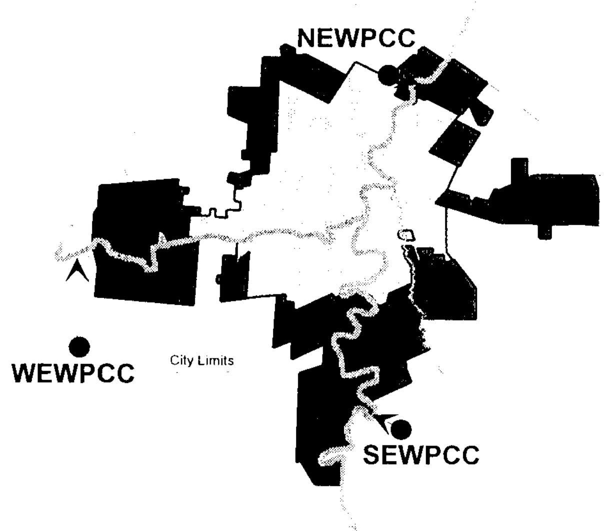
City of Winnipeg
Water and Waste Department

Combined Sewer Overflow Management Study

PHASE 2 Technical Memorandum No. 2

INFRASTRUCTURE/TREATMENT



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In Association With:

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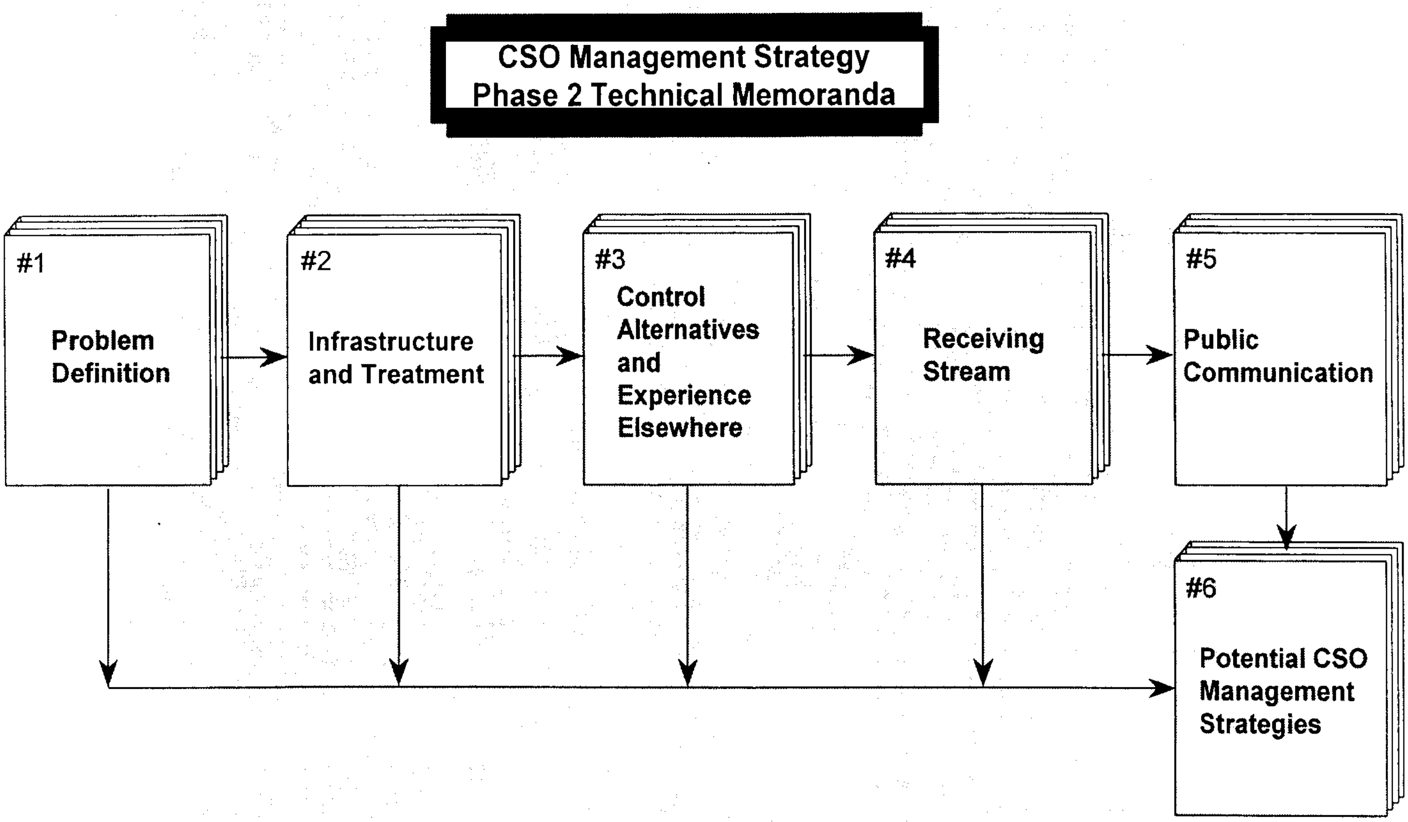
August 1995

PREAMBLE

This Technical Memorandum (TM) is one of a series of TM's intended for internal discussion. It is not intended as a report representing the policy or direction of the City of Winnipeg.

This particular TM is part of a group of Phase 2 reports as shown in the schematic.



Each of the Phase 2 TMs draws on information developed in the prior Phase 1 TMs. In addition, the Phase 2 TMs document information and study analyses sequentially. Ideally, therefore, the TMs should be read in the sequence shown.

ACKNOWLEDGEMENTS

The Study Team acknowledges, with sincere appreciation, the contribution of many individuals and agencies consulted in the course of Phase 2 of the CSO Management Study. The Study Team especially acknowledges the assistance of the City of Winnipeg Project Management Committee and the Advisory Committee.

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1.0 INTRODUCTION

This technical memorandum (TM) presents the Phase 2 analyses relating to the sewerage infrastructure, i.e., the interceptors and the treatment plants, particularly as these systems deal with wet weather flow (WWF) associated with combined sewer overflows (CSOs). Specifically, the sewer systems, the flood pumping stations (as these may represent opportunities to assist in CSO control), and the wastewater treatment facilities are discussed. The Phase 1 TM #2, "Infrastructure", and TM #3, "Treatment", provide useful background to this document.

2.0 SEWER SYSTEMS

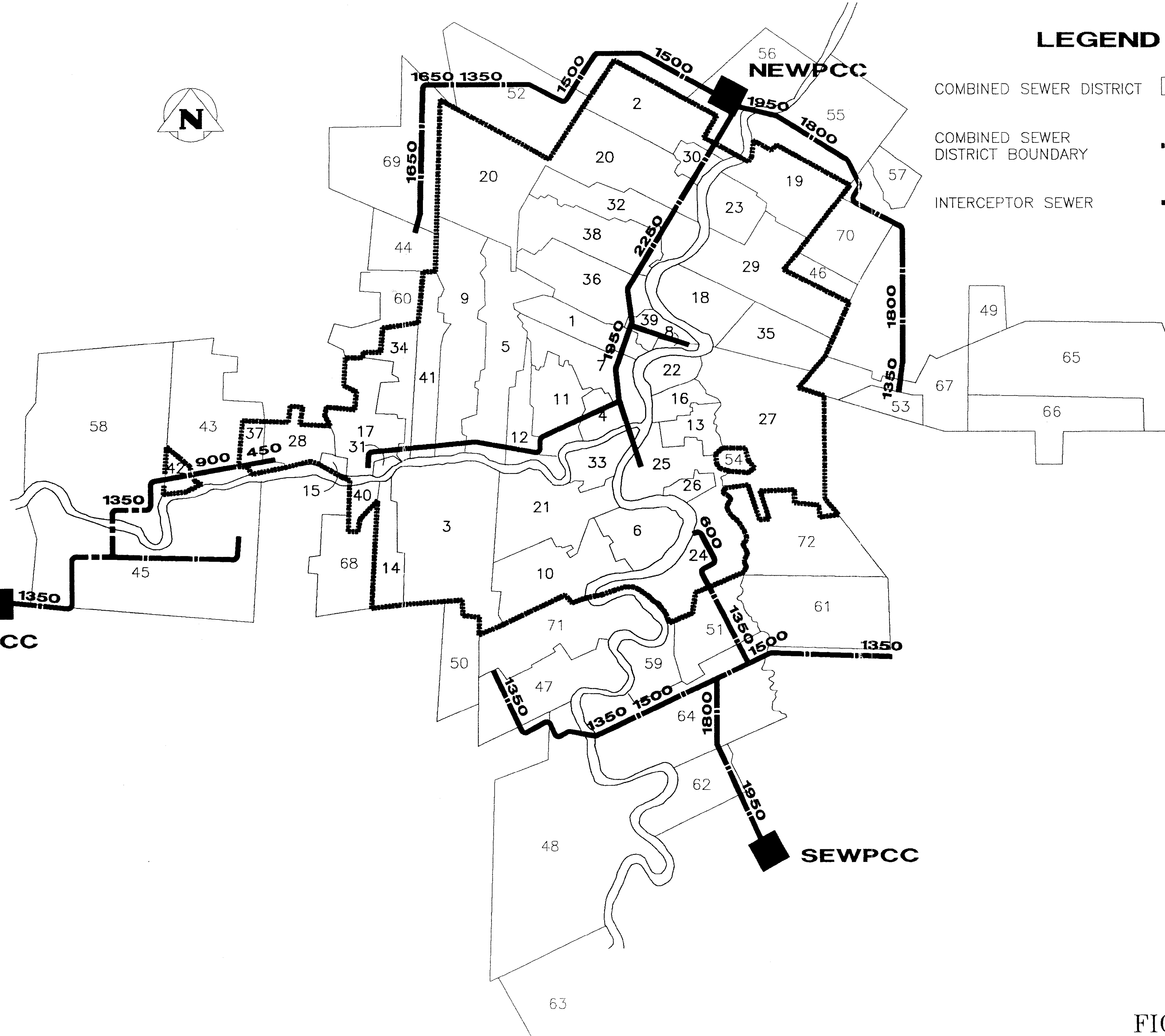
Wastewater in the City of Winnipeg is collected and conveyed to the water pollution control centres (WPCC) through three types of sewer systems namely, combined, sanitary and interceptor sewers. The combined and sanitary sewers collect the wastewater from the source and convey it to the interceptor sewer system. The interceptor sewers convey the wastewater from the individual sewer districts to the WPCC's. The sewer districts and the interceptor sewer system are shown on Figure 2-1.

2.1 COMBINED SEWERS

Combined sewer systems were installed in Winnipeg (and most other major Cities in North America) prior to the 1960s. These sewers convey both sanitary sewage and surface runoff. The sanitary flows are collected through service connections to the individual residences and buildings. Surface runoff is collected by catchbasins which discharge to the combined sewers. These combined flows are routed through the trunk sewer system, and, originally discharged directly to the rivers.

Interceptor sewers were built in the 1930s, along with the associated diversion weirs, diversion structures, pumping stations and secondary sewers, to convey the combined wastewater to the newly constructed North End Water Pollution Control Centre (NEWPCC).

COMBINED SEWER DISTRICTS		SEPARATE SEWER DISTRICTS	
1	ALEXANDER	43	AINSLIE
2	ARMSTRONG	44	BROOKLANDS
3	ASH	45	CHARLESWOOD
4	ASSINIBOINE	46	CHELSEA
5	AUBREY	47	CRANE
6	BALTIMORE	48	FORT GARRY
7	BANNATYNE	49	LAKESIDE MEADOWS
8	BOYLE	50	LINDEN WOODS
9	CLIFTON	51	MAGER SEPARATE
10	COCKBURN	52	MAPLES
11	COLONY	53	MISSION GARDENS
12	CORNISH	54	MISSION SEPARATE
13	DESPINS	55	NORTH KILDONAN
14	DONCASTER	56	NORTH MAIN
15	DOUGLAS PARK	57	OAKLEIGH
16	DUMOULIN	58	PARKDALE
17	FERRY ROAD	59	PULBERRY
18	HART	60	RIVERBEND SANITARY
19	HAWTHORNE	61	SOUTHDALE
20	JEFFERSON	62	SOUTH ST. VITAL
21	JESSIE	63	ST. NORBERT
22	LAVERENDRYE	64	ST. VITAL
23	LINDEN	65	TRANSCONA
24	MAGER	66	TRANSCONA CNR
25	MARION	67	TRANSCONA WEST
26	METCALFE	68	TUXEDO
27	MISSION	69	TYNDALL PARK
28	MOORGATE (CONWAY)	70	VALLEY GARDENS
29	MUNROE	71	WILLOW
30	NEWTON	72	WINDSOR PARK
31	PARKSIDE		
32	POLSON		
33	RIVER		
34	RIVERBEND		
35	ROLAND		
36	SELKIRK		
37	STRATHMILLAN		
38	ST. JOHN'S		
39	SYNDICATE		
40	TUXEDO (CHATTAWAY)		
41	TYLEHURST		
42	WOODHAVEN		



LEGEND

COMBINED SEWER DISTRICT 32

COMBINED SEWER DISTRICT BOUNDARY

INTERCEPTOR SEWER

FIG 2-1

ISTRICT.dwg

The system was designed to convey dry weather flow (DWF) and a nominal amount of WWF (approximately 2.75 times DWF) to the treatment plants.

The CSO Districts have a long history of basement flooding from intense summer storms. This is due to increased development and impervious area which have increased the volume and rate of runoff since the sewers were designed. The City has initiated flood relief programs to alleviate this problem. The relief measures have consisted of installing relief piping, catchbasin inlet restrictions and separation of sewers in selected areas. These programs are described in detail in the Phase 1 TM #2.

The CSO Districts had also been subject to basement flooding and sewer backup due to high river levels during spring flood conditions. This was alleviated through the installation of gate chambers (to isolate the system from the high river stages) and flood pumping stations to pump WWF to the rivers. The flood pumping stations are discussed in more detail in Section 3.0.

2.2 SANITARY SEWERS

Separate sanitary sewers and land drainage sewers have been installed in all new developments in the City since the 1960s. Overland stormwater runoff is channelled through catchbasins into the land drainage sewer (LDS) system. The LDS system conveys the runoff to the local surface water courses.

The sanitary sewers collect domestic, commercial and industrial wastewater and convey it to the interceptor system. Under DWF conditions, all wastewater is conveyed to the WPCCs for treatment. Under WWF conditions, stormwater enters the sewer system through foundation drainage (weeping tiles), infiltration, direct inflow and, in extreme cases, can cause sanitary sewer overflows (SSOs).

The weeping tile flows can contribute a major load to the sanitary sewers during wet weather periods. The magnitude of weeping tile flows are dependent on several factors, but the main factor is poor lot grading. Stormwater from the area surrounding the house and from rooftops will pond around the building, flow down the foundation wall to the weeping tile system, and then through the internal plumbing and service connection to the sanitary system. To improve this situation, the City has carried out an extensive householder information campaign on the benefits of providing positive lot drainage away from the foundation and extending the downspouts from the roof eavestroughs.

The practice of connecting weeping tile to the sanitary sewers is now prohibited by the City of Winnipeg Sewer Bylaw. In new developments, the weeping tile flow drains to a separate sump in the basement and is pumped to the surface. This flow is intended to flow overland and, eventually, enter the LDS system. In this way, the WWF conditions in the new sanitary sewer systems are dramatically reduced.

The sanitary sewers also convey other extraneous flows. This is made up of sewer infiltration and direct stormwater inflow (I/I). Infiltration occurs when groundwater enters the sewer system through joints in the sewer and connection piping and manholes. Studies (Wardrop, 1979) have shown that I/I can be a fairly significant component during wet weather periods. Direct stormwater inflows can enter the system through manhole covers (ponding on the streets) and through cross connections to the land drainage sewer systems.

The sanitary sewers in the City are currently designed based on the following criteria:

- Domestic Wastewater: 270 Litres per day per person;
- Peaking Factor: Harmon equation;
- Groundwater Infiltration: 2200 Litres per hectare per day;
- Manhole Inflow: 20,800 Litres per hectare per day;
- Commercial Wastewater: 28,000 Litres per hectare per day;
- Industrial Wastewater: Source dependant; and
- Weeping Tile Flow (in existing areas): 4.5 Litres per minute per house (approximately 78,600 L/ha/day).

The I/I design component (i.e. infiltration and inflow from manholes and weeping tile drains) is by far the most significant component and results in a design flow that is many times the average wastewater, or dry weather flow, component.

The sanitary sewers, in most cases, convey the wastewater to a sanitary pumping station. From these stations, the wastewater is pumped to the Interceptor sewer system. The pumping stations are designed on the same basis as the sewer system. Emergency overflows are built into the stations in the event of flows in excess of the pumping capacity or system failure. In this event, the sanitary sewer system overflows to the rivers or to land drainage sewers.

2.3 INTERCEPTOR SEWERS

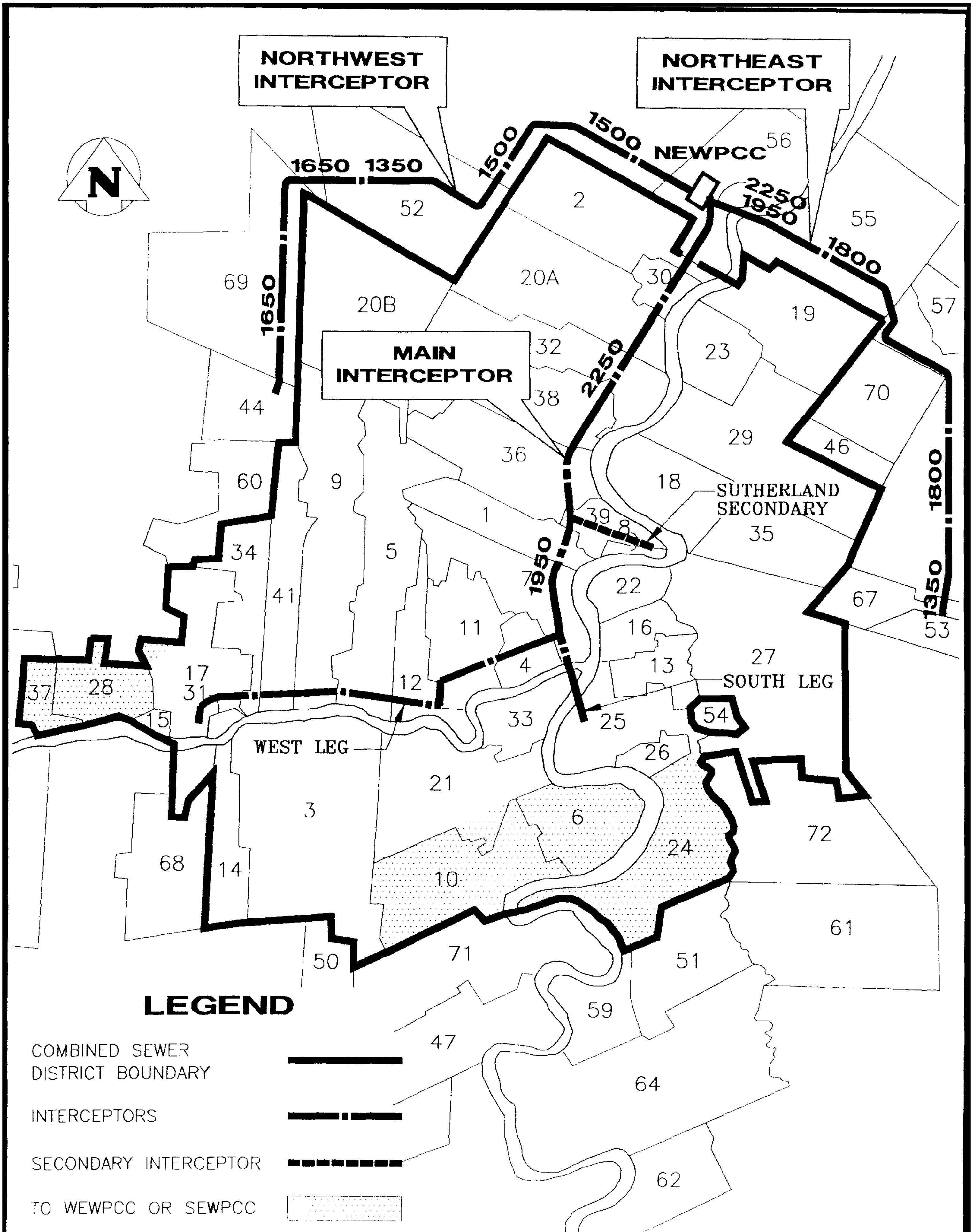
The Interceptor sewers convey the sanitary and combined sewage from the individual sewer districts to the WPCCs. There are 5 major interceptor sewer systems in the City (see Figure 2-1). The Main, Northeast and Northwest Interceptor systems are tributary to the NEWPCC, while the SEWPCC and WEWPCC each have one tributary Interceptor system.

