ensure an energy efficient installation that meets the functional requirements of the facility, the following light sources will be used:

Lamp Source	**Areas of Utilization**
HID-High Pressure Sodium	Exterior lighting
Fluorescent-Standard T-8 lamp	Throughout the building interior

Fluorescent fixtures installed throughout the building will be two lamp vapour proof type with a polycarbonate refractor.

Exterior lighting will be provided by surface mounted High Pressure Sodium fixtures with lexan shields to deter vandalism.

Lighting control will be as follows:

Exterior:	Photocell/timeclock, hand-off-auto exterior lighting, contactor cabinet
Interior:	Line voltage switches

6.8 Life Safety Systems

All existing life safety systems will be upgraded to ensure that all current codes and standards are met. Revisions will include the emergency lighting system, illuminated exit signage, and replacement of the building fire alarm system.

6.9 Emergency Lighting Units

Existing emergency lighting units do not provide the required coverage when the electrical service is interrupted. New battery packs and additional remote heads will be installed to ensure proper coverage throughout the building.

Emergency lighting will be provided by self contained, battery operated emergency lighting units at all exits and access to exits.

6.10 Exit Signs

Exit signs will operate at 120 VAC when the normal power supply is available and have a second connection to the emergency lighting battery packs, to operate in event of a power failure. Illuminated exit signs will be provided at all exit doors and as required by the National Building Code. Exit signs will be constructed of extruded aluminum and utilize LED type lamps.

6.11 Fire Alarm System

The existing fire alarm system will be upgraded and additional devices will be added throughout the facility incorporating the following features:

A fire alarm control panel at the main entrance to the Water Treatment Plant.

Pullstations at all exit doors and where required by the National Building Code.

Thermal detectors where required by the National Building Code and local authorities having jurisdiction.

Duct detectors in all supply and return ducts of major HVAC units.
Horn / strobe signal devices, capable of generating a temporal sound pattern.

6.12 Telephone System

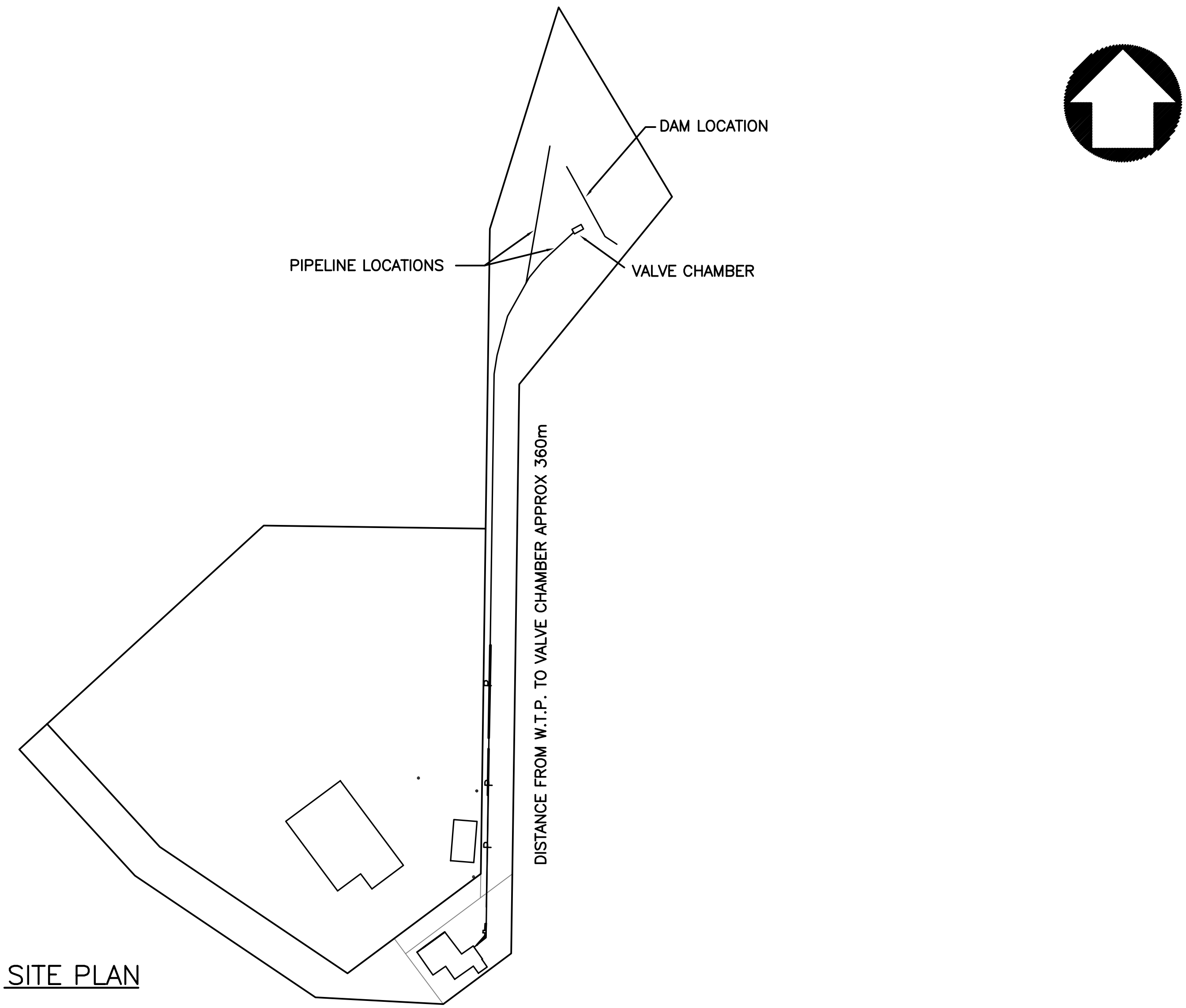
A new telephone service will be installed in the building. The existing telephone located in the NWTPC tunnel will be abandoned. A new telephone panel will be installed and a complete telephone wiring system shall be provided consisting of outlet boxes, conduit, cabling, and single jack outlets, as required.