2.3.1 Main/NE/NW

The Main, Northwest and Northeast Interceptors are tributary to the NEWPCC and are shown on Figure 2-2. The Main Interceptor serves the older part of the City and receives flows from combined sewer districts only. In contrast, the Northeast and Northwest Interceptors only convey flows from separate sewer districts.

The Main Interceptor conveys the diverted flows from 34 of the 42 combined sewer districts (approximately 9,200 of the 10,500 ha.) in the City. The main branch of the Interceptor, from Broadway to the NEWPCC, was constructed during the 1930s and ranges in size from 1950 mm to 2250 mm. From Broadway, the Interceptor system branches to the south and to the west.

The south branch ranges in size from 900 mm to 1100 mm and starts on Main Street at the north side of the Red River. The Interceptor flows to the south side of the Assiniboine River, then crosses under the river through a 4 barrel, 300 mm diameter siphon. The Interceptor


NORTHWEST INTERCEPTOR
NORTHEAST INTERCEPTOR
MAIN INTERCEPTOR
NEWPCC
SUTHERLAND SECONDARY
SOUTH LEG
WEST LEG
N
1650
1350
1500
2250
1950
1800
1950
2250
1650
1800
1350
2
20A
20B
52
56
55
57
19
23
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32
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47
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62
LEGEND
COMBINED SEWER DISTRICT BOUNDARY
INTERCEPTORS
SECONDARY INTERCEPTOR
TO WEWPCC OR SEWPCC

FIG 2-2

sewer then runs from the siphon outlet on the north side of the river, along Main Street, to Broadway.

The west branch starts at Ferry Road and Portage Avenue. It runs along Portage Avenue, Raglan Road, Wolseley Avenue, Furby Street and Broadway. The west branch ranges in size from 750 mm to 1650 mm, and includes a 600 mm diameter siphon under Omands Creek. The west branch also has a section of 1200 m parallel relief sewer on Wolseley Avenue, between Aubrey Street and Ethelbert Street.

There are two major overflows (relief points) on the Main Interceptor. They are at St. John's Avenue and at Jefferson Avenue. The overflows are 1500 mm connections from the Interceptor to the combined trunk sewers. The connections are gated (flap gates) to prevent flows from the trunk sewers from entering the Interceptor. Therefore, for an overflow to occur, water levels must be higher in the Interceptor than in the combined system. A schematic of the St. John's overflow is shown on Figure 2-3.

The Main Interceptor system was designed to have a minimum flowing full capacity equal to 2.75 times ADWF (MacLaren, 1961). The hydraulic capacity of the Main Interceptor is rated at 6.14 cubic metres per second (cms) or 531 million litres per day (ML/d) (Wardrop/MacLaren, 1981). The flow diversions from the combined sewer systems were also designed based on intercepting 2.75 times the projected peak dry weather flow from the district (WWDD, 1965). In addition, the secondary sewers from the diversions to the interceptor were designed based on conveying 2.75 times DWF with the Interceptor sewer flowing full.

The Northeast and Northwest Interceptor sewer systems are also shown on Figure 2-2. The Northeast Interceptor ranges in size from 1350 mm to 1950 mm and conveys sanitary sewage from the North Kildonan and Transcona areas. The Northwest Interceptor ranges in size from 600 mm to 1650 mm and services the Brooklands and Maples areas. The individual districts tributary to these Interceptors are shown on Figure 2-1.

The Northeast Interceptor crosses the Red River adjacent to Whellams Lane. The crossing comprises a two-barrel inverted siphon, consisting of a 500 mm and a 800 mm pipe. The

ST. JOHN'S DISTRICT INTERCEPTOR OVERFLOW

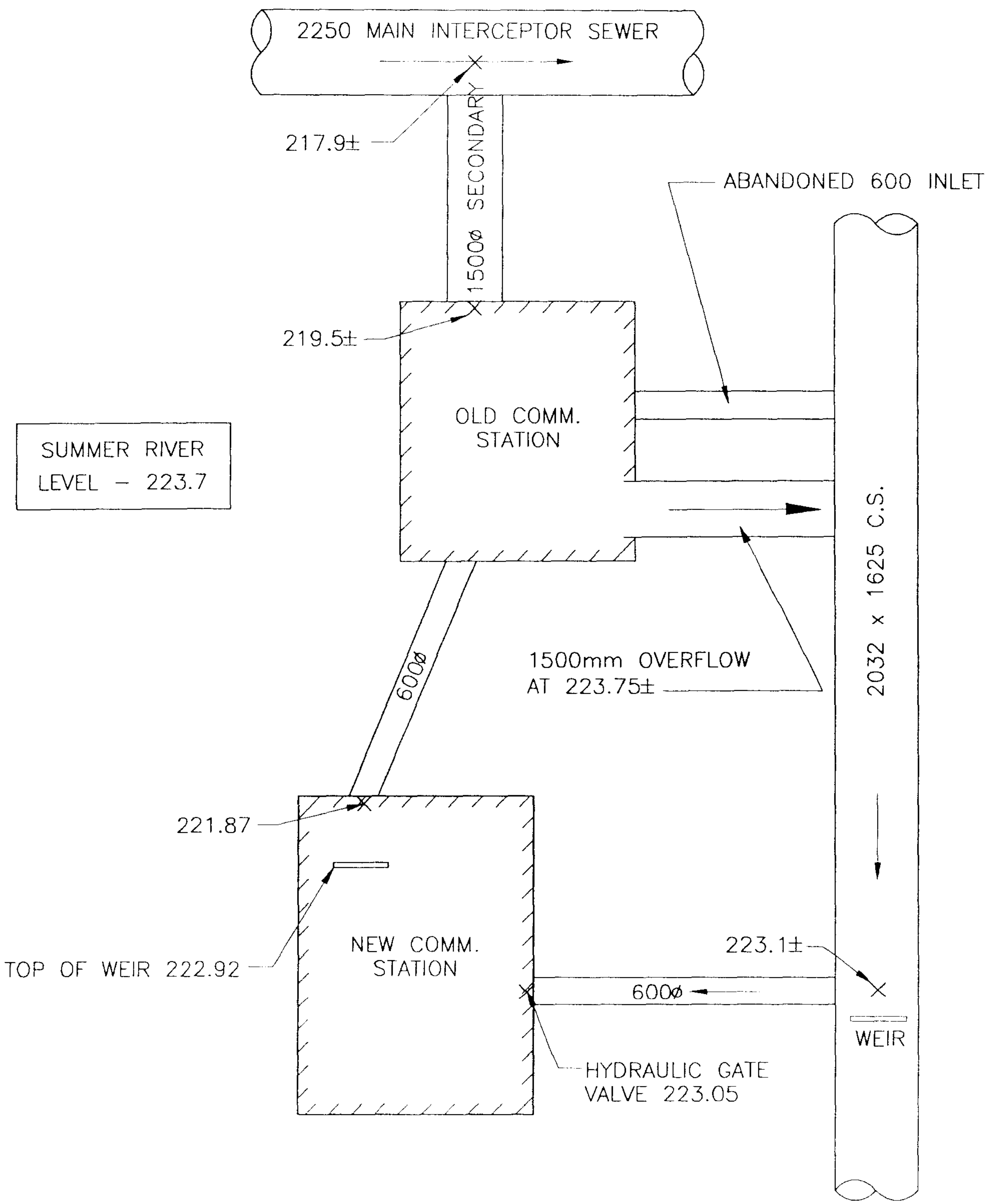


FIG. 2-3

interceptor continues east (at 1800 mm) to Main Street where it intercepts the North Main branch Interceptor (1350 mm). The Northeast Interceptor then runs south on Main Street, connects with the 1500 mm diameter Northwest Interceptor, where it increases to 1950 mm. The Interceptor then discharges to the 2250 mm Main Interceptor, immediately upstream of the NEWPCC.

The Northeast Interceptor has a 1200 mm overflow to the river. The overflow is part of the structure for the river undercrossing siphon inlet. The overflow discharges directly to the Red River. A flap gate in the structure prevents river water from entering the interceptor system. A 750 mm diameter overflow was also constructed on the Northwest Interceptor at Leila Avenue and Inkster Boulevard. The overflow consisted of a cross connection between the 1350 mm Interceptor and the 2750 mm LDS. Plugs have been installed at both ends of the cross connection.

As noted previously, the Northeast and Northwest Interceptors discharge to a common 1950 mm diameter sewer. The rated capacities of the Northeast and Northwest Interceptors are 3.89 cms (336 ML/d) and 4.51 cms (390 ML/d), respectively, while the capacity of the common 1950 mm sewer is 6.83 cms (590 ML/d) (Wardrop/MacLaren, 1981). The discrepancy between the accumulated capacities of the Northeast and Northwest Interceptors and the capacity of the common sewer (726 ML/d vs. 590 ML/d) reflects the in-system storage designed into the Northeast and Northwest Interceptors. The capacity of the common sewer could be increased to match the Interceptors by nominally surcharging the pipe. Conversely, evidence that the Interceptors have been under surcharge was discovered during the field investigations (Wardrop/Tetres, 1995, Draft) possibly indicating that the in-system storage is, at times, being utilized.

The Main Interceptor can deliver 6.14 cms of combined WWF (rated capacity) and, at the same time, the Northeast and Northwest Interceptors have flow capacities of 3.89 and 4.51 cms, respectively, of separate sanitary flow (considering entry of significant extraneous flow). Considering the capacity of the common sewer that the Northeast and Northwest Interceptors drain to (6.83 cms), this results in a total flow (rated capacity) of 12.97 cms (1121 ML/d) to the NEWPCC.

The foregoing illustrates the complex hydraulic interaction between the Main, Northeast and Northwest Interceptor systems and the NEWPCC. This interaction was studied in greater detail through the use of a detailed hydraulic model. The results of this analysis are contained in Section 2.5 of this document.

2.3.2 SE/WE

The South End (SE) and West End (WE) Interceptor sewer systems are also shown on Figure 2-1. These Interceptors convey flows from primarily separate sewer districts. However, the SE and WE Interceptors also receive diverted combined sewer flows from four and three districts, respectively.

The SE Interceptor conveys separate wastewater flows from the Fort Garry, St. Norbert, St. Vital and St. Boniface areas (see Figure 2-1 for tributary separate sewer districts), as well as the following four combined sewer districts:

- Cockburn/Calrossie;
- Baltimore;
- Mager; and
- Metcalfe.

The Interceptor system ranges in size from 1350 mm to 1950 mm.

The D'Arcy pumping station collects the wastewater from the Fort Garry Interceptor and pumps it across the Red River. The major emergency overflow points of the SE Interceptor system are at Killarney Avenue and the D'Arcy pumping station for the Fort Garry systems (west of the Red River) and at the St. Mary's outfall for the St. Vital Interceptor (east of the Red River). The system is designed to deliver about 3 x DWF to SEWPCC (MacLaren 1986), i.e. the SEWPCC is designed to treat this PWWF.

The WE Interceptor conveys separate wastewater flows from the St. James and Charleswood areas (see Figure 2-1 for tributary separate sewer districts) and the following three combined sewer districts:

- Woodhaven;
- Moorgate; and
- Strathmillan.

The Interceptor ranges in size from 375 mm to 1350 mm. The Interceptor system runs to the Perimeter Road pumping station on Wilkes Avenue, east of the Perimeter Highway. From there the wastewater is pumped through a 750 mm forcemain to the WEWPCC.

Wastewater generated north of the Assiniboine River is transported in the St. James Interceptor, which follows the river and flows from east to west along Assiniboine Avenue and Assiniboine Crescent. A major overflow provision for the St. James Interceptor is located at the Parkdale Outfall, just upstream of the Interceptor crossing of the Assiniboine River. After crossing the river, the Interceptor continues south to the Community Row pumping station.

Wastewater generated south of the Assiniboine River is transported in the Grant Avenue Trunk/Charleswood Westdale Interceptor. The Grant Avenue/Charleswood Interceptor flows west on Grant Avenue to the Community Row pumping station. There is provision for overflow at this point. The combined flow (from the St. James Interceptor and the Grant Avenue/Charleswood Interceptor) is then lifted into the Charleswood interceptor and conveyed to the Perimeter Road pumping station. The Perimeter Road pumping station pumps all the wastewater flow to the WEWPCC. The WEWPCC is designed for 3.5 times DWF (MacLaren 1986).

2.4 MAIN INTERCEPTOR

The main interceptor is of primary importance to the study due to the number of tributary combined sewer districts (34) and the size of the tributary area (9,200 ha.). A detailed understanding of the Main Interceptor hydraulics was not available, as discussed in Phase 1, due to the following factors:

- the potentially large variation in interception rates in some districts, especially those districts without pumping stations and regulator valves;
- the manner in which the overflows operate, and the potential for overflows at other points in the system; and
- the complexity of the overall system, due to the contributions from the Northeast and Northwest Interceptors.

Of these factors, by far the most complex is the performance of the various district diversions. While these installations were originally similar in nature, modifications had been carried out over the years. Therefore, priority field investigations were initiated in Phase 1 to develop a more complete understanding of these diversions. The investigations enabled detailed hydraulic modelling and a description of the existing WWF operation of the interceptor system.

2.4.1 Field Inspections

The detailed field investigations of the district diversion facilities and the spot inspections of the Main, and Northwest and Northeast Interceptors were conducted during the period of June to August, 1994. These investigations were carried out with the assistance of the City of Winnipeg staff, using equipment for confined entry procedures. The interception points were reviewed for:

- weir type and height;
- sediment depth in the combined trunk sewer;
- depth of flow;
- comminutors;
- regulator valves, and if present, operating protocol;
- gate type and size;
- dry weather overflows;
- any anomaly that could impact system hydraulics; and
- structural condition.

This information, including schematics of the hydraulic flow path, conditions, components and photographic diary of the inspection is included for each interception point in the draft Inspection Report (Wardrop/TetrES, May 1995).

The Main Interceptor system, due to the depth of the sewer (> 15 metres at the downstream end) and the hazardous entry conditions, has not been monitored for its structural condition. The City plans to conduct such an investigation. For this study, due to the importance of the Main Interceptor as the backbone of the City's central collector system, it was determined that visual spot inspections would be valuable in providing a preliminary estimate of the structural condition. These inspections were carried out and included a review of the conditions in the Northeast, Northwest and Sutherland Secondary Sewers.

In general, the structural condition of the Main Interceptor, in spite of its age (over 60 years) is surprisingly good, based on the limited inspection. One section, near Sutherland, exhibited signs of localized sulphide attack and needs further inspection. The NE and NW Interceptors are newer installations and, as expected, appeared to be in sound condition. Excerpts from the draft inspection report, containing information on the hydraulic components of the district diversion structures and on the structural condition of the interceptor sewers, is contained in Appendix A.

2.4.2 Interception Points

The combined sewer interception points are generally of the same basic configuration. A typical installation is shown on Figure 2-4 and is described below.

A weir in the combined trunk sewer is used to divert the flows to an offtake pipe. The weirs are typically 200 mm to 600 mm high, and of concrete construction. The offtake pipe flows from the trunk to a comminutor chamber (a chamber designed to accommodate a mechanical rotating screening/grinding device to cut suspended material in the sewage). The piping is generally less than 15 metres in length, and 450 to 525 mm diameter, with the invert at or below the invert of the trunk sewer. A 500 mm diameter (usually) regulator valve is mounted on the outlet of the offtake piping, on the inside of the chamber. The regulator gate controlled

TYPICAL DIVERSION CONFIGURATION

(WITH PUMPING)

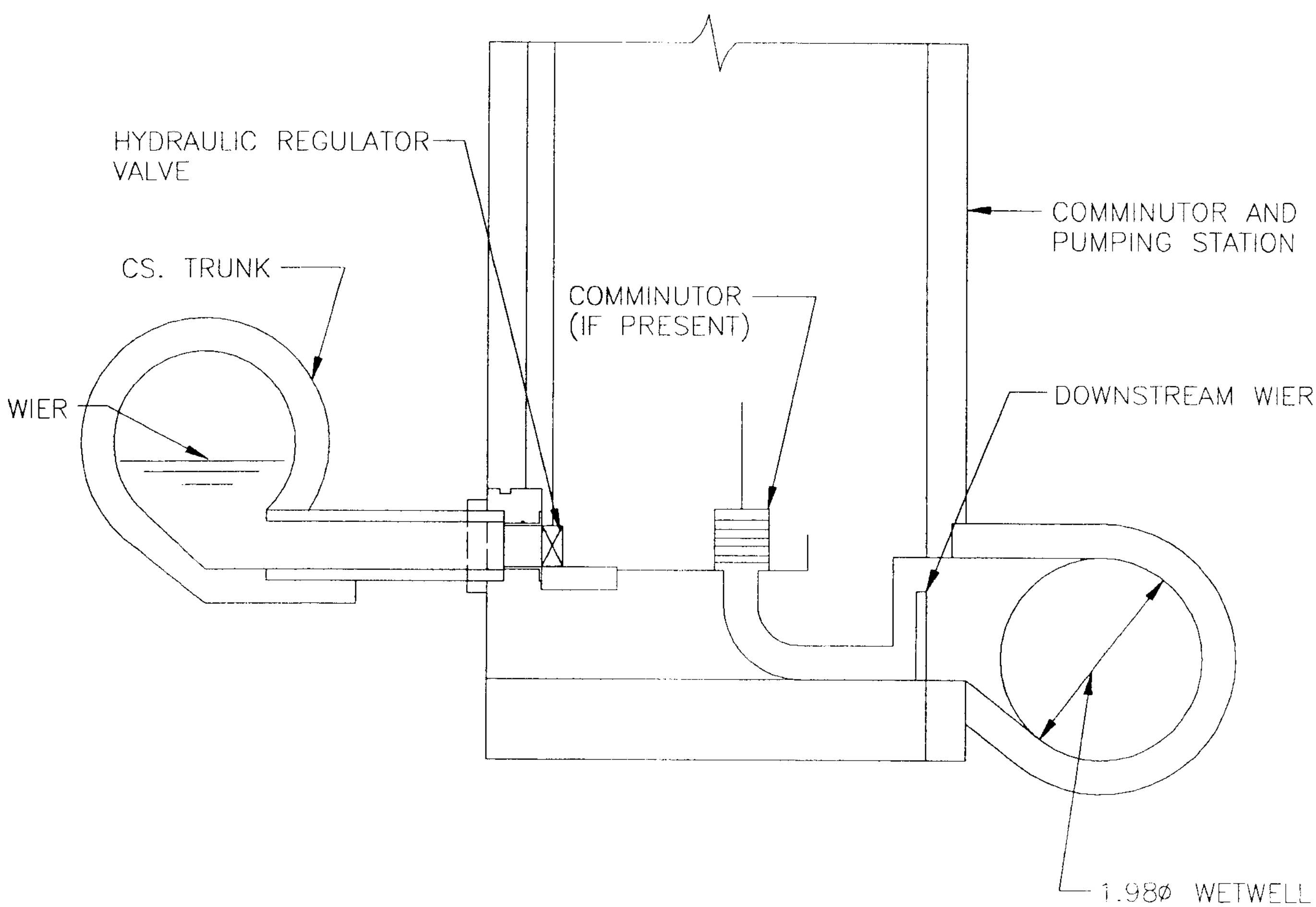


FIG 2-4

CSO-CH15.dwg

the water level inside the chamber and was set to close in response to rising levels, thus regulating flow into the chamber and preventing the comminutor from flooding. These gates were originally installed in every station. Over the years a number of the gates have been removed, or locked open by City staff. The current status of the remaining regulator gates is shown on Figure 2-5.

The wastewater flows through the regulator gate into the comminutor channel. The channel is typically about 750 mm wide and 2.5 to 3 metres long, and leads to the comminutor. The comminutors were initially installed to provide continuous cutting of coarse sewage matter to prevent clogging of the local pumping facilities and to mitigate the impact of these solids on the treatment plant. With the installation of mechanical screens at the NEWPCC, the need for the comminutors in the gravity flow districts became redundant. The majority of the comminutors have since been removed. A few of the installations remain in service in the pumped districts to prevent clogging of the local pumps. The location of the districts with functioning comminutors is shown on Figure 2-6.

After passing through the comminutor, the wastewater flows through the downstream channel and over a weir. The weir was required to maintain a downstream head on the comminutor, thus providing a large surface area to the rotating screens. However, these weirs have also been removed at a number of locations coinciding with the comminutor removal. After passing over the weir, the wastewater flows to either a wetwell (pumped districts) or directly to a secondary sewer (gravity districts). The locations of the pumped and gravity districts are shown later on Figure 2-8.

The inspection of the interception points revealed that the existing hydraulic conditions at these points is significantly different from the original design intent. The removal of the comminutors and associated weir and the deactivation of the regulating gates means that the restrictions to flow are greatly reduced. This typically means that more than the intended 2.75 times DWF will be intercepted during WWF. These complications are discussed later in this document.

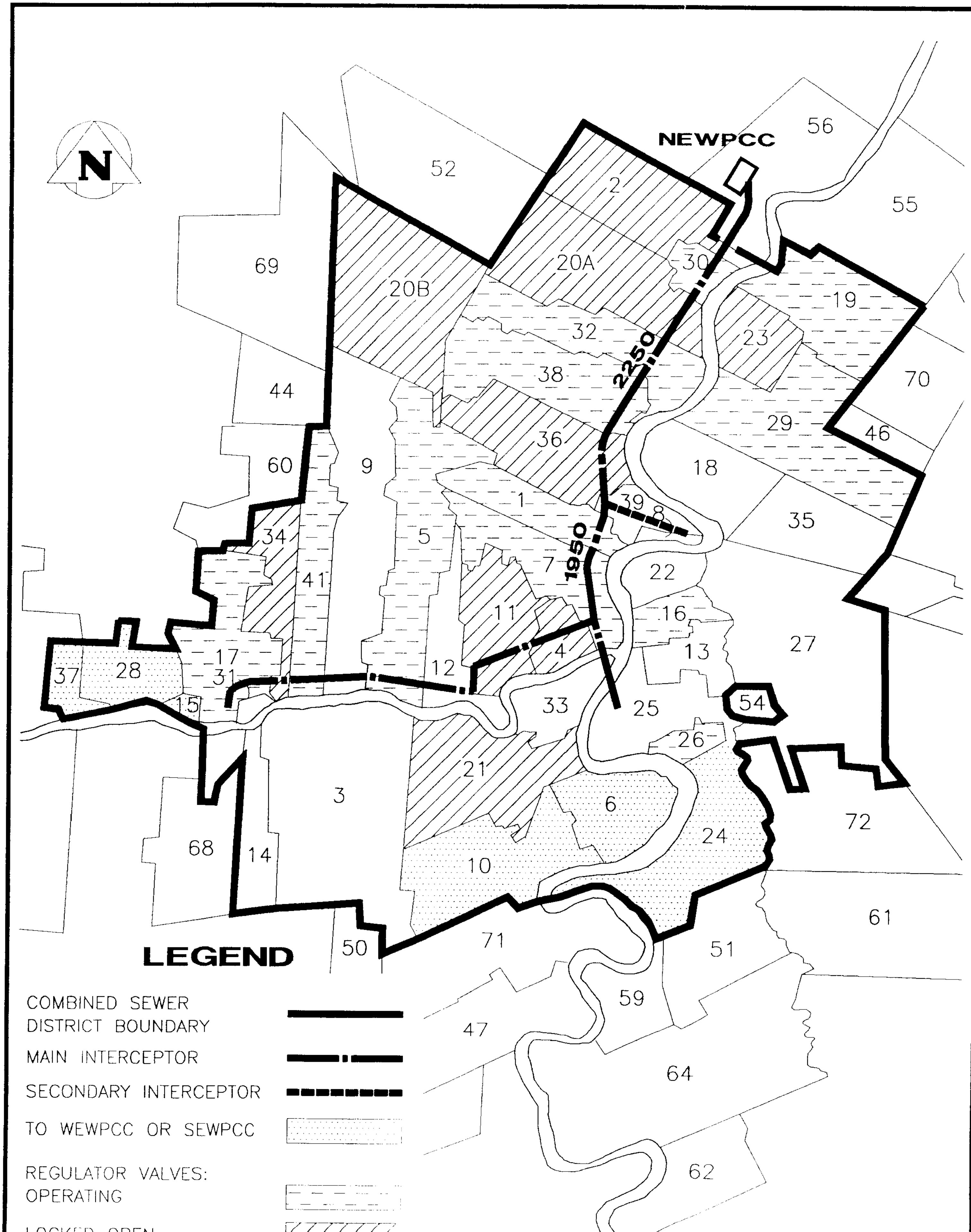


FIG 2-5

CSO-OH17.dwg

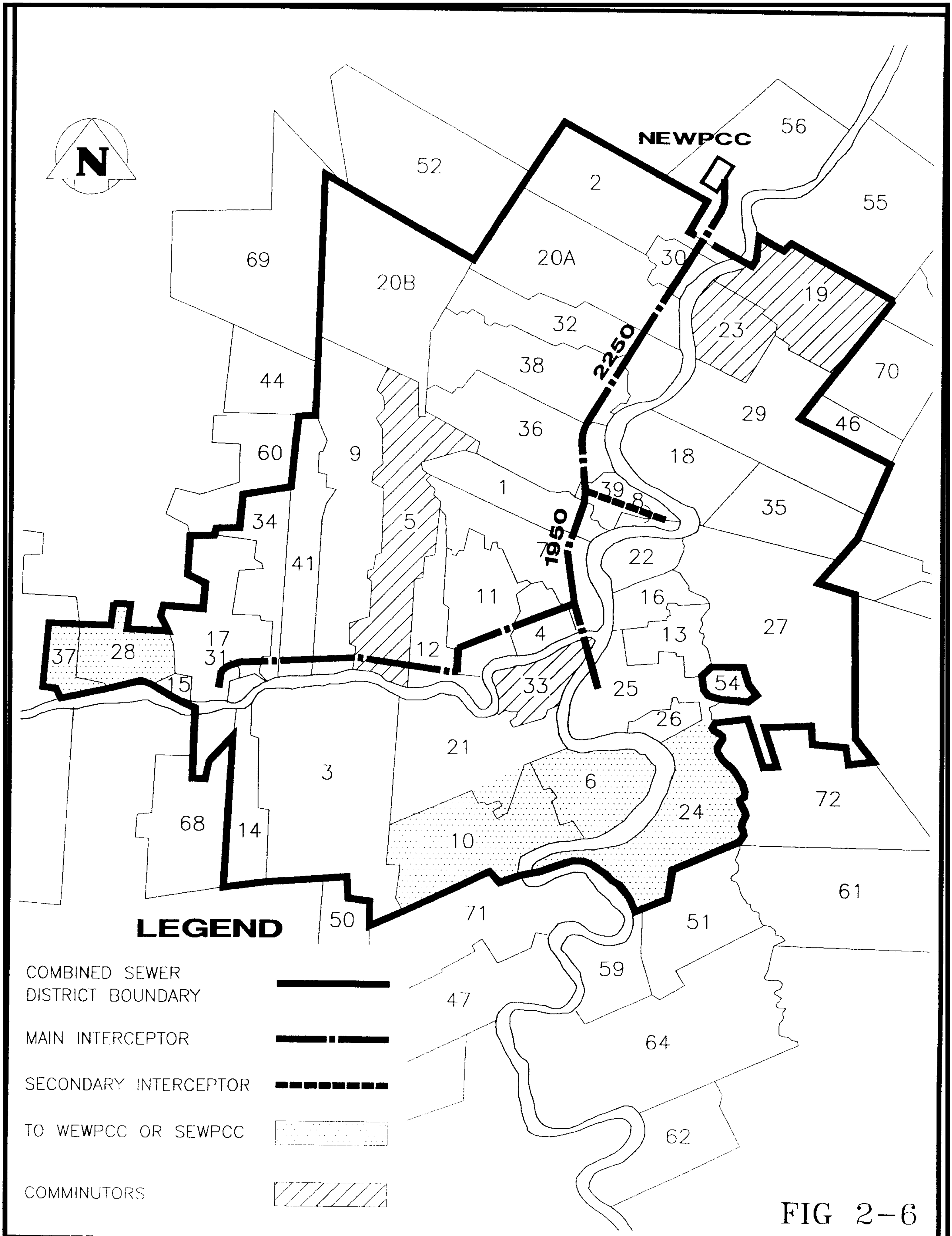


FIG 2-6

CSO-CH18.dwg

Dry Weather Overflows (DWOs)

The elimination of dry weather overflows (DWOs) at combined sewer interception points is a priority in any CSO control program.

The City realized the importance of eliminating all dry weather overflows to the rivers, and therefore included reporting of such DWOs discovered in the field inspections, and the implementation of necessary remedial measures as part of the Terms of Reference of this study. These overflows occur when the dry weather flows in the trunk sewer exceed the diversion capacity of the interception point. This could be a result of the weir being set too low, a lack of capacity in the secondary sewer or pumping facility, or by illicit discharge into the sewer. In any event, the dry weather flow overtops the weir and is discharged to the river.

The weir heights and downstream conveyance capacity (gravity or pumping) were initially designed based on intercepting 2.75 times average dry weather flow (ADWF). The ADWF was determined from the results of the sewer gauging program where winter flows in the trunk sewer were measured by the use of temporary weirs for short intervals. In some districts, where development is nearly complete, the gauging data used for setting the weir height compares well with today's estimates of flow. For example, the 1964 design notes for the Polson District comminutor station indicate an ADWF of 31 litres per second. The current ADWF estimate, based on wastewater flow being 1.35 times water consumption, is 32 litres per second. In this example, which can probably be extended to many of the older districts, there has been a negligible increase in ADWF since the installation of the diversions.

However, dry weather overflows were noted during the field inspections at Tylehurst and Cockburn Districts (see Figure 2-7, a photograph of DWO at Tylehurst). Both of these districts have been identified by the City operations staff as having occasional overflows in the summer months, which cannot be attributed to WWF. The cause of the overflows is apparently an increase in DWF over the summer months. One explanation for this phenomenon is that a significant amount of groundwater, used for cooling purposes, is being discharged to the sewers. This explanation is supported by the number of apartment buildings in each district that may utilize groundwater for cooling. In this regard, summer sewer

FIG 2–7

gauging should be initiated to quantify the summer DWF in order to develop a rationale for the implementation of remedial measures.

Dry weather overflows are also reportedly occurring from the storm relief sewer outfall on Donald Street in the Assiniboine District and the combined trunk sewer in the Aubrey District. The apparent cause of the Donald Street overflow is backup of flow from the combined system into the relief piping. The relief piping is designed to augment the capacity of the combined sewer system under WWF. However, due to lack of gradient in the combined sewers, coupled with sedimentation, some DWF is not reaching the diversion, but rather overflowing into the relief system. The occurrence of DWOs in the Aubrey District were reported to the Study Team by members of the public during the initial open house at the Forks. The cause of the overflows is not known. Means of correcting these situations should be investigated.

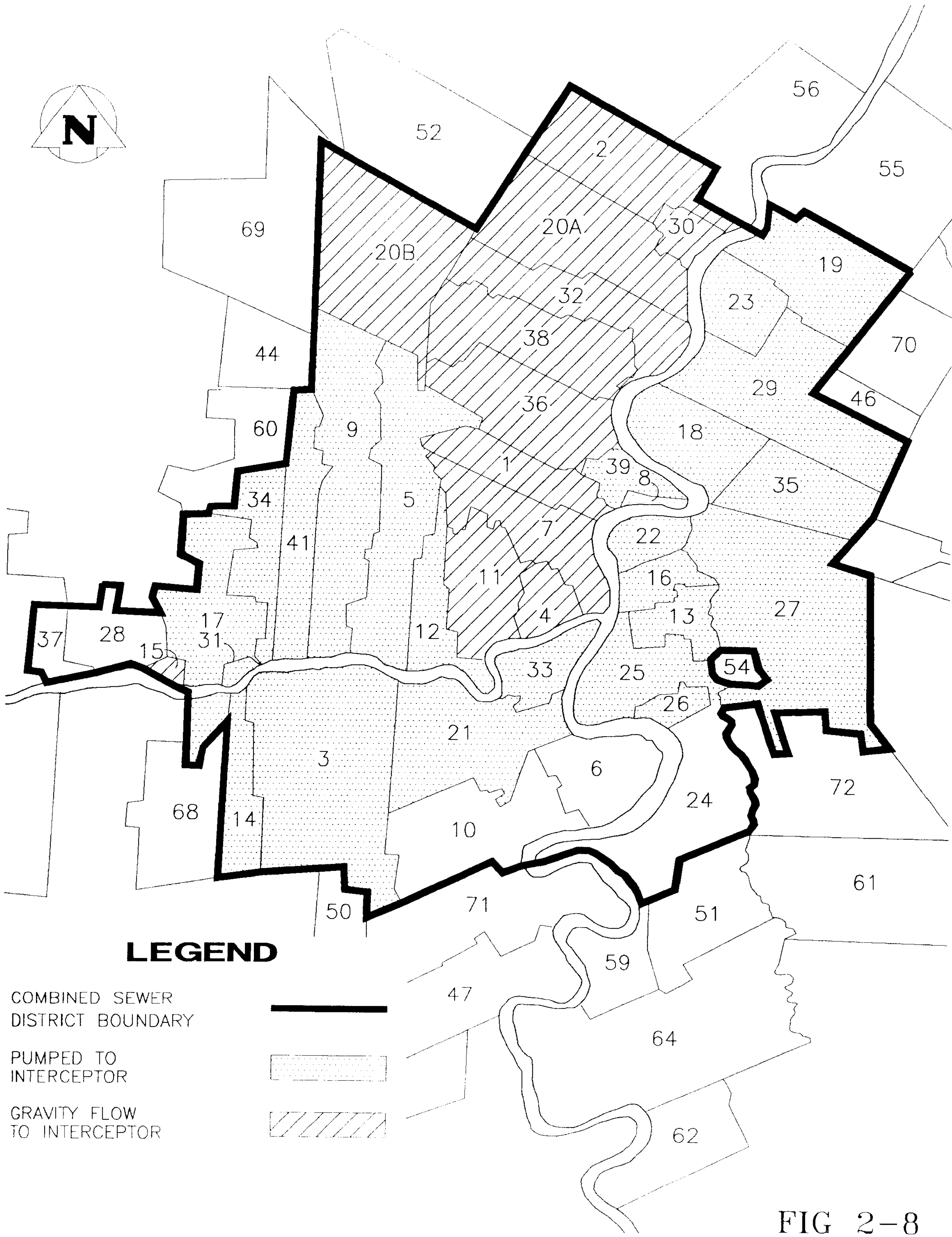
Combined Sewer System Interactions

The interaction between the combined sewer system, the WWF diversion facilities, pumping stations, interceptor sewers and the WPCCs is quite complex. Information on each component in the system was gathered, reviewed and analyzed in detail to gain a better understanding of the physical processes that occur under WWF. This information included:

- A review of the existing WWF diversion drawings in conjunction with the information on routing, sizes and configurations obtained from the field inspections. This information allowed the analysis of the configuration of the connections between the combined trunk sewers and the pumping stations or secondary sewers (depending on the District - see Figure 2-8).

- Pump curves for the wastewater pumping stations were obtained and analyzed to determine the installed peak pump capacities of the stations. This results of this analysis were reviewed with WWDD staff and are shown on Table 2-1.