All telephone wiring will be run in conduits. All telephone conduit will be provided with ground bushings and bonded to ground. A ground bus will be provided at the telephone panel.

Consideration should be given to providing a UPS for the phone system, as well as a cordless system to allow the operator to travel throughout the plant during a call, as this will be useful during trouble shooting.

6.13 Electrical Sketches

- SKE-1 Site Plan
- Equipment Plan
- Elevations
- Single Line Diagram



SITE PLAN

NO.	REVISION	DATE	INITIAL

IQALUIT WATER TREATMENT PLANT

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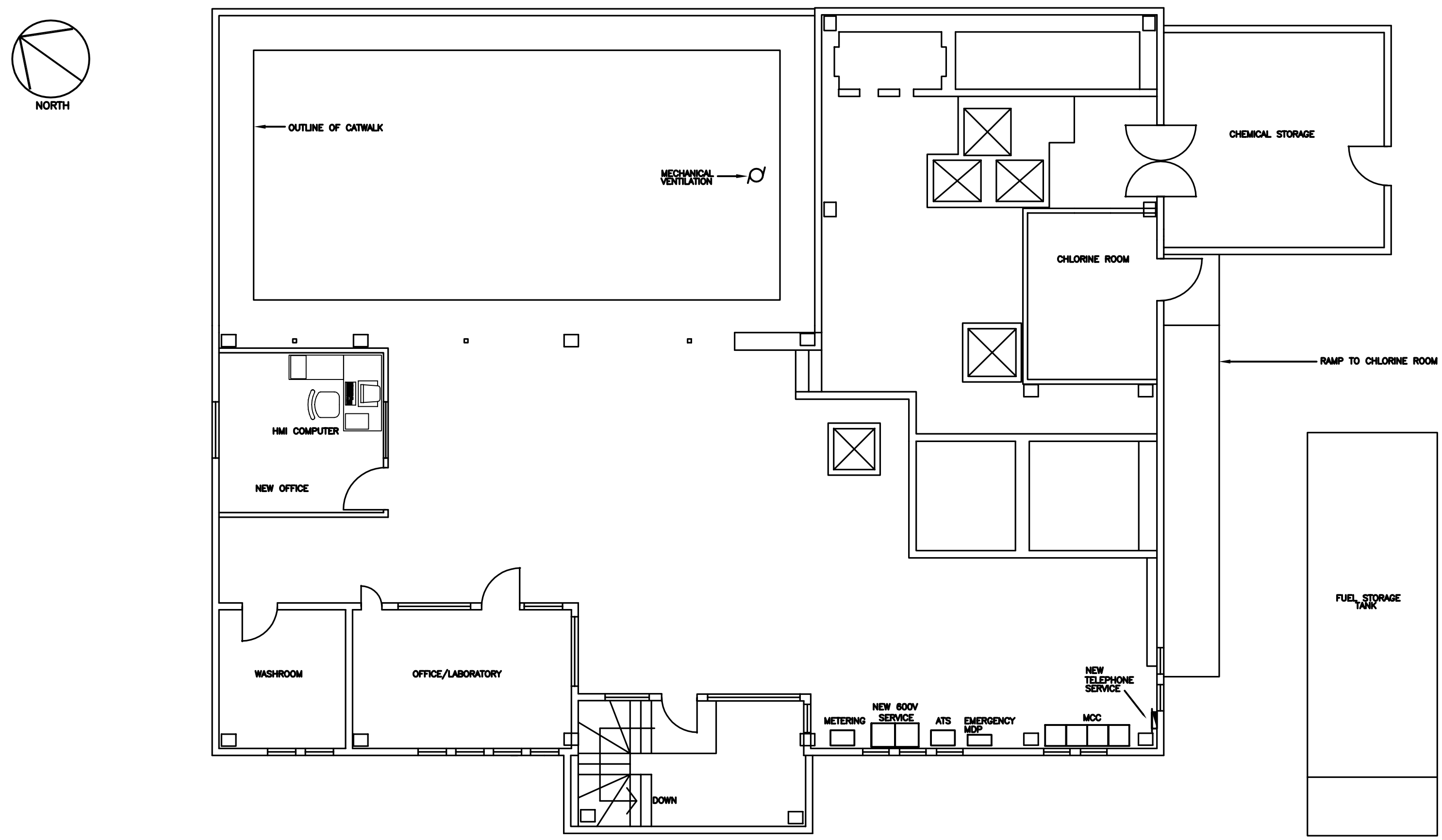

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IQALUIT WATER TREATMENT PLANT

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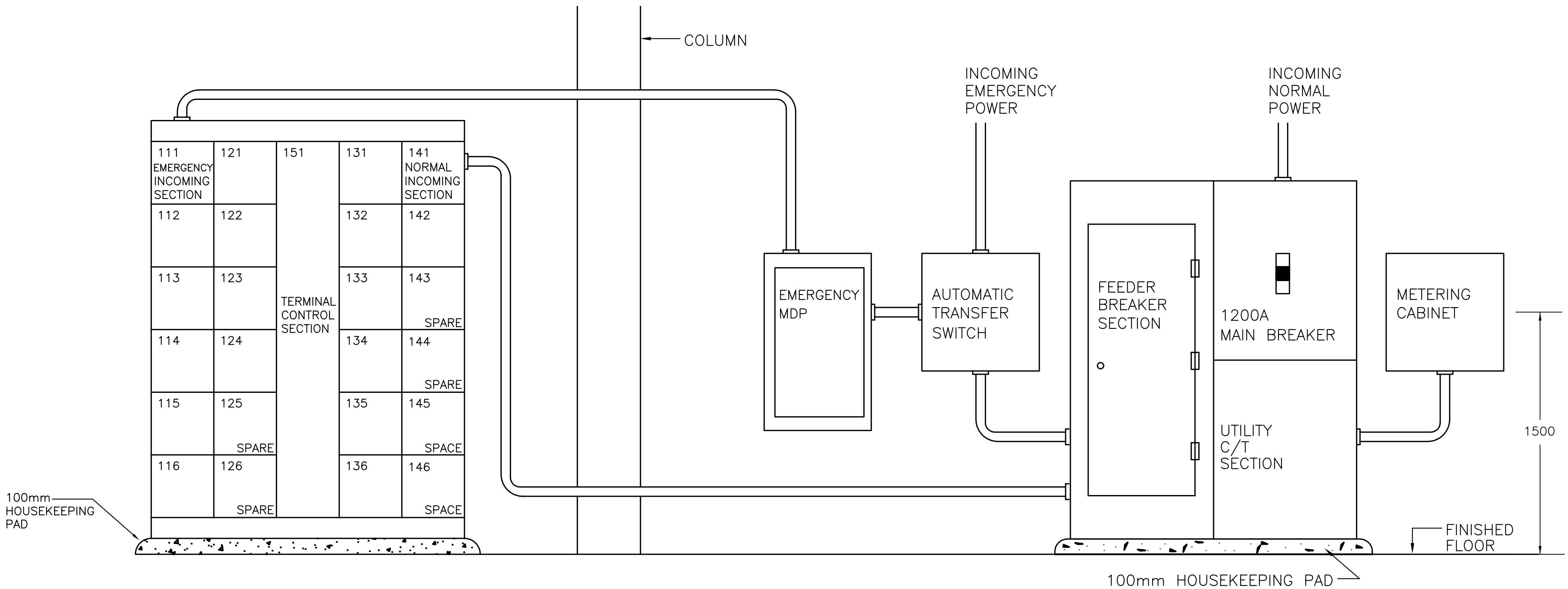

MCC ELEVATION

N.T.S.

MAIN DISTRIBUTION CENTRE ELEVATION

N.T.S.