- Drawings of the secondary sewers from the diversions to the interceptor were reviewed.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20A
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19
32
23
44
38
70
60
9
36
29
46
18
1
39
8
35
34
5
41
7
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11
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31
12
4
15
33
25
54
26
21
3
6
72
24
68
14
10
61
50
71
51
LEGEND
59
47
COMBINED SEWER
DISTRICT BOUNDARY
64
PUMPED TO
INTERCEPTOR
GRAVITY FLOW
TO INTERCEPTOR
62

FIG 2-8

Table 2-1
CSO Pumping Station Capacities

District Pumping Station	Pumping Capacity L/sec	USgpm	Comments
Ash	301	4,800	also pumps Doncaster
Aubrey	327	5,200	
Baltimore	201	3,200	also pumps Cockburn
Boyle	30	480	to be abandonded; flow diverted to Syndicate
Chataway (Tuxedo)	36	575	pumps to Doncaster
Clifton	236	3,750	
Cockburn	75	1,190	also pumps Calrossie
Cornish	107	1,700	
Despins	132	2,100	
Dumoulin	151	2,400	also pumps Laverendrye
Ferry Road	215	3,420	
Hart	101	1,610	
Hawthorne	220	3,500	
Jessie	176	2,800	
Linden	113	1,800	
Mager Drive	377	6,000	also pumps Metcalfe and Baltimore
Marion	220	3,500	
Metcalfe	44	700	
Montcalm	1,130	18,000	pumps Roland and Mission Districts
Moorgate (Conway)	85	1,350	
River	94	1,500	
Riverbend	295	4,700	
Syndicate	69	1,100	
Tylehurst	176	2,800	
Woodhaven	50	800	

- The gate chambers on the combined trunk sewers were analyzed in the context of the WWF perspective. When river levels are higher than the weir elevations, the river level controls when an overflow will occur.

- Pumping operations at the NEWPCC were reviewed in detail. The pumping protocol at the plant sets the hydraulic conditions at the downstream end of the interceptor. The normal operating condition during the winter season is to keep operating levels in the surge tank above the obvert of the outlet of the incoming interceptor. This is done to prevent foul air from entering the plant. During the open water season, generally May to October, the pumps operate so that the level in the surge well is lowered to take advantage of the available storage in the interceptor for WWF. The pumping protocol and pumping capacity at the different set-points is discussed in detail in Section 2.4.3.

The hydraulic conditions at each of the interception points were then modelled as part of the hydraulic analysis.

2.4.3 Hydraulic Modelling

In Phase 1, a preliminary analysis was carried out to determine the capacity of the main interceptor system (see Phase 1, TM #1). While this analysis provided an initial estimate on the interceptor capacity, it was apparent that more sophisticated models would be required to analyze the system under WWF conditions. This would include modelling not only the interceptor, but ultimately the combined trunks, outfalls, overflows, diversions and the NEWPCC pumping conditions. In assessing the WWF conditions in the system, it was determined that the model would have to be capable of handling the following hydraulic conditions:

- open channel and pressure (surcharge) flow;
- backwater conditions;
- flow transfers from weirs, orifices and pumps;
- multi-unit pumping at district stations and at the NEWPCC;
- multiple flow inputs;

- multiple river stage boundary conditions; and
- circular and egg-shape conduits.

The foregoing hydraulic conditions were the basic parameters used in the process of screening and selecting a model for the detailed analysis of the interceptor system including the interaction with the NEWPCC.

Model Selection and Development

The number of available models which would be able to analyze the interceptor sewer system was limited due to the requirement for modelling non-uniform, turbulent flow subject to backwater and surcharge conditions. The potential models had to be capable of performing the dynamic routing of sanitary and wet weather flows from the combined sewer system, through the diversions and to the outfalls, and through the interceptor system to the WPCC, as well as linking to the regional runoff and receiving stream models.

A working session was held on April 25 and 26, 1994 to evaluate the background information and to develop a technical framework for system assessment, including the selection of computer-based mathematical models. The workshop was attended by specialist consultants and local key study team members. As a result of the working session, XP-SWMM was selected as the detailed model for Phase 2 analysis of the interceptor system. A complete report on the working session, including the discussions, findings and recommendations, is included in Phase 1, TM #7, "Technical Framework."

The next step, was to develop the model of the Main Interceptor system. Initially the model comprised the Main Interceptor, secondary sewers, and the NEWPCC surge well and outfall, as well as the downstream portions of the Northeast and Northwest Interceptors. The process stream itself, at the NEWPCC, was not included in the model. All wastewater pumped at the NEWPCC receives treatment as discussed in Section 4.0, and it is not necessary for hydraulic analysis of the interceptors to include the treatment process. The hydraulics of the plant process are independent of that of the interceptors. Therefore, for hydraulic analysis, it was assumed that the wastewater pumped from the surge well went directly to the plant outfall.

The information (pipe sizes, lengths, and invert elevations) necessary for building the model was obtained from the drawing archive at the Water and Waste Department. The initial model consisted of:

- 87 conduits, ranging in size from 750 mm to 2250 mm in diameter;
- 76 junctions, (manholes or points receiving inflow);
- 1 storage junction (Surge well at the NEWPCC);
- 6 pumps (MP1 to MP6 at the NEWPCC);
- 1 outfall (NEWPCC); and
- 1 overflow (on the Northeast Interceptor).

A schematic of the Main Interceptor model is shown on Figure 2-9. The model was checked for errors and continuity and deemed accurate for the preliminary analysis of the Main Interceptor hydraulics.

Action Plan

An action plan was prepared for the Interceptor modelling on the project. The plan covered the steps involved from the initial preparation of the model through to the assessment of selected control options and pilot testing. The flow chart of the plan is shown on Figure 2-10.

Interceptor Hydraulics

The Main, Northeast and Northwest Interceptors were analyzed to determine the hydraulic capacity and performance of the system. This analysis was designed to provide a preliminary estimate of the conveyance capacity and to show, with particular emphasis on the Main Interceptor, the weak links in the system. The hydraulic capacity was determined by routing progressively larger flows, in multiples of DWF, through the model.

Complete records for gauged wastewater flow at the districts were not available. However, from a database of water meter readings, water consumption can be determined for any of the districts. For preliminary analysis, the DWF values were established based on a comparison between the pumping records at the NEWPCC versus monthly water usage

CSD-275.dwg

MAIN INTERCEPTOR SEWER HYDRAULIC MODEL SCHEMATIC

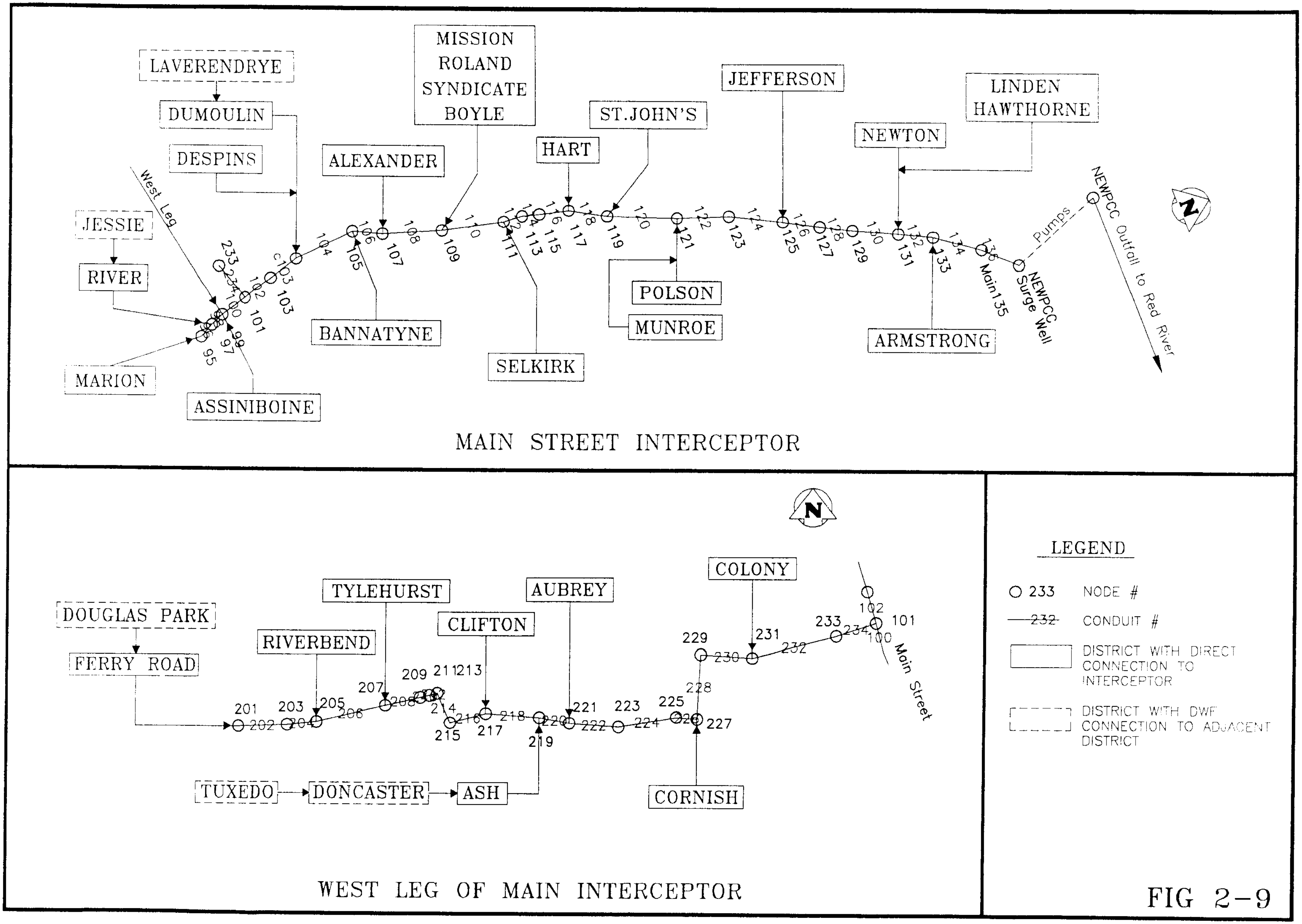


FIG 2-9

ACTION PLAN FOR INTERCEPTOR MODELLING

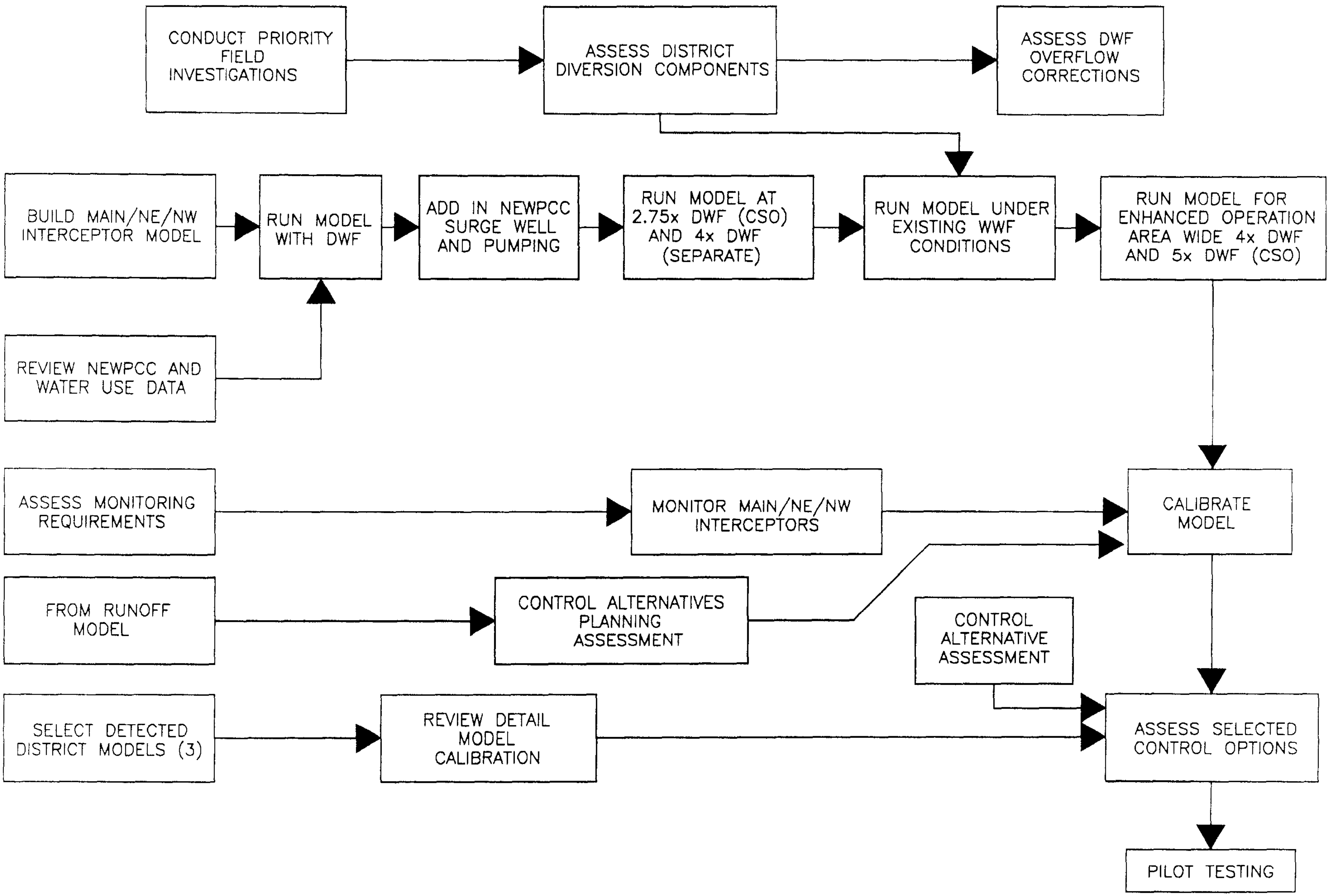


FIG 2-10

records for the NEWPCC service area. A review of the records for the period from January 1992 to February 1994 indicated that, for the dry weather season (i.e., November to February), wastewater flows exceeded water use by a factor of 1.35. The additional flows (ie. those in excess of the water consumption) were attributed to infiltration and extraneous flows. The 1.35 factor was then applied to the water consumption records for the individual districts (CSO and separate) for the month of January, 1993. The ADWF resulting from this calculation for the individual drainage districts contributing to the NEWPCC is shown on Table 2-2 as well as the estimated total DWF.

The shortcomings of this method of determining ADWF in this manner are evident. This is the case when any information from a large regional database is interpolated down to the individual units (i.e., in the present case, the district by district record of PDWF may be higher or lower than the overall average). However, while the resultant data may not be representative of any given individual drainage district, the overall impact on the major components, ie. the Interceptors and the NEWPCC, is representative. In this case, the water consumption data was reasoned to be adequate for the Phase II planning level hydraulic analysis of the Main, Northeast and Northwest Interceptor systems and the plant.

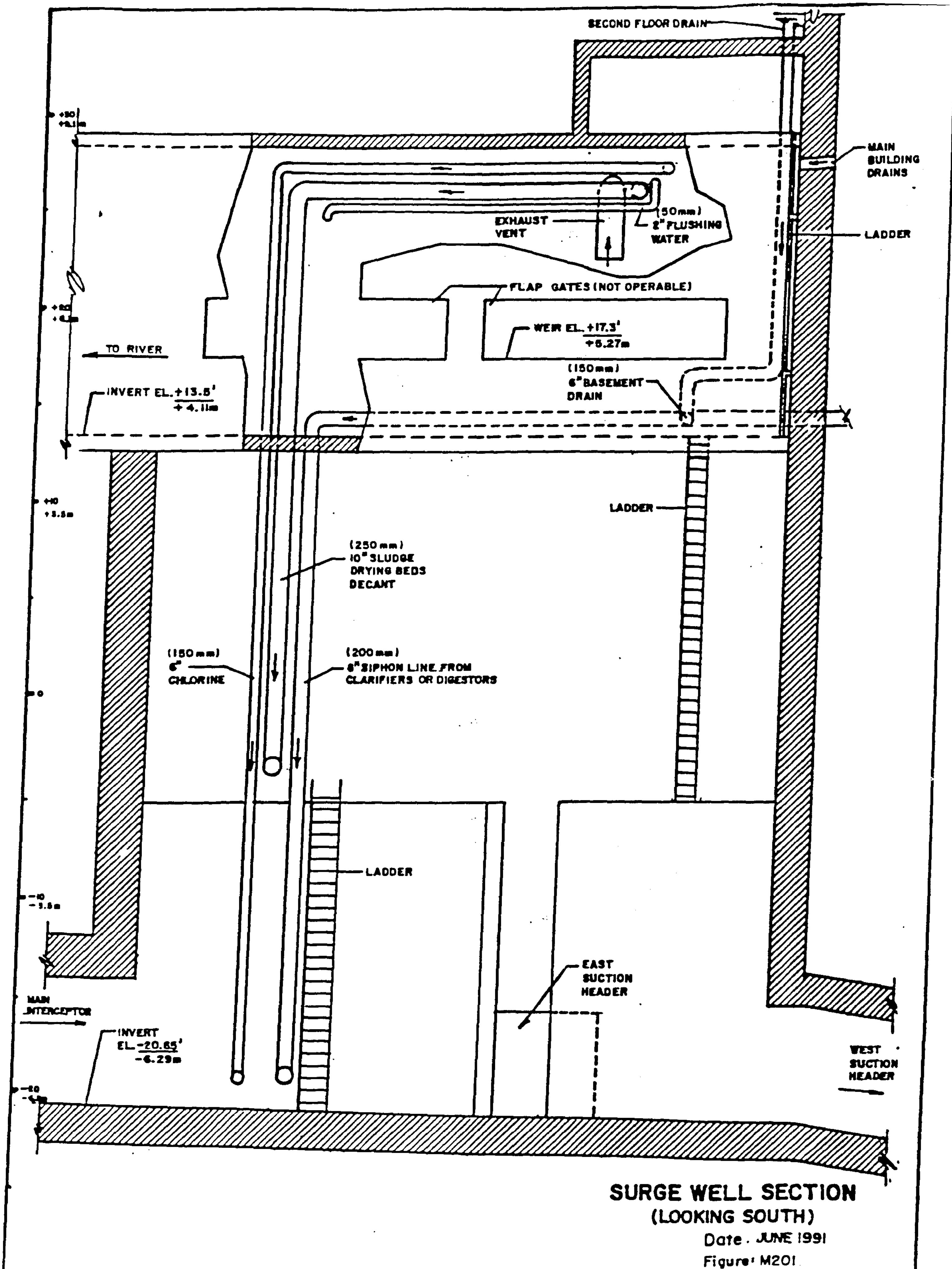
Flows from each district were input to the interceptor model to correspond with the existing secondary sewer or forcemain connections. This was done to ensure that the cumulative flows for each reach of the interceptor sewer was representative. The interceptor model was initiated by routing ADWF through the system until dry weather equilibrium was reached. The model was then stopped, and saved for use as the initial condition for future runs with WWF, by utilizing the Extran "hot start" utility.

<u>NEWPCC Pumping</u>

The flows from the Main Interceptor, including the Northeast and Northwest Interceptors discharge to the surge well at the NEWPCC. A cross section of the surge well is shown on Figure 2-11). The Main Interceptor connects to the surge well at elevation 215.48 (-6.29m City datum). Operating levels in the surge well, and thus the interceptor, can be changed, which can be important in affecting the available hydraulic head (and flow capacity) in the interceptors.