NO.	REVISION	DATE	INITIAL

IQALUIT WATER TREATMENT PLANT

MUNICIPALITY OF IQALUIT
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EARTH TECH

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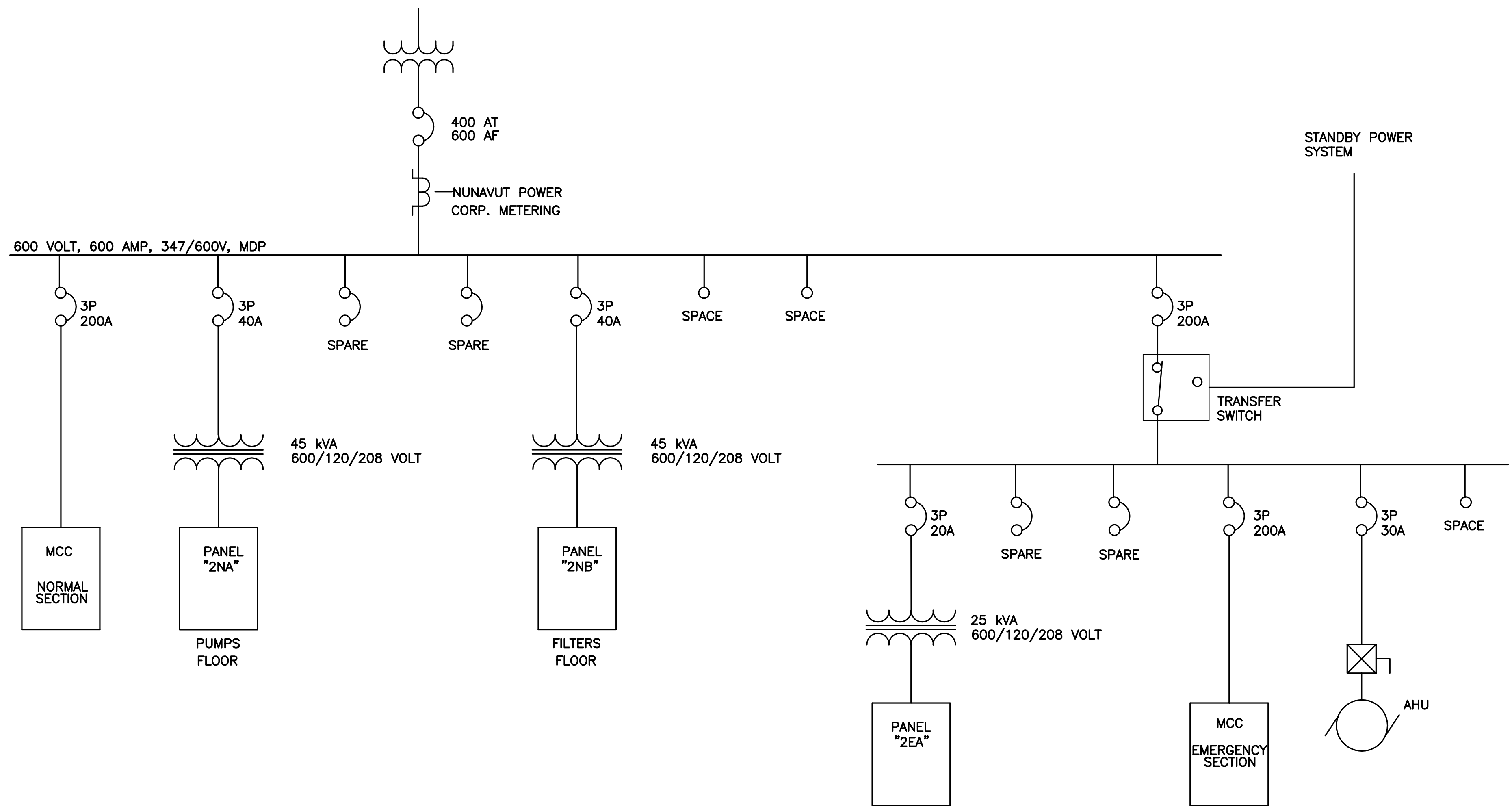

PROPOSED SINGLE LINE DIAGRAM

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7.0 INSTRUMENTATION AND CONTROLS

7.1 Control Philosophy

The control philosophy for the Water Treatment Plant (WTP) is to be executed by the major field instruments and control devices as shown on the attached Piping and Instrument Diagrams (P&ID's) A and B. The written description that follows should be read in conjunction with these drawings. (These drawings are duplicated from Section 3, and are at end of this Section.)

The control scheme will use the main storage reservoir level as the variable that will drive the plant flow. As the level in the reservoir drops the WTP inlet valve will open to increase the plant throughput, and as long as there is a demand in the distribution system, the plant inlet control valve will not close completely. As storage tank levels drop, the plant inlet flow control valve will open to respond to the increased demand. This change in the plant inlet flow is monitored by the inlet flow meter. The inlet flow meter will generate control signals for all active chemical dosing systems, allowing the chemical dosing rates to correspond with the new inlet flow rate. Depending on the time of year and the inlet water quality, some chemical dosing systems may not be required and will therefore be set in an inactive mode by the control system, thus ensuring only the required chemical dosing rates are altered by the control system.

Plant flow exiting the flash mixer will be routed through the existing flocculator and then through one or both of the new flocculator trains. This operation is controlled using manual valves located at the inlet to the old sedimentation tanks. Flocculator mixers are either on or off, accordingly. Flow exiting the floccuators is divided between the filter tanks that are on line.

Filters are of the constant head type. The level control valve on each filter discharge line strives to maintain a preset level in the filter. As head losses across the filter bed increase, due to the accumulation of trapped particles in the filter bed, the level control valve opens until it reaches a maximum open position. Once the flow control valve has been in the maximum open position for a set time or the filter level rises above a pre-set point, the filter will be removed from service and be set into a "ready for backwash" mode. Operator will have the option of manually initiating or allowing automatic backwash sequences. An on-line turbidity analyzer monitors filter effluent turbidity. One analyzer will be provided for each set of filters and will cycle between monitored filters. A high filter effluent turbidity will also place that filter in "ready for backwash" mode.

Wastewater generated during the filter backwash sequence will be routed to the backwash waste storage tanks. The available backwash storage volume is only sufficient to contain the volume of one filter backwash. Therefore, initiation of any filter backwash sequence will not be permitted until the waste storage tank is confirmed empty by the low level switch in the wastewater storage tanks.

Chlorine and fluoride will be added to the filter effluent as it enters the clearwell. Fluoride addition will be flow paced from the plant inlet flow meter. Chlorine addition will be manually adjusted to maintain a constant residual chlorine level at the treated water reservoir discharge, as detected by a residual chlorine analyzer. A pH probe will continuously monitor the clearwell effluent. The pH measurement will be used to control the addition of caustic soda, if required. Water from the clearwell then flows to the treated water reservoir. The clearwell and reservoir levels are currently monitored by a Magnetrol ultrasonic level detection system. The level detection system will be maintained as it is of current design and has a good future service life. As water exits the reservoir the flow is measured by an existing magnetic flow meter. The flow meter will also be maintained as it is of current design and has a good future service life. A residual chlorine analyzer will also monitor the water as it enters the distribution system.

7.2 Programmable Logic Controller (PLC) Based Plant Control System

It is proposed that the plant control scheme be executed using a programmable logic controller (PLC) as the primary control platform. The plant will also be made operable in manual mode, in event of a PLC failure. Manual mode operation in event of a PLC failure will be based on the manual operation of motor stop/start, manual operation of valves, and the ability to see the key plant status alarms / indicators that are wired directly to the control panel.

The configuration of the PLC based control system is outlined in the attached PLC Schematic drawing. As shown in the drawing, the system consists of several functional blocks including: Field devices, PLC, operator interface, critical alarm and status indicators, Human Machine Interface (HMI) computer and printer, and uninterruptible power system. Each of these functional blocks are discussed in greater detail below.

7.3 Programmable Logic Controller

The PLC selected for this application will be specified to be by a known manufacturer with a track record of rugged and reliable technology and would have the following features.

- Be of modular design. Modular design will allow for the removal/replacement of modules with the system on-line without causing disruption to cabinet wiring or other operating modules.

- Be equipped with power supply, processor, memory, analog input/output, and discrete input/output modules.

- Be expandable to incorporate a sufficient number of modules to allow for a maximum of 250 input/output signals (in any mix of analog or discrete signals).

- Be 120 volt AC powered and have an internal battery back-up for the memory module.

7.4 Operator Interface

The operator interface will provide a single location for the operator to interact with the entire control system. The Panelview video display terminal (VDT) will be configured to provide both graphic and text based system status information. The operator will interact with the system using both the touch screen and the front panel keypad. The information provided on the Panelview VDT will be real time and represent the current plant status: Trending alarm logs and reports will not be available on this device.

7.5 System Communications

The PLC will communicate with both the Panelview VDT and the HMI computer using Ethernet protocol. Ethernet has been selected, as it is a current protocol that is flexible and easily configured.

7.6 Connections To Field Devices

Field devices will be wired to the main PLC location from four (4) main sources; the Motor Control Center (MCC), the field instruments located on the pump floor, the field devices located on the filter floor and from the controls systems provided with packaged equipment such as the filters. MCC signals will be discrete information such as motor status (running, stopped, tripped) or control signals to motor starters (stop, start).

The field devices located on both the pump floor and the filter floor will involve both analog and discrete signals. Analog devices will generate inputs to the PLC such as temperature, pressure, flow, and level values. The PLC will generate analog output control signals to flow valves, level valves, etc. Examples of discrete signals to and from the field devices are devices such as level switches, valve position switches and valve open/close signals. Where there are large numbers of field devices it is more efficient to install a marshalling panel in the area, and wire each device to the marshalling panel. A single multi-conductor cable is then routed to the main PLC cabinet. All multi-conductor cables will be provided with spare conductors to allow the addition of new field instruments.