Table 2-2
Average DWF Based on Water Use

District	acres	Cubic Metres per month Jan 93 Use	Cubic Metres per day Jan 93 Use	Average Use Litres/sec. Jan 93	Average DWF Litres/sec. Ave. Use*1.35
Alexander	395	70281.4	2267.1	26.2	35.4
Armstrong	360	39925.2	1287.9	14.9	20.1
Ash	1815	162115.9	5229.5	60.5	81.7
Assiniboine	218	166361.5	5366.5	62.1	83.9
Aubrey	1091	140481.5	4531.7	52.4	70.8
Bannatyne	650	304537.5	9823.8	113.7	153.5
Boyle	66	27612.3	890.7	10.3	13.9
Clifton	1221	153261.9	4943.9	57.2	77.2
Colony	568	266590.8	8599.7	99.5	134.4
Cornish	354	70711.5	2281.0	26.4	35.6
Despins	291	64222	2071.7	24.0	32.4
Doncaster	384	49296.4	1590.2	18.4	24.8
Douglas Park	62	2505.1	80.8	0.9	1.3
Dumoulin	206	25280.6	815.5	9.4	12.7
Ferry Road	721	116255.4	3750.2	43.4	58.6
Hart	560	77796.5	2509.6	29.0	39.2
Hawthorne	643	70695.2	2280.5	26.4	35.6
Jefferson E	1075	93124.6	3004.0	34.8	46.9
Jefferson W	1403	189785.1	6122.1	70.9	95.7
Jessie	987	130880.9	4222.0	48.9	66.0
La Verendrye	177	17718.2	571.6	6.6	8.9
Linden	394	33976.9	1096.0	12.7	17.1
Marion	571	63558.9	2050.3	23.7	32.0
Metcalfe	87	10416.7	336.0	3.9	5.3
Mission	1860	286493.7	9241.7	107.0	144.4
Munroe	1002	115499.3	3725.8	43.1	58.2
Munroe Annex	262	23652.9	763.0	8.8	11.9
Munroe S	194	14204.2	458.2	5.3	7.2
Newton	202	19787.4	638.3	7.4	10.0
Polson	648	63364.5	2044.0	23.7	31.9
River	311	139285.9	4493.1	52.0	70.2
Riverbend C	511	69747.5	2249.9	26.0	35.2
Riverbend S	331	34939	1127.1	13.0	17.6
Roland	515	51088	1648.0	19.1	25.8
Selkirk	805	133788	4315.7	50.0	67.4
St. Johns	876	167336.8	5398.0	62.5	84.3
Syndicate	188	19573.4	631.4	7.3	9.9
Tuxedo	130	8026.6	258.9	3.0	4.0
Tylehurst	534	43499.6	1403.2	16.2	21.9



SECOND FLOOR DRAIN
MAIN BUILDING DRAINS
LADDER
(50mm) 2" FLUSHING WATER
EXHAUST VENT
FLAP GATES (NOT OPERABLE)
WEIR EL. +17.3' +5.27m
TO RIVER
INVERT EL. +13.5' +4.11m
(150mm) 6" BASEMENT DRAIN
LADDER
(250mm) 10" SLUDGE DRYING BEDS DECANT
(150mm) 6" CHLORINE
(200mm) 8" SIPHON LINE FROM CLARIFIERS OR DIGESTORS
LADDER
EAST SUCTION HEADER
MAIN INTERCEPTOR
INVERT EL. -20.65' -6.29m
WEST SUCTION HEADER
+10 +3.5m
0
-10
SURGE WELL SECTION
(LOOKING SOUTH)
Date. JUNE 1991
Figure: M201

FIG 2-11

The operation on the pumping facilities and the surge well levels are of importance to the interceptor modelling in that they determine the end of pipe hydraulic condition. As flows fluctuate, so do the levels in the surge well that provide the backwater condition for the determination of the upstream hydraulic grade line. Therefore, the surge well, as well as the pumps and their operations, must be an integral part of the interceptor model.

From the surge well the flow is directed to two suction headers that lead to the six pump intakes. The west suction header is 2.29m high and 1.68m wide and is connected to the sewage pumps numbered MP2, MP3 and MP4. The east suction header is 1.52m high and 2.13m wide and is connected to pumps MP1, MP5 and MP6.

The level in the surge well is measured by an ultrasonic (Milltronics) level sensor. The levels are transmitted to the controller that governs the pump duty selection. The transmitter also contains a low level switch that will lock-out all the pumps should the level in the surge well fall below elevation 216.27 (-5.5m City datum).

The pumps are brought on and off line according to a duty selection. If DWF conditions exist, then three pumps are used. If WWF conditions exist, then all six pumps are designated a duty status. Upon a rising surge well level, the pumps come on line in ascending order and are taken off in reverse order as the levels in the surge well recede. Table 2-3 illustrates this process, which pumps are available for specific duty selection, and the range of pumped flows possible for each duty. The levels are not supplied since they are dependent upon specific set points.

The set points are used to establish specific surge well levels at which the duty pumps go on and off line. These points are input to the control system and the preprogrammed levels are referenced to determine duty pump selection. Under normal operating procedure, the elevations of the DWF and WWF setpoints are 217.80 (-3.66m) and 217.19 (-4.57m), respectively. The duty selection and surge well levels for these setpoints are shown on Table 2-4.

The surge well was modelled as a storage junction with a constant surface area of 85 square metres (actual size). Each of the six pumps were modelled independently using the dynamic

TABLE 2-3
RAW SEWAGE PUMPING CONTROL STRATEGIES

DRY WEATHER FLOW (DWF)			
SURGE WELL LEVEL	DUTIES RUNNING	FLOW - MIN TO MAX	
		ML/d	cms
Increasing Level			
-	1	77 to 188	0.89 - 2.18
*	1 & 2	186 to 376	2.15 - 4.35
**	1,2 & 3	263 to 564	3.04 - 6.53
Decreasing Level			
*	1 & 2	186 to 376	2.15 - 4.35
-	1	77 to 188	0.89 - 2.18

PUMP DUTIES (DWF):

1. P2 or P4
2. P1, P5L, P3 or P6
3. P2 or P4

WET WEATHER FLOW (WWF)			
SURGE WELL LEVEL	DUTIES RUNNING	FLOW - MIN TO MAX	
		ML/d	cms
Increasing Level			
-	1	77 to 188	0.89 - 2.18
*	1 & 2	186 to 376	2.15 - 4.35
**	1,2 & 3	283 to 564	3.04 - 6.53
***	1,2,3 & 4	456 to 680	5.28 - 7.87
****	1,2,3,4 & 5	646 to 868	7.48 - 10.05
*****	1,2,3,4,5 & 6	834 to 1056	9.65 - 12.22
Decreasing Level			
****	1,2,3,4 & 5	646 to 868	7.48 - 10.05
***	1,2,3 & 4	456 to 680	5.28 - 7.87
**	1,2,& 3	283 to 564	3.04 - 6.53
*	1 & 2	186 to 376	2.15 - 4.35
-	1	77 to 188	0.89 - 2.15

PUMP DUTIES (WWF):

1. P2 or P4
2. P1, P5L, P3 or P6
3. P2 or P4
4. P1, P3, P5H or P6
5. P1, P3, P5H or P6
6. P1, P3, P5H or P6

TABLE 2-4

SET POINTS FOR DRY WEATHER FLOW			
DWF SET POINTS	DUTY SELECTION	SURGE WELL LEVEL	
		City Datum	m
City datum: -12' (-3.66 m)	Start 2nd Duty	-9.0	219.02
	Start 3rd Duty	-6.0	219.94
Geodetic datum: 218.11	Stop 3rd Duty	-10.0	218.72
	Stop 2nd Duty	-15.5	217.04
SET POINTS FOR WET WEATHER FLOW			
WWF SET POINTS	DUTY SELECTION	SURGE WELL LEVEL	
		City Datum	m
City datum: -15' (-4.57 m)	Start 2nd Duty	-13.5	217.65
	Start 3rd Duty	-12.0	218.11
Geodetic datum: 217.19	Start 4th Duty	-10.5	218.56
	Start 5th Duty	-9.0	219.02
	Start 6th Duty	-8.0	219.33
	Stop 6th Duty	-15.0	217.19
	Stop 5th Duty	-16.5	216.74
	Stop 4th Duty	-17.5	216.43
	Stop 3rd Duty	-18.0	216.28
	Stop 2nd Duty	-18.5	216.13

head pump utility in XP-Extran. The pump start and stop elevations were set in accordance with the WWF setpoint. The pumping rates were established to match the existing operations for a specific range, ie, between 218.11 and 218.56, pump rates ranged from 3.04 to 6.53 cubic metres per second (cms). No attempt was made to model the NEWPCC processes. The pump model was developed to artificially route the flows from the surge well directly to the NEWPCC outfall sewer, which discharged to the river.

2.5 EXISTING WWF OPERATION

This section comments on the manner in which the interceptor sewers respond to WWF conditions in general, using hydraulic analysis and other monitoring information as noted earlier. The primary emphasis is on the Main Interceptor, since it is dominated by combined sewage.

2.5.1 Main Interceptor

Hydraulic analyses were done on the Main Interceptor for a number of different conditions, to estimate how the system responded to widespread WWF conditions. The analyses began with a review of the "design" conditions and progressed to an evaluation of likely field conditions.

The initial conditions for each analysis were based on ADWFs in each of the Main/NE/NW Interceptors. A set point of -4.57 m (-15') was used for the downstream condition (at the NEWPCC surge well) to establish a pumping protocol consistent with existing WWF operations at the NEWPCC.

Conditions at 2.75 DWF

The initial model runs for interceptor analysis assumed typical area - wide interception rates of 2.75 times ADWF as input from the CSO Districts and 4 times ADWF from the Separate Districts. The CSO District value of 2.75 was selected on the basis of the initial design of the district diversions. The Separate District value of 4 was selected as being the design criteria (i.e., 4 times DWF for in-system storage oversizing determination) for the Northeast and Northwest Interceptors (WWDD, 1983). This model run was done to assess the overall hydraulic adequacy of the interceptors.

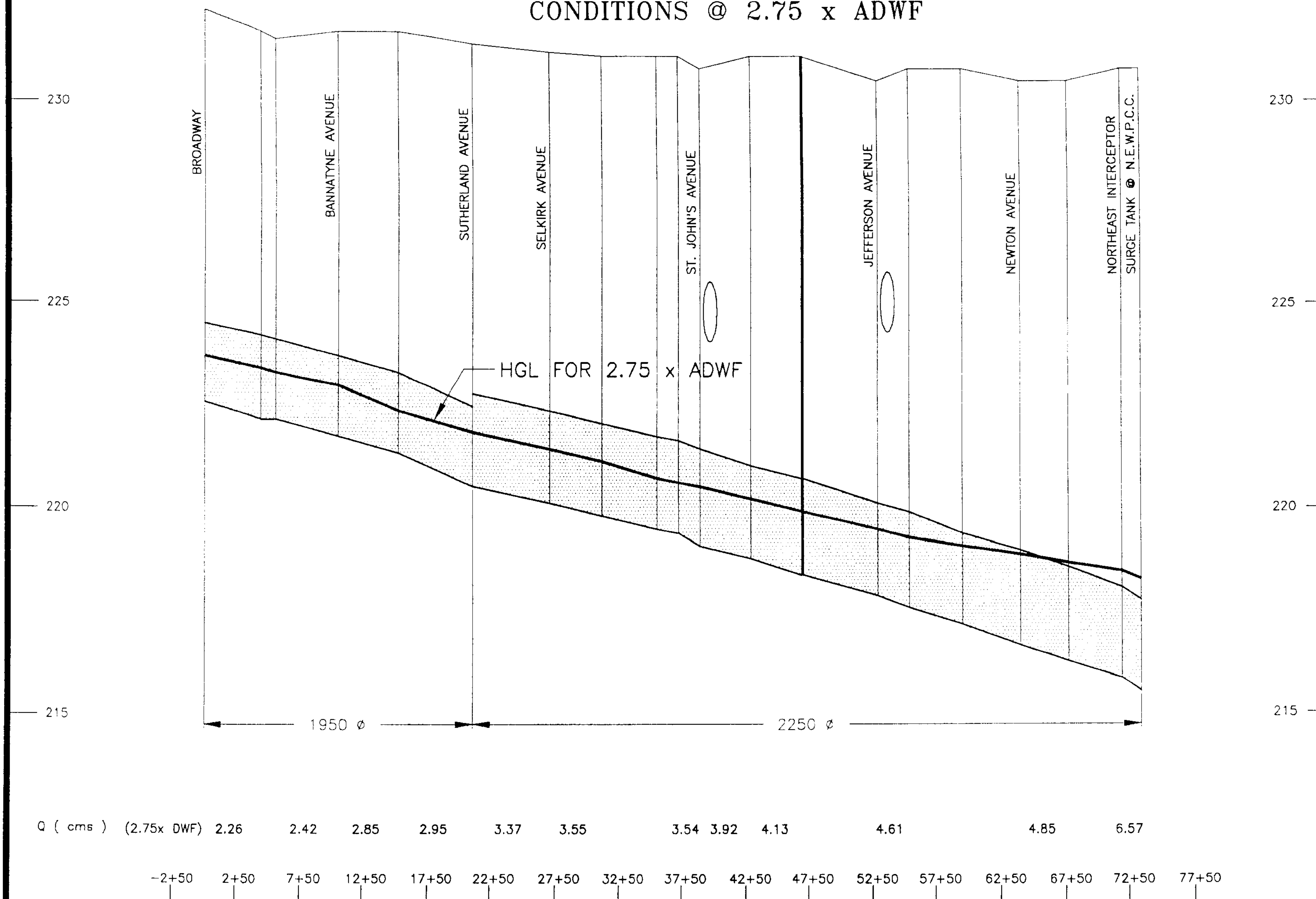
The results of this analysis indicated that the Main Interceptor can easily convey flows of 2.75 times ADWF. A hydraulic grade line plot of the Main Interceptor, from Broadway to the surge well at the NEWPCC is shown on Figure 2-12. A similar plot of the west leg of the interceptor, from Ferry Road to Main Street, is shown on Figure 2-13. The interceptor sewer, with the exception of the downstream end adjacent to the plant, will be flowing less than full under these conditions. The surcharge at the downstream end is due to the level in the surge well. This level is required to activate the number of duty pumps to convey the flow to the plant. The NEWPCC pumps are able to maintain this level while accepting these flows from the three interceptors (Main, NE and NW).

Conditions at Incipient Overflow

In the foregoing analysis, 2.75 x ADWF was assumed to be diverted from each of the CSO Districts. This was based on the initial design intent. The design considered the hydraulic losses through the system, including those losses through the comminutors (see Figure 2-14) and regulator valves. However since that time, the majority of the comminutors have been removed from the stations (see Figure 2-5). These modifications had a major impact on the hydraulics through the chambers. The result is that the diversions are no longer operating under the original design condition.

To determine the actual diversion capacities, a detailed hydraulic analysis was conducted for each district. The analyses were based on the data obtained during the field investigations, augmented by the original design drawings and calculations (where available). The diversion capacities were based on incipient overflow conditions, ie. levels in the trunk sewer at the top of the weir. The hydraulic losses in the diversion components from the trunk sewers through to the wet well (pumped districts) or secondary sewers (gravity districts) were determined and the resultant peak flow calculated. In addition, the limiting flow factor in the diversion, i.e.,