7.7 Connections To Packaged Equipment

Some of the packaged equipment (such as the filters) may be provided with a package control system. In order to interface with this equipment, control ties will be made in one of two ways. If the packaged equipment is provided with its' own PLC, an Ethernet connection will be established between the PLC's. If the packaged equipment is not provided with its' own PLC, individual signals from with the equipment's control system will be intercepted and routed to or from the plant PLC.

7.8 Interconnection With Human Machine Interface (HMI) System

The interconnection with the HMI system will be provided by an Ethernet link. This link will allow the HMI system to continuously import control system status data.

7.9 Human Machine Interface (HMI) System

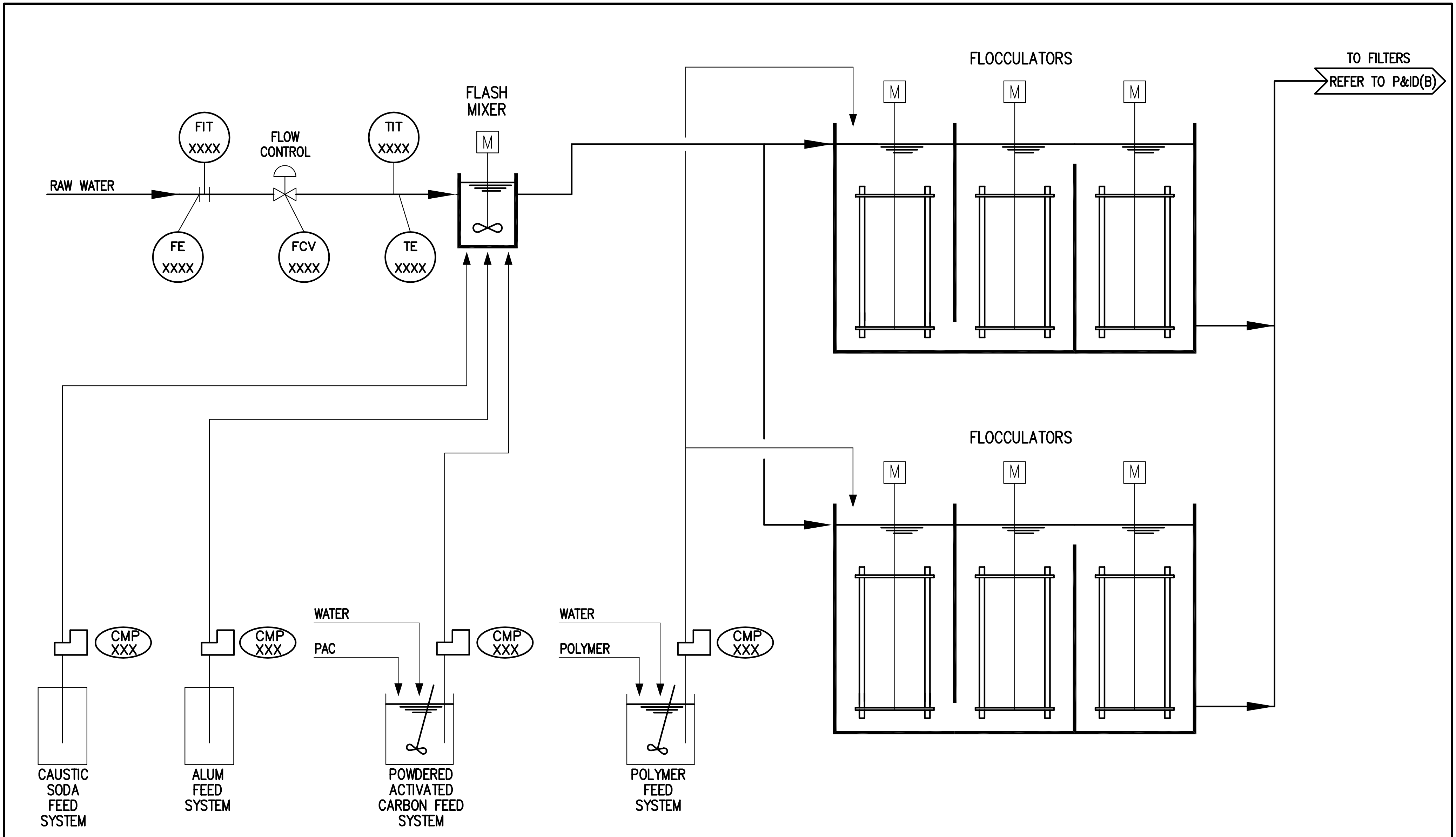
The design intent is to provide a HMI computer as part of the plant control system. The function of this computer is to continuously monitor the status of any or all process variables. The computer provides a view only location from which to monitor the plants operation; process changes cannot be initiated from the HMI computer. This data collection function subsequently allows the operator to view trends, generate reports, or review alarm logs in order to trouble shoot process upsets. It is important to note that the control system does not rely on the HMI system to operate. In event of an HMI failure, the PLC will continue to control the plant but all data recording and subsequent viewing and/or reporting ability is lost. The HMI computer hardware and software are readily expandable to include input from any other control devices used on the municipality's water distribution and tempering network.