MAIN INTERCEPTOR - MAIN STREET
CONDITIONS @ 2.75 x ADWF

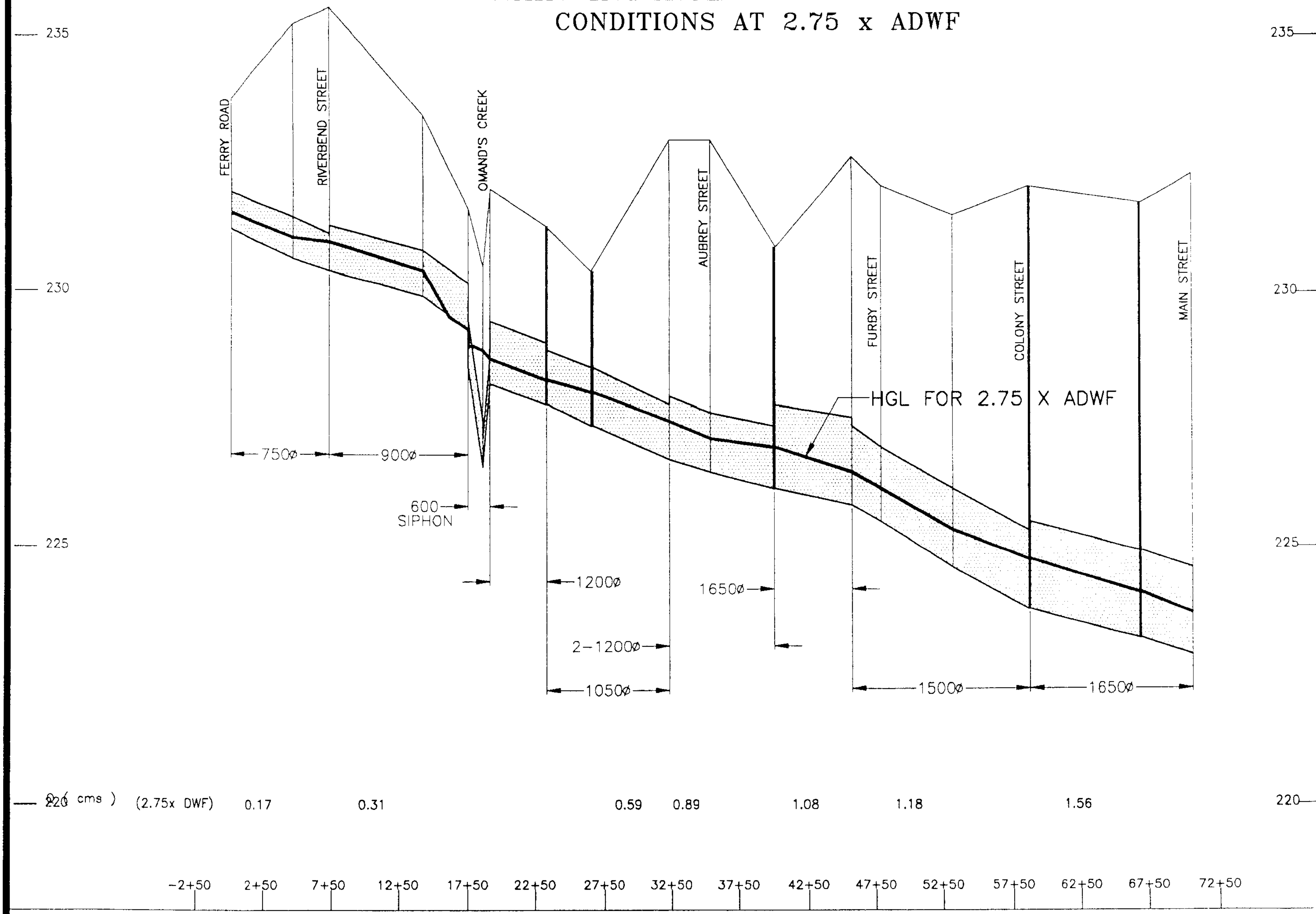


Time of Simulation : 10 : 04 : 30 Cycle No. : 1209 / 1440

FIG 2-12

MAIN-275.dwg

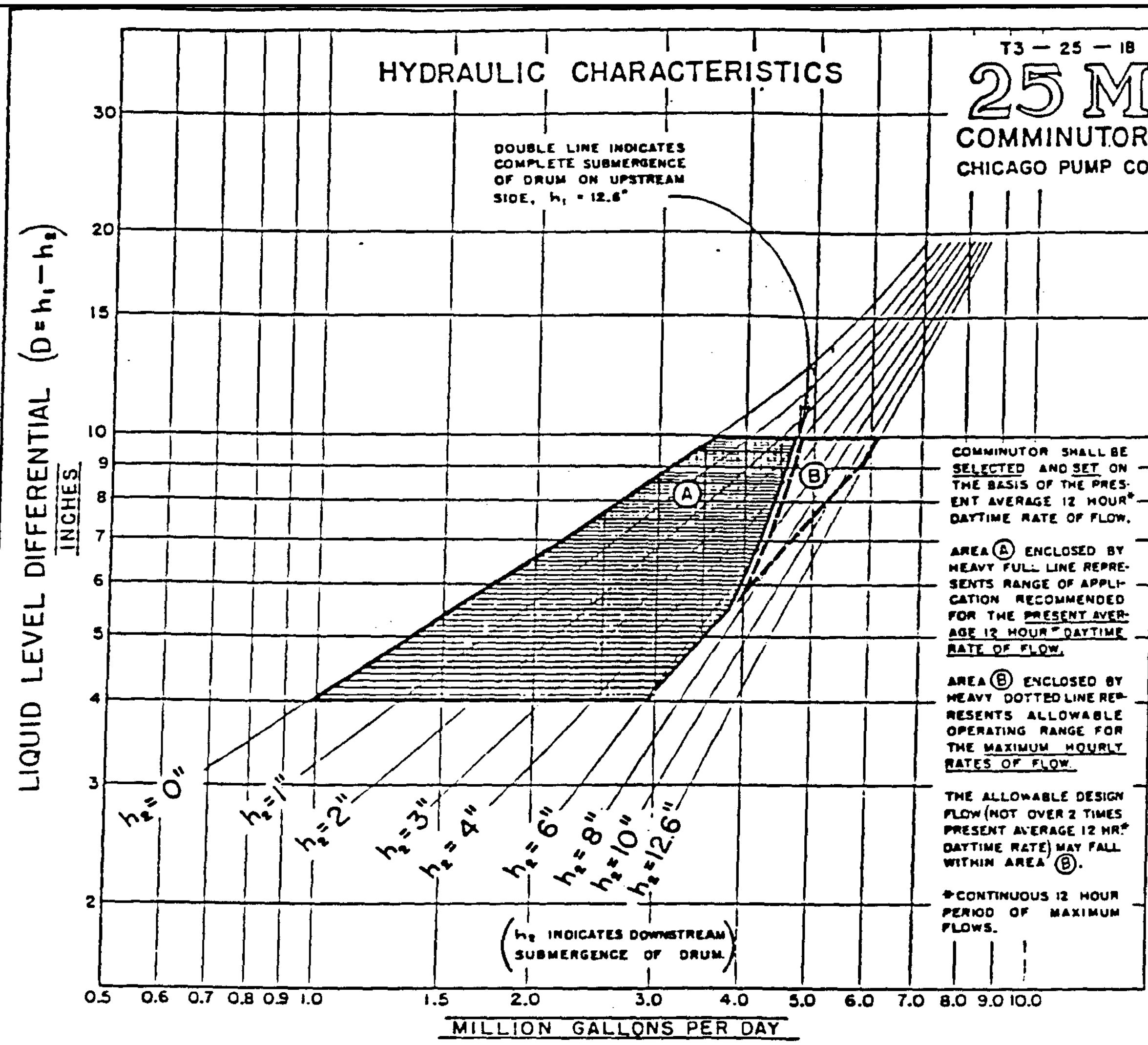
MAIN INTERCEPTOR - WEST LEG
CONDITIONS AT 2.75 x ADWF



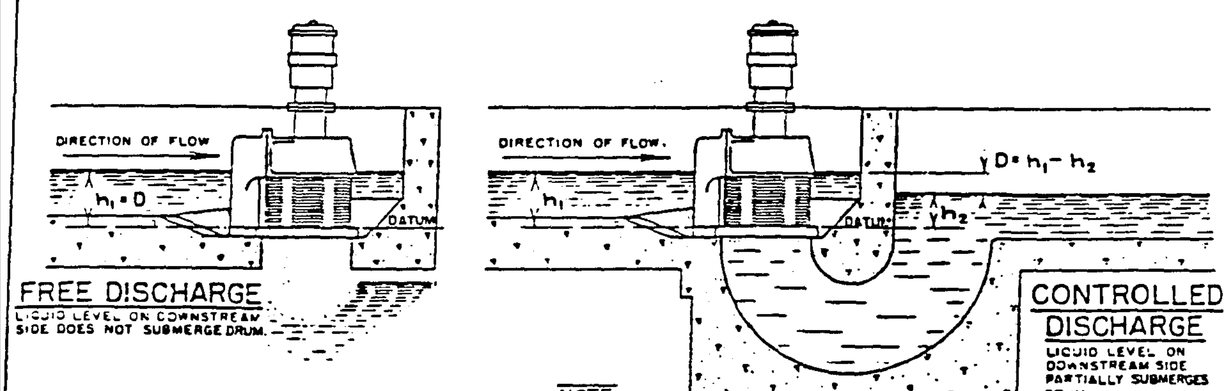
Time of Simulation : 11 : 04 : 30 Cycle No. : 1329 / 1440

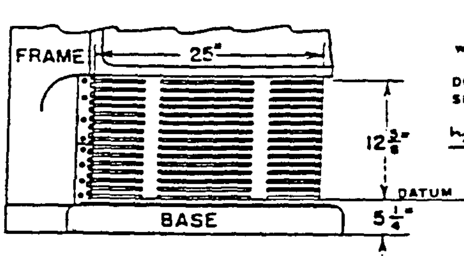
FIG 2-13

WESTL275.dwg



RATE OF FLOW THROUGH COMMINUTOR





NOTE
WHEN LIQUID LEVEL ON DOWNSTREAM SIDE IS BELOW DATUM h_2 ALWAYS = 0

GENERAL DATA	
AVERAGE SLOT WIDTH	1/8 INCH
DRUM INLET AREA	325 SQ. IN.
DRUM OUTLET AREA	312 SQ. IN.
DRUM SPEED	23 R.P.M.
SIZE MOTOR REQUIRED	1 1/2 H. P.
NUMBER OF SLOTS	90

FIG 2-14

the weir height, pumping capacity, or diversion pipe, was determined. The results of this analysis for the CSO districts tributary to the NEWPCC, are shown on Table 2-5.

The system modifications had a significant impact on the district diversion capacities. This is most notable in the gravity flow districts. For example, the flows in Armstrong and Newton are equivalent to 26.2 and 16.6 times ADWF, respectively. The pumped districts, are in most cases limited by the capacity of the existing pumps which are usually close to 2.75 times DWF or above (with all pumps running).

On Table 2-5 the diversion capacities are expressed in terms of multiples of ADWF. The ADWF values are based on 1.35 times water consumption. As noted in Section 1.4.3, this provides a valid estimate of DWF for the NEWPCC service area, but may not be representative on a district basis. To obtain a better estimate of current DWF, information from the City sewer gauging program was acquired.

The information consisted of the sewer gauging data gathered since 1992 (that had been reduced to digital format). This data included gauged flows from 11 of the 42 combined sewer districts. (While additional historical gauging data was available, it was determined that the data from the 11 districts would provide an accounting of the accuracy of the DWF estimates based on water use.) The data was analyzed to determine the average hourly flow and the average DWF for the gauging period. The ADWF (from the sewer gauging data) was then compared to the values derived from the water use records (see Table 2-6). This analysis indicated close correlation in some districts (eg., Hawthorne, Syndicate), but a wide variation in others (eg. Bannatyne, Mager). The results also indicate that the sum of the flows from the sewer gauging is much less than the sum of the flows derived from the water use records. This would seem to indicate that:

- there are several districts (none of them monitored since 1992) that dominate in terms of extraneous flows; or
- there is a major extraneous flow (eg. leaking river crossing); or
- the data from the NEWPCC may not accurately reflect the wastewater flows; or
- a combination of the above.

Table 2-5
CSO District Diversion Capacity (NEWPCC)

District	DWF	2.75 times DWF	Diversion Capacity *		Diversion Type	Limiting Factor
	cms	cms	cms	multiple of DWF		
NEWPCC						
Armstrong	0.020	0.055	0.524	26.2	gravity	weir
Linden	0.017	0.047	0.060	3.5	pumped	comminutor
Hawthorn	0.036	0.099	0.113	3.1	pumped	pump Q
Newton	0.010	0.028	0.166	16.6	gravity	weir
Munroe	0.077	0.213	0.237	3.1	gravity	weir
Polson	0.032	0.088	0.356	11.1	gravity	weir
Jefferson	0.143	0.392	0.569	4.0	gravity	weir
St. Johns	0.084	0.232	0.173	2.1	gravity	weir
Hart	0.039	0.108	0.101	2.6	pumped	pump Q
Selkirk	0.067	0.185	0.453	6.8	gravity	weir
Roland	0.026	0.071	0.324	12.5	gravity to P/S	diversion
Mission	0.144	0.397	0.517	3.6	gravity to P/S	diversion
Montcalm P/S			0.841	4.9	P/S	N/A
Boyle	0.014	0.038	0.030	2.1	pumped	pump Q
Syndicate	0.010	0.027	0.069	6.9	pumped	pump Q
Alexander	0.035	0.097	0.155	4.4	gravity	weir
Bannatyne	0.153	0.422	0.613	4.0	gravity	weir
Assiniboine	0.084	0.231	0.425	5.1	gravity	weir
Despins	0.032	0.089	0.132	4.1	pumped	pump Q
Dumoulin	0.013	0.035	0.136	10.5	pumped	pump Q
Laverendrye	0.009	0.025	0.015	1.7	gravity to P/S	pump Q
Jessie	0.066	0.181	0.176	2.7	pumped	pump Q
River	0.070	0.193	0.094	1.3	pumped	pump Q
Marion	0.032	0.088	0.220	6.9	pumped	pump Q
Colony	0.134	0.370	0.425	3.2	gravity	weir
Cornish	0.035	0.098	0.107	3.1	pumped	pump Q
Aubrey	0.071	0.195	0.214	3.0	pumped	pump Q
Clifton	0.077	0.212	0.236	3.1	pumped	pump Q
Tylehurst	0.050	0.138	0.176	3.5	pumped	pump Q
Riverbend	0.053	0.145	0.108	2.0	pumped	weir
Ferry Road	0.059	0.161	0.126	2.1	pumped	pump Q
Douglas Park	0.001	0.004	0.095	95.0	gravity to P/S	pump Q
Ferry Road P/S			0.126	2.1	P/S	pump Q
Tuxedo	0.004	0.011	0.036	9.0	pumped	pump Q
Doncaster	0.025	0.068	0.100	4.0	gravity to P/S	pump Q
Ash	0.082	0.225	0.301	3.7	pumped	pump Q
Ash P/S			0.301	2.7	P/S	pump Q
All Districts	1.804	4.961	7.351	4.1		

**** At incipient overflow of the diversion weir, ie. imminent overflow***

Table 2-6
Comparison of Sewer Gauging to Water Use Records

District	ADWF at 1.35* water use	ADWF from sewer gauging	Diversion Capacity * (cms)	Diversion Capacity as a multiple of DWF	
				water use	gauging
Hawthorne	0.036	0.038	0.113	3.1	3.0
Boyle	0.014	0.002	0.030	2.1	15.0
Syndicate	0.010	0.008	0.069	6.9	8.6
Bannatyne	0.153	0.093	0.613	4.0	6.6
Despins	0.032	0.016	0.132	4.1	8.3
Dumoulin	0.013	0.004	0.136	10.5	34.0
Laverendrye	0.009	0.007	0.015	1.7	2.1
River	0.070	0.043	0.094	1.3	2.2
Riverbend	0.053	0.035	0.108	2.0	3.1
Mager	0.158	0.092	0.309	2.0	3.4
Strathmillan	0.003	0.005	0.062	20.7	12.4
Total	0.551	0.343	N/A	N/A	N/A

** at incipient overflow of the diversion weir, ie. imminent overflow*

In any case, it is apparent that the individual district diversion capacities should be reviewed, based on the data gathered from the sewer gauging program. This will be done in Phase 3.

The initial hydraulic analysis of the interceptor system was based on flows in multiples (1.0 and 2.75) of assumed ADWF values. Based on the results of the hydraulic analyses of the diversions under incipient overflow, which indicated a wide variation in the interception rates from district to district, it was determined that the previous simulations did not provide a good representation of the existing WWF operation of the interceptor system. Therefore the interceptor system was reassessed using the interception rates for each district as calculated at incipient overflow.

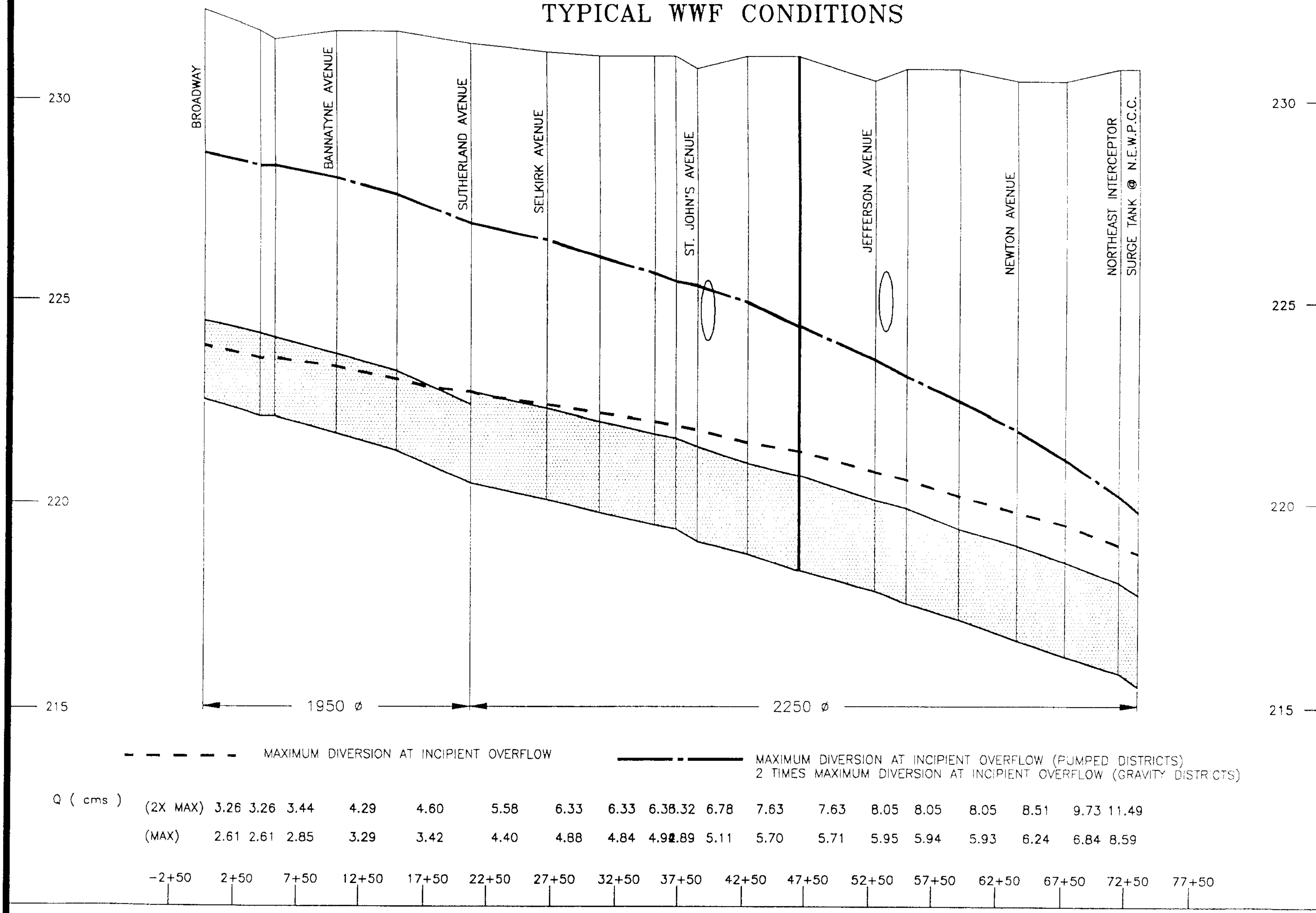
The input for the separate sewer districts tributary to the Northeast and Northwest interceptors was adjusted to 8 times ADWF. This was done for two reasons. The individual district wastewater collection and pumping facilities are sized based on DWF plus infiltration and inflow, both direct and from weeping tile flows (where applicable). The I/I component varies from district to district, but the aggregate flow is typically in the range of 7 to 9 times the DWF. Hence, the separate districts are typically capable of conveying 8 times DWF to the interceptors. Secondly, although the interceptors were designed to convey 4 times DWF, the development of the tributary area has not progressed to the stage that the ultimate DWFs have been realized. Therefore, the Northeast and Northwest Interceptors are capable of conveying 8 times current DWF.

These multiples of DWF were then input to the interceptor model. The results of this computer simulation are shown on Figure 2-15 (Main Street) and Figure 2-16 (West Leg). This indicates that the Main Interceptor would be surcharged from approximately Sutherland Avenue to the NEWPCC (mainly the backwater effects of the pump operation). The remainder of the Main interceptor, and the West leg of the interceptor would be flowing less than full.