7.10 Field Instruments

The field instruments selected for use in the water treatment plant will also be specified to be by a known manufacturer with a track record of rugged and reliable technology. Device accuracy, capital cost, ease of maintenance, and predictable service life will be considered when selecting instruments.

7.11 Sketches

Piping and Instrumentation Diagrams (P and ID) – A
Piping and Instrumentation Diagrams (P and ID) – B



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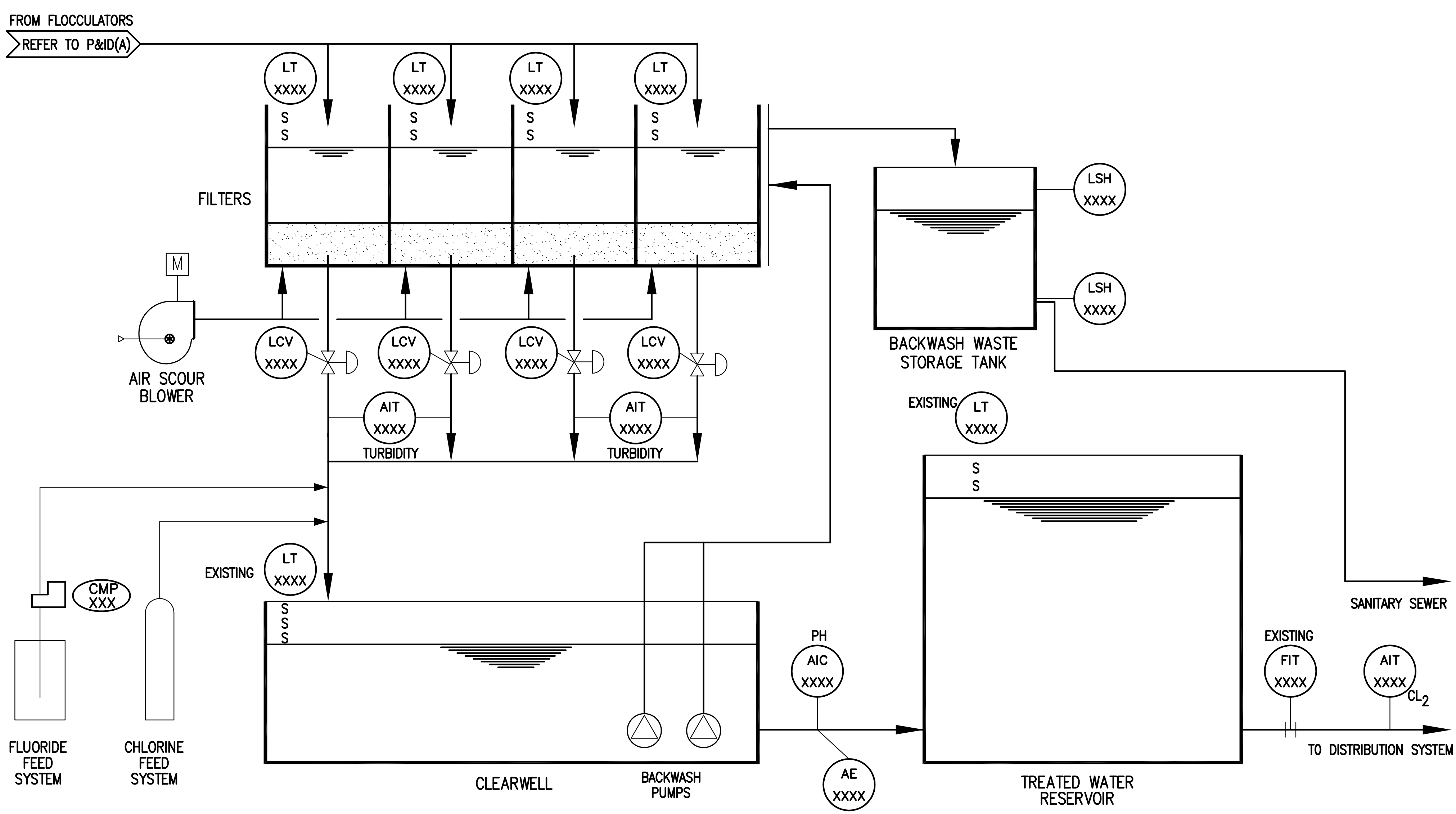

NO.	REVISION	DATE	INITIAL

IQALUIT WATER TREATMENT PLANT

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8.0 COST ESTIMATES

On the following page is the revised estimate, based on the updated design requirements, detailed in the report. The estimates as presented do not account for phasing of certain scopes of work such as the raw water intake replacement and the installation of the new filters, etc. This can be incorporated into future additions, as required.