Conditions at Typical WWF

An additional computer simulation was carried out for the NEWPCC interceptor system to reflect more realistic WWF conditions. During the detailed hydraulic analysis of the interception points it was realized that, during WWF, flow levels in the trunks would rise

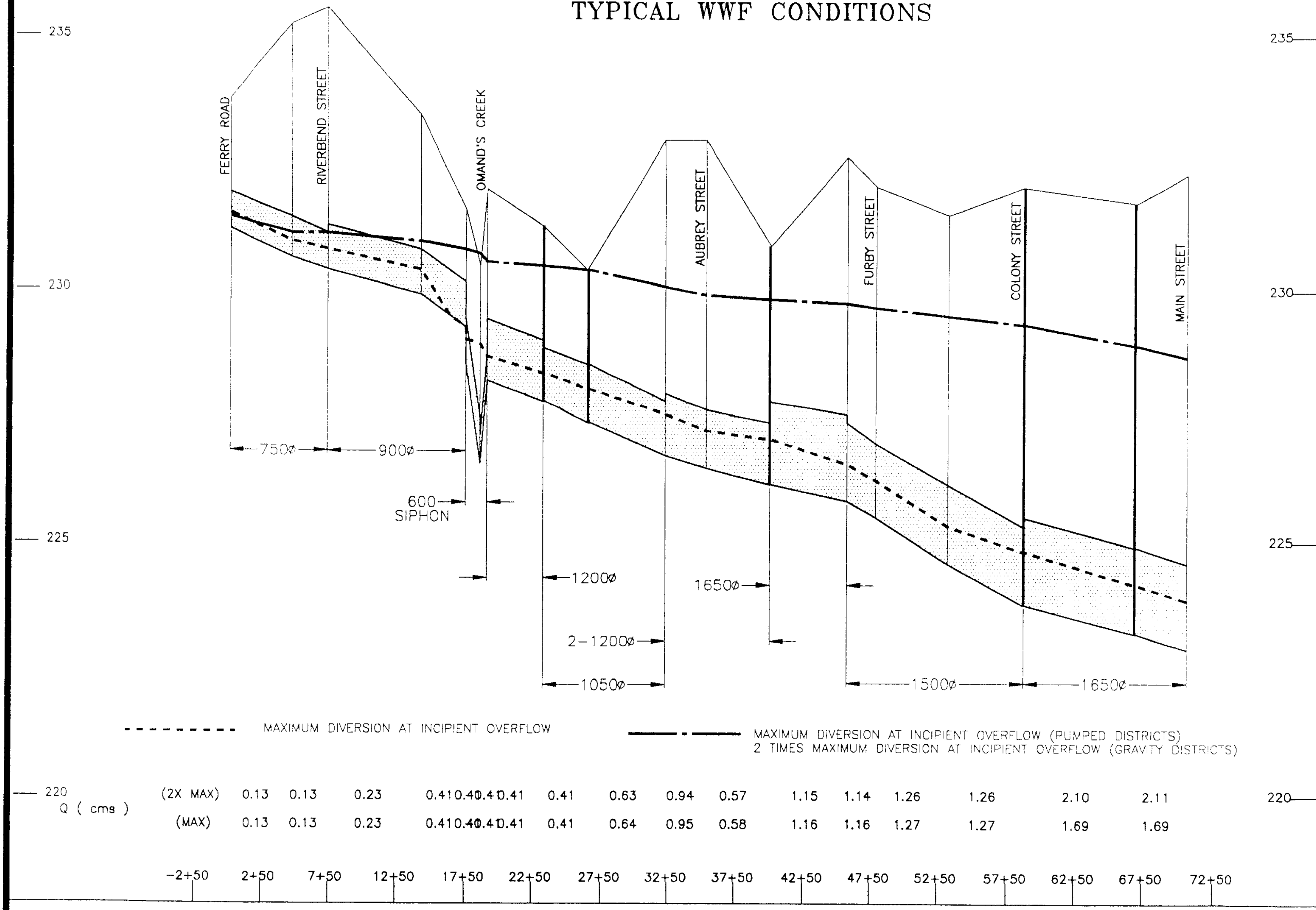
MAIN INTERCEPTOR – MAIN STREET
TYPICAL WWF CONDITIONS



Time of Simulation : 10 : 04 : 30 Cycle No. : 1209 / 1440

FIG 2-15

MAIN INTERCEPTOR – WEST LEG
TYPICAL WWF CONDITIONS



Time of Simulation : 11 : 04 : 30 Cycle No. : 1329 / 1440

FIG 2-16

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above weir levels. This would result in increased head on the off-take piping which would increase the flows through the diversion chambers (from those occurring at incipient overflow of the weir). This effect is most severe in the gravity districts. In the pumped districts, the peak pumping rate would govern the maximum flow to the interceptor.

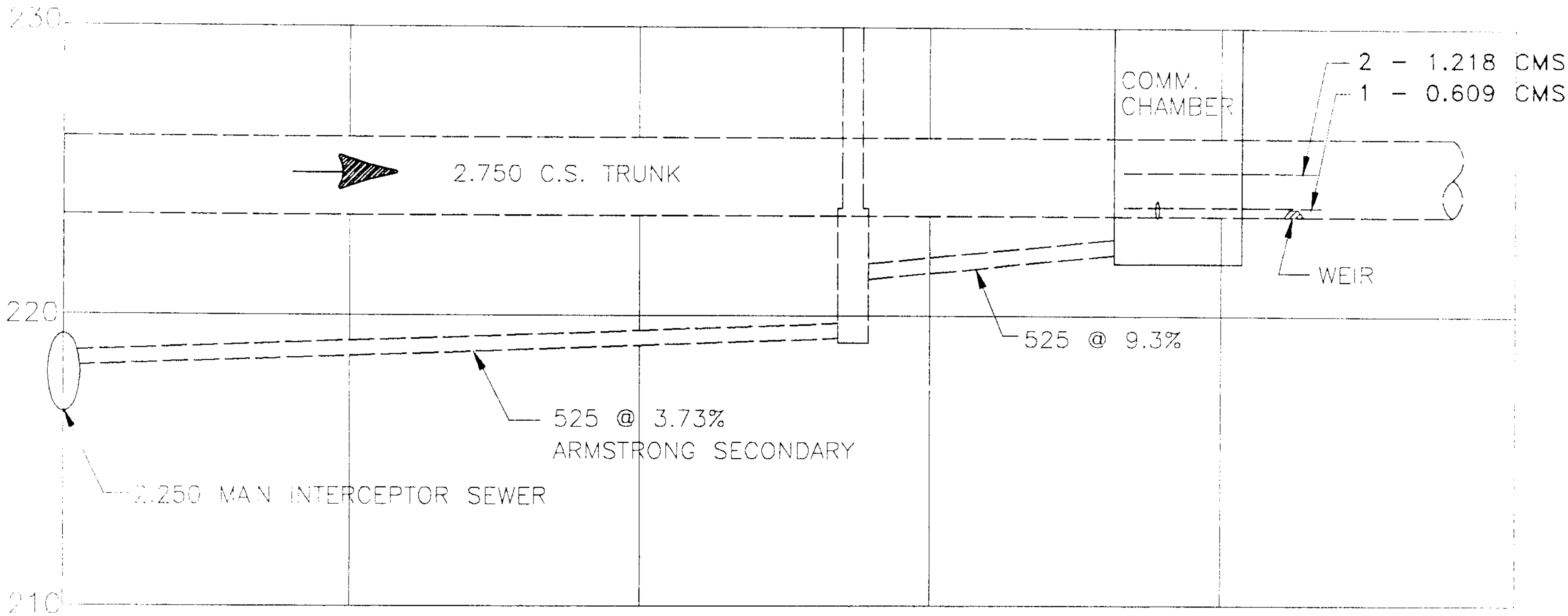
A sample calculation was carried out for the Armstrong District. This indicated that, if the trunk was flowing at approximately half full, the flow through the diversion would be doubled from 0.61 cms to 1.22 cms (see Figure 2-17) . These conditions (flowing half full) are almost certain to occur in most of the trunks after a significant rainfall event. Therefore, to review the interceptor hydraulics under more realistic conditions, the model input (to the Main Interceptor) consisted of peak pumping capacity from the pumped districts and two times the diversion capacity (at incipient overflow) from the gravity districts. The separate district flows to the NE and NW Interceptors remained at 8 times ADWF.

The results of this analysis are also plotted on Figures 2-15 and 2-16 and indicate that flows of this magnitude exceed the capacity of the interceptor system. The hydraulic grade line plot shows that the interceptor would overflow at St. Johns Avenue (the overflow was not modelled), and street flooding would occur at Clifton Street and at Omand's Creek on the West leg.

It is also apparent that, at these levels in the interceptor, there would be insufficient head available for the secondary sewers to convey two times the diversion rate from the gravity districts to the interceptor. Therefore, the next step in assessing the interceptors WWF performance will include the combined sewer trunks and interception points in the model. This, in conjunction with input hydrographs provided by the regional runoff model (from actual storm events), will provide a dynamic look at the existing WWF performance of the interceptor system. This will be part of the works in Phase 3.

The results of the foregoing hydraulic modelling analyses have helped to develop an understanding of how the Main Interceptor system and the NEWPCC perform under WWF conditions. It is assumed that the NE and NW Interceptors can convey all diverted flows from the separate districts (approximately 8 x ADWF) to the NEWPCC. Furthermore, at the beginning of the storm, flows to the Main Interceptor are dominated by the gravity districts

ARMSTRONG DISTRICT
CALCULATED DIVERSION RATES



1 - AT 224.0 (WEIR HEIGHT) - 0.609 CMS (21.5 CFS.)

2 - AT 225.25 - 1.218 CMS (43 CFS.)

FIG 2-17

at the downstream end of the Interceptor. When these flows reach the NEWPCC, there is a rise in the level in the surge well until the required number of pumps come on duty to pass the flows on to the primary treatment facilities. The surge well then becomes the downstream control for the Interceptor hydraulics. As the flows from the upstream end of the Interceptor (west and south legs) reach the WPCC, there is a corresponding rise in the hydraulic grade line along the Interceptor.

Since the flows from the districts on the upstream reaches are mainly pumped, they are not significantly impacted by the surcharge conditions. Therefore, the levels at the upstream end of the Interceptor continue to rise. At this time, if the levels are higher than the levels in the combined trunks, overflows would start to occur at Assiniboine and Colony. (The regulator gates in these districts are locked open, therefore, wastewater in the Interceptor could flow through the connection piping into the trunk sewers and discharge to the rivers.) As the levels in the Main Interceptor increase, the next overflows would occur at St. John's, and finally at Jefferson.

2.5.2 South End Interceptor

A detailed hydraulic analysis of the South End interceptor system was not conducted. This was due to the relative lack of influence of combined sewage flows on the interceptor system. The combined sewer area comprises approximately 20% of the 7,700 ha. tributary area. This area is made up of the following four combined sewer districts:

- Cockburn/Calrossie;
- Baltimore;
- Metcalfe; and
- Mager Drive.

The flow diversions from the various districts is unique in that only the Mager Drive District is connected to the interceptor system (see schematic on Figure 2-18). The minuscule Calrossie area (approximately 10 ha.) is directed by gravity into the Cockburn combined sewer collection system. Diverted combined sewer flows from Cockburn are pumped to the

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SEWPCC CSO DISTRICT SCHEMATIC

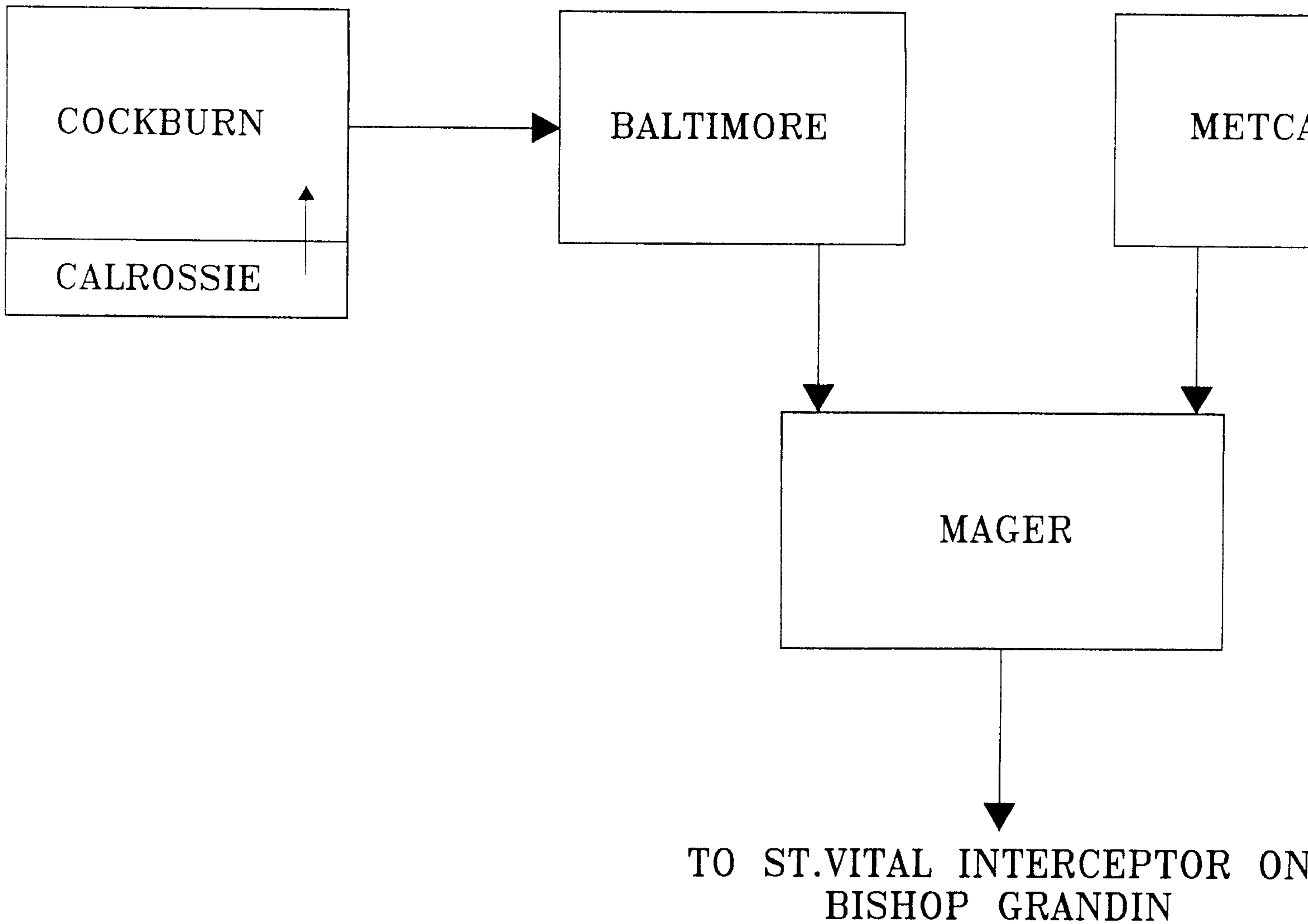

FIG 2-18

Baltimore system. Diverted flows from Baltimore are then pumped to the Mager Drive District. Combined flows from the small Metcalfe District are also pumped to the Mager Drive System. In turn, flows from Mager are pumped to the St. Vital Interceptor on Bishop Grandin Boulevard. In this regard, the combined districts' diversion capacity to the SEWPCC Interceptor system is limited to the capacity of the Mager Drive pumping station.

The diversion capacities for each of the Districts, and the Calrossie area were also analyzed for incipient overflow conditions. The results of this analysis are shown on Table 2-7. This indicates that, since all diverted flows from the combined area pass through the Mager District, the peak diversion from the combined area to the SEWPCC interceptor system is 0.309 cms or 2 times ADWF (based on water use records). This further indicates that, since flows of up to 6 times ADWF have been recorded at the SEWPCC (Wardrop/Tetr*ES* 1993), the system is dominated by WWF from the separate areas. Therefore, in Phase 3, the WWF from the separate districts should be investigated further to determine the impact on the Interceptor system and on the SEWPCC.

The cascading effect of intercepted flows may have a negative impact on the number of overflows to the river. For example, an isolated storm event in the Baltimore and Metcalfe Districts would result in 0.201 cms (peak pumping capacity) and 0.044 cms, respectively, being diverted to the Mager District. This flow, in addition to DWFs from Mager (0.091 cms), surpasses the pumping capacity of the Mager District (0.309 cms). Therefore, the Mager District could overflow without receiving any significant rainfall. This potential situation could be remedied through the extension of the Interceptor system into the Baltimore, Cockburn, and Metcalfe Districts. Since the districts overflow to the potentially critical south leg of the Red River, the merits of an expanded Interceptor system will be analyzed further in Phase 3.

2.5.3 West End Interceptor

The three tributary combined sewer districts (Strathmillan, Moorgate and Woodhaven) comprise less than 10% of the WEWPCC tributary area. Accordingly, PWWF to the plant is only moderately influenced by combined flows.

Table 2-7
CSO District Diversion Capacity (SE & WEWPCC)

District	DWF cms	2.75 times DWF cms	Diversion Capacity * cms	Diversion Capacity * multiple of DWF	Diversion Type	Limiting Factor
SEWPCC						
Calrossie	0.001	0.003	0.028	28.0	gravity	weir
Cockburn	0.033	0.092	0.075	2.3	pumped	pump Q
Cockburn P/S	0.034	0.095	0.075	2.2	P/S	pump Q
Baltimore	0.028	0.080	0.201	7.2	pumped	pump Q
Baltimore P/S	0.062	0.175	0.201	3.2	P/S	pump Q
Metcalfe	0.005	0.014	0.044	8.8	pumped	pump Q
Mager	0.091	0.251	0.309	3.4	pumped	pump Q
Mager P/S	0.158	0.44	0.309	2.0	P/S	pump Q
WEWPCC						
Moorgate	0.023	0.064	0.085	3.7	pumped	pump Q
Strathmillan	0.003	0.009	0.062	20.7	gravity	weir
Woodhaven	0.012	0.032	0.027	2.3	pumped	pump Q

** at incipient overflow of the diversion weir, ie. imminent overflow*

The diversion capacities of the three districts were also calculated for incipient overflow conditions. This information is also shown on Table 2-7.

The total diversion capacity from the three districts at incipient overflow is 0.174 cms. This is probably close to the maximum diversion rate since Moorgate and Woodhaven are governed by pumping capacity, leaving only the gravity district, Strathmillan, with increased diversion flow potential. In comparison, the rated capacity of the WEWPCC is 1.30 cms (112 ML/d). Therefore, as in the SEWPCC system, the interceptor system is dominated by WWF from the separate districts. Accordingly, these WWFs from the separate areas should be investigated in Phase 3 to determine their impact on the WEWPCC system.

2.6 RECOMMENDED MONITORING

At the start of the study, there was a significant lack of information regarding the hydraulic performance of the combined sewer interception points, interceptor conveyance, and NEWPCC pumping and surge well levels during WWF. In Phase 1, programs were initiated to inspect the district diversion structures and to monitor and record raw sewage pumping and surge well levels at the NEWPCC. In Phase 2, flow and level monitors were placed on the downstream ends of the Northeast and Northwest Interceptors. Unfortunately, since installation of the monitors, the City has experienced a prolonged period of dry weather, resulting in little usable WWF data (until very recently). However, in the long-term, with this data, the tributary flow data from the Main Interceptor will be determined from comparison to the NEWPCC pumping data.