Further, allowances for engineering and contingencies have not been included. For budget purposes we suggest the following:

Estimated Capital Cost	$2,960.000.00
Engineering at 15%	$ 435.000.00
	$3,335,000.00
Contingency at 20%	$ 665,000.00
Total Estimated Cost	**$4,000,000.00**

EARTH TECH CANADA ***Estimate***
Yellowknife, NT

Project: IQALUIT WTP UPGRADE Date: 3-Mar-04
Project No.: 49630 Estimator: M.Cronk

Dept.	Phase	Task	Description	Quan.	UofM	Material Cost		Total Cost
						Unit	Total Cost	
			ARCHITECTURAL/STRUCTURAL					
			Excavation/Backfill	350	m3	45	15,750	15,750
			Concrete (all inclusive)	30	m3	1,100	33,000	33,000
			Waterproofing New Walls	65	m2	125	8,125	8,125
			Wood Wall Systems	500	m2	200	100,000	100,000
			Wood Roof Systems	400	m2	210	84,000	84,000
			Misc Steel Walkways/Handrails	1	ls	50,000	50,000	50,000
			Chemical Loading Ramp	1	ls	20,000	20,000	20,000
			Misc Concrete Demolition	1	ls	25,000	25,000	25,000
			Xypex WTP Clearwells	1000	m2	25	25,000	25,000
			New Windows and Doors	1	ls	6,500	6,500	6,500
							subtotal >	**367,375**
			PROCESS					
			Raw Water Intake and Inlet Systems	360	m	500	180,000	180,000
			Flash Mixer	1	ls	15,000	15,000	15,000
			Floculators	6	ea	30,000	180,000	180,000
			Supply & Install New Filters	4	ea	300,000	1,200,000	1,200,000
			Vertical Turbine Backwash Pumps	1	ls	125,000	125,000	125,000
			Modify Struct. For Backwash Waste	1	ls	100,000	100,000	100,000
			Chemical Feed Systems	1	ls	130,000	130,000	130,000
			Process Piping Modifications	1	ls	150,000	150,000	150,000
			Asbestos Removal - NIC					
							subtotal >	**2,080,000**
			MECHANICAL					
			Heating System	1	ls	125,000	125,000	125,000
			Ventilation System	1	ls	105,000	105,000	105,000
			Safety Upgrades	1	ls	45,000	45,000	45,000
			Replace NTPC Service in tunnel	1	ls	5,000	5,000	5,000
			Chlorine Room					
			New Fuel Oil Day Tank Replacement	1	ls	5,000	5,000	5,000
							subtotal >	**285,000**
			ELECTRICAL					
			New Service Main Distribution	1	ls	30,000	30,000	30,000
			Transfer Switch & Emergency MDP	1	ls	15,000	15,000	15,000
			Motor Control Center	1	ls	35,000	35,000	35,000
			208v Motor Starters/Disconnects	1	ls	15,000	15,000	15,000
			120/208v Distribution Equipment	1	ls	15,000	15,000	15,000
			Interior/Exterior Lighting	1	ls	28,000	28,000	28,000
			Emergency Lighting/Exit Signage	1	ls	10,000	10,000	10,000
			Fire Alarm System	1	ls	25,000	25,000	25,000
			Conduit & Wiring	1	ls	3,500	25,000	25,000
			Telephone System	1	ls	3,500	3,500	3,500
			New Mechanical/HVAC Electrical	1	ls	12,500	12,500	12,500
			Heat Trace Systems	1	ls	11,000	11,000	11,000
							subtotal >	**225,000**
			INSTRUMENTATION/CONTROLS					
			Control Valves	1	ls	43,000	43,000	43,000
			Transmitters and Primary Elements	1	ls	58,000	58,000	58,000
			Analytical Instruments	1	ls	35,000	35,000	35,000
			Conduit / Wiring and Cabinets	1	ls	20,000	20,000	20,000
			PLC c/w Panelview VDT and Peripherals	1	ls	15,000	15,000	15,000
			HMI Computer and Peripherals	1	ls	4,500	4,500	4,500
			UPS and Software	1	ls	16,000	16,000	16,000
			Programming and Commissioning	1	ls	62,500	62,500	62,500
							subtotal >	**254,000**

Estimate Totals >	**2,957,375**

9.0 SCHEDULE

On the following page is one possible schedule based on an aggressive completion date. This schedule shows the final design starting early in 2002, and construction completed by June 2003.