In addition, levels (only) are being recorded at 5 points along the Main Interceptor sewer. Levels are also being recorded at 4 locations in the Clifton District. However, the latter monitoring relates to the use of inline storage as a control alternative and is discussed in TM #3, "Control Alternatives." The location of the monitoring equipment is shown on Figure 2-19.

The foregoing inspections and monitoring data were required to provide an accurate description of the Main/Northeast/Northwest Interceptor system. However, additional

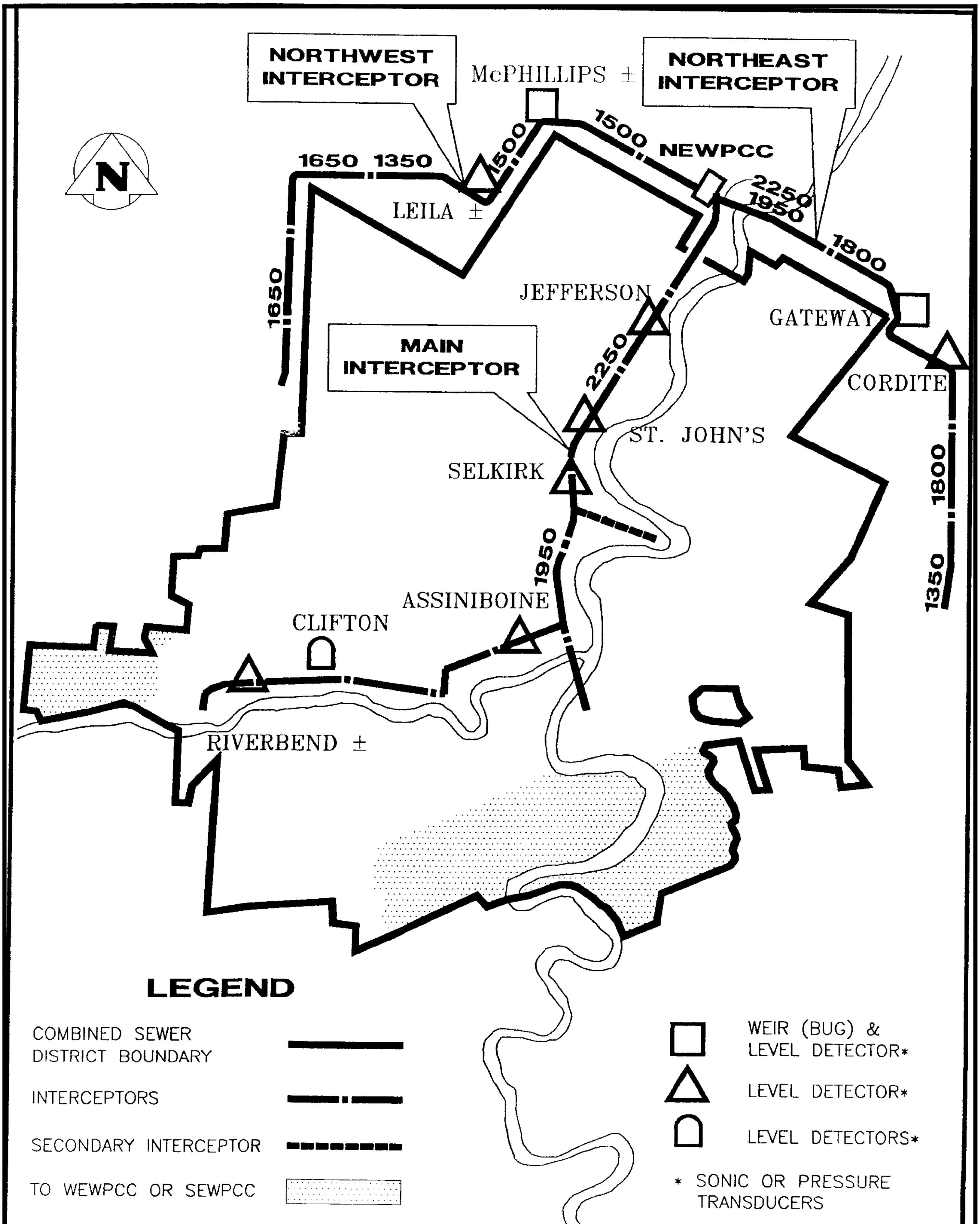

FIG 2-19

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monitoring of the systems is recommended for future analysis to verify flow conditions in the interceptors and on a district level. In addition, WWF monitoring of the NEWPCC and SEWPCC interceptor systems is recommended to enhance the WWF perspective and define the impact of the CSO districts. Details of the recommended monitoring works are included in the following sections.

2.6.1 Main/NE/NW

It is recommended that the existing monitoring of the flows and levels in the Main, Northeast and Northwest interceptors, as well as the NEWPCC pumping and surge well levels be continued. This data is necessary to quantify the WWF contributions from the Northeast and Northwest tributary areas. It also establishes the downstream hydraulic conditions (of flow and level) at the plant and provides DWF data for the entire service area. Furthermore, the level recorders on the Main Interceptor, near the St. Johns and Jefferson Interceptor overflow structures, will provide an indication of when the Interceptor overflows to the combined sewer system.

In addition, the FAST alarm systems should be investigated with regard to combined sewer overflows. The majority of the combined sewer districts are tied into the FAST alarm network. These alarms are basically Flygt-type ball switches in the combined sewers or diversion chambers which tip over and transmit a signal to a central location whenever a certain level is reached. The location of these alarms should be reviewed with City Operations personnel to ensure that they are properly placed (in view of system modifications, eg., comminutor removal, etc.) to indicate the time and duration of a CSO. At that time, a procedure could be implemented to ensure the FAST alarm data is passed along to the study team. This data would be used to determine the combined systems response to rainfall and characterize the overflow potential.

2.6.2 South End

The total flows to the SEWPCC are currently being monitored and recorded. In addition, monitoring information is being gathered at the D'Arcy and Windsor Park pumping stations (pump flow meters and alarm history) and the Killarney secondary sewer (flow, velocity, depth and quality). We recommend additional level monitoring at the major overflow points at the D'Arcy pumping station (Fort Garry) and the St. Mary's outfall (St. Vital interceptor). This would provide valuable data on the amount of WWF entering the system (with the current monitoring data) and establish the interceptor systems response and overflow potential (volume and duration) for WWF events. Since the diverted flows from the tributary combined sewer districts are pumped, their impact and potential for increased interception rates can easily be established. The data will also be valuable in assessing the impact of I/I flows on treatment at the WPCC.

2.6.3 West End

Flows at the WEWPCC are, like the SEWPCC, monitored and recorded. In addition, data is being gathered at the Community Row and Perimeter Road pumping stations (pump flow meters and alarm history) with proposed (1996) monitoring installations at Dieppe and Parkdale (flow, velocity, depth and quality). Similar to the SEWPCC system, we recommend additional level monitoring at the major interceptor system overflows at Parkdale (St. James) and the Community Row pumping station (Charleswood).

2.6.4 Sewer Gauging

The City has been gathering DWF data on the combined and separate sewer districts for a number of years. The sewer gauging is typically carried out for 1 to 2 weeks per district during the winter months. The data provides an up-to-date, as well as a historical perspective on DWF in any particular district.

The accumulated data will be of great importance in assessing control alternatives based on district interception rates (i.e., increased interception and inline storage). In this regard, we would recommend a continuation of the sewer gauging program, with emphasis placed on the combined sewer districts. A priority list could be established upon review of the complete sewer gauging data.

We also recommend summer DWF monitoring for districts which experience DWF overflows (eg., Tylehurst, Cockburn, Aubrey and Assiniboine). This data could be used for comparison to wintertime DWF to determine the extent of the increase in sewer flows (i.e., illicit discharges). The collection of the above data would be the first step in the elimination of DWOs to the rivers. Review of the data would suggest the appropriate course of further action, which might consist of further monitoring to determine the source of any illicit discharges or an increase in the district interception rate.

2.7 ENHANCED OPERATION OF MAIN INTERCEPTOR

The Phase 2 interceptor analysis has indicated that there is capacity in the interceptor sewer system and the NEWPCC to convey and treat flows in excess of 2.75 times ADWF. This additional capacity may provide CSO control potential through the use of increased interception rates in conjunction with in line storage. This potential for enhanced operation of the interceptor system was investigated through further hydraulic analyses assuming the interception rates could be modified (through extensive upgrades) to yield selected diversion rates of 4.0 x DWF, 5.0 x DWF, etc., until the full hydraulic capacity of the interceptor was reached.

For this analysis, the flows were increased to 4 times ADWF from the CSO Districts while using 8 times DWF from the Northeast and Northwest interceptors. This was done to match the design criteria for the conveyance and pumping facilities in the separate districts. These interceptors can easily convey these flows since the NE and NW service areas are far from being fully developed.

The results from the subsequent run indicated that the interceptor was surcharged for it's entire length on Main Street. However, the surcharge levels were still below the elevations required for an overflow at either St. Johns or Jefferson. The hydraulic grade line plot is also shown on Figure 2-20.

The surcharge conditions extended down the west leg of the interceptor down Broadway, to a point west of Colony Street. The remainder of the west leg was flowing less than full, however near surcharge conditions were noted at Riverbend Street and Greenwood Street (where flows from the Ash pumping station enter the interceptor). The hydraulic grade line for 4 times ADWF is also shown on Figure 2-21.

The results from this run indicate that the interceptor can safely convey 4 times ADWF, and the resultant flows are less than the capacity of the pumps at the NEWPCC.

In view of the system's capability to pump/carry more than 4 times ADWF in the CSO districts, its capacity in response to input flow at 5 times ADWF (from the CSO Districts; 8 times from the Separate areas) was tested. Under these conditions the entire Main Street interceptor is under approximately 3 metres of surcharge. These flows would still not result in an overflow at the two designed overflow locations at St. Johns Avenue and Jefferson Avenue (see Figure 2-20). However, under these conditions, the west leg of the interceptor is at capacity (see Figure 2-21). Surcharge levels nearly reach ground levels at Clifton Street.

These surcharge levels may be acceptable provided that the individual districts can still deliver 5 x ADWF to the Interceptor. This may not be a problem on the west leg considering that, with the exception of Colony, flows from all tributary districts are pumped to the Interceptor. However, the high surcharge levels may also result in the Interceptor overflowing (by backup through the diversions) to the CS trunk at the gravity district connections at the upstream end of the system (i.e., Assiniboine, Colony). The capability of the district diversion components to deliver 5 x ADWF and potential overflows, when the Interceptor is surcharged to these levels, will be investigated in detail in the Phase 3 analysis.

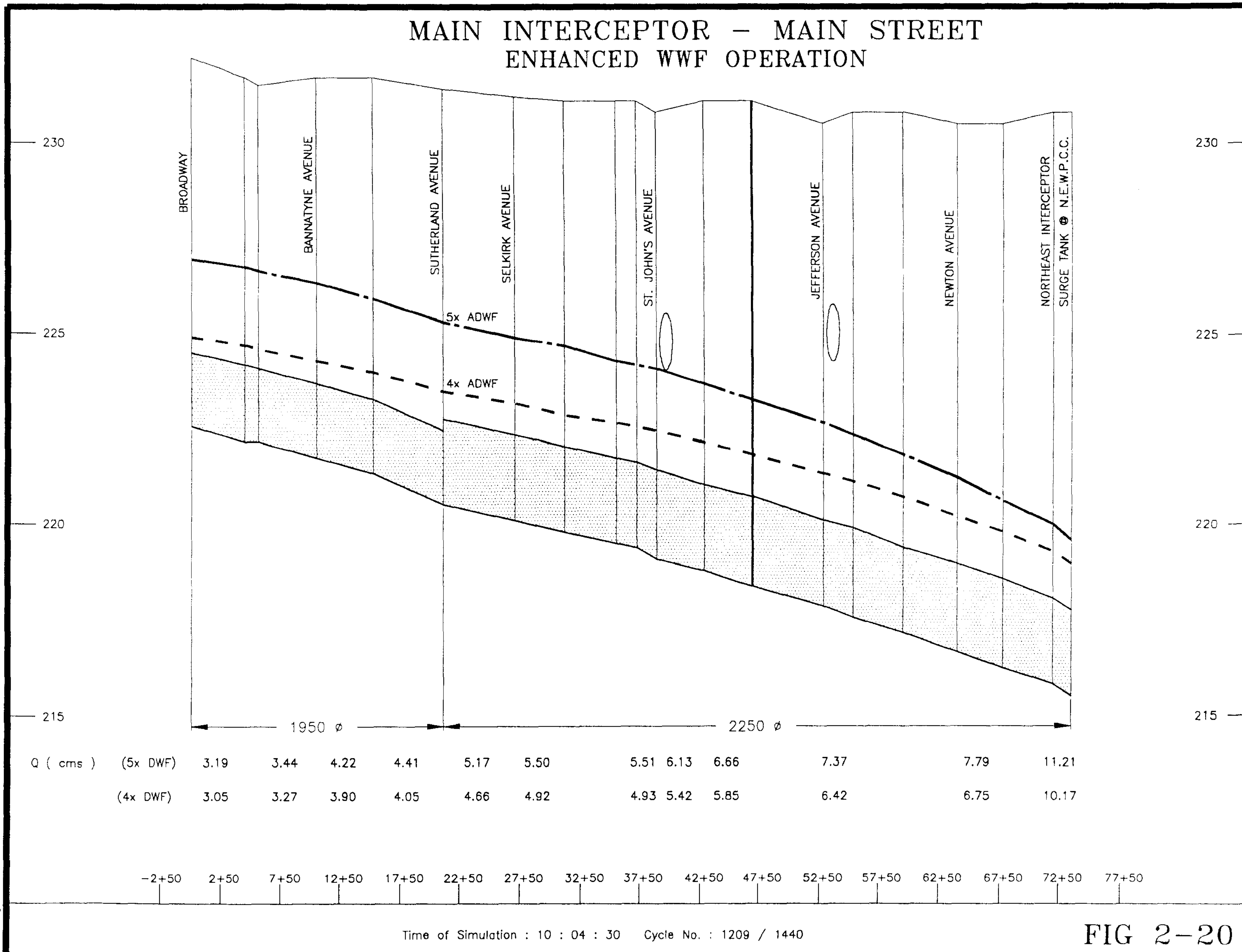
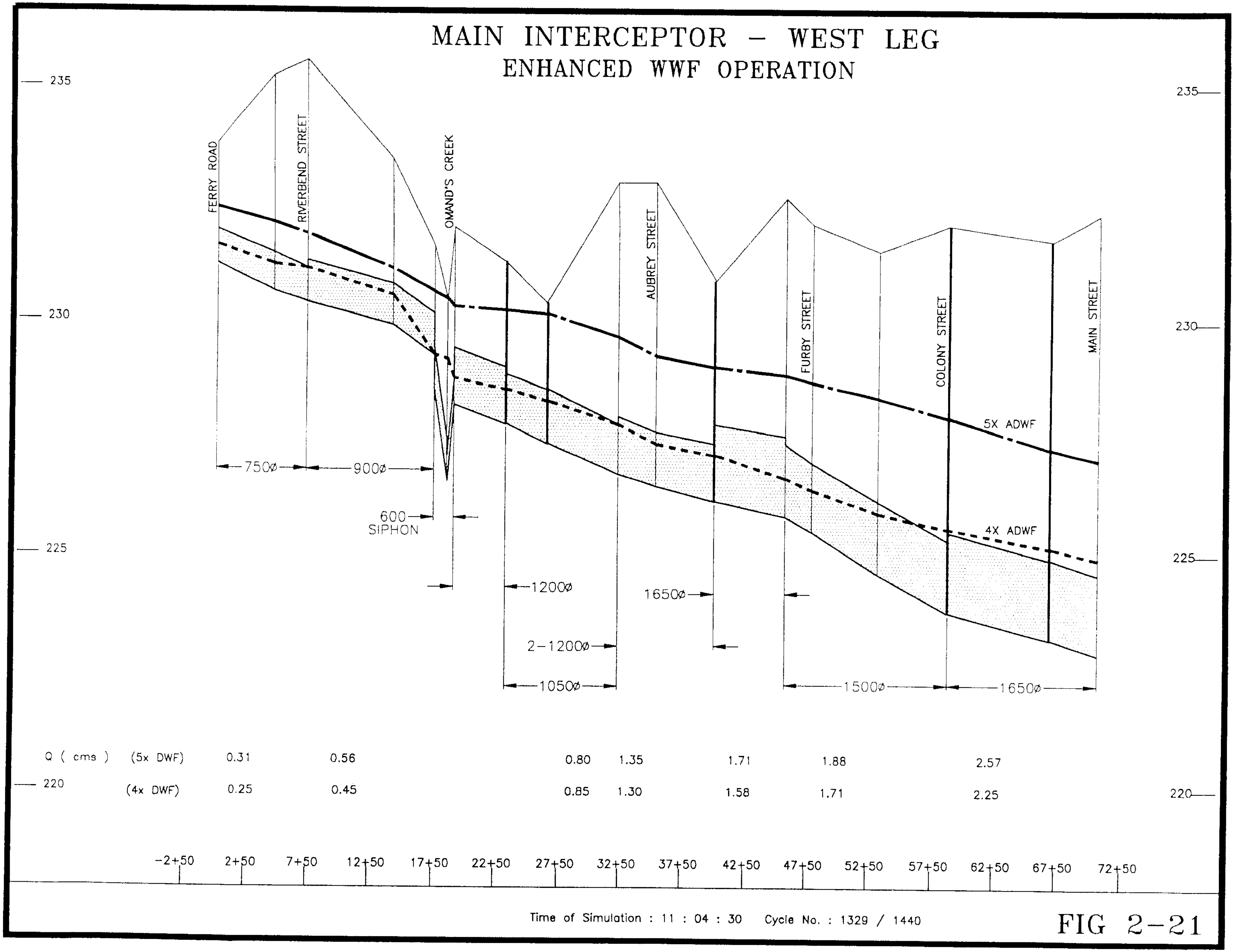

MAIN INTERCEPTOR – MAIN STREET
ENHANCED WWF OPERATION
230
225
220
215
BROADWAY
BANNATYNE AVENUE
SUTHERLAND AVENUE
SELKIRK AVENUE
ST. JOHN'S AVENUE
JEFFERSON AVENUE
NEWTON AVENUE
NORTHEAST INTERCEPTOR
SURGE TANK @ N.E.W.P.C.C.
5x ADWF
4x ADWF
1950 ø
2250 ø
Q (cms) (5x DWF) 3.19 3.44 4.22 4.41 5.17 5.50 5.51 6.13 6.66 7.37 7.79 11.21
(4x DWF) 3.05 3.27 3.90 4.05 4.66 4.92 4.93 5.42 5.85 6.42 6.75 10.17
-2+50 2+50 7+50 12+50 17+50 22+50 27+50 32+50 37+50 42+50 47+50 52+50 57+50 62+50 67+50 72+50 77+50
Time of Simulation : 10 : 04 : 30 Cycle No. : 1209 / 1440
MAIN .dwg

FIG 2-20



MAIN INTERCEPTOR – WEST LEG
ENHANCED WWF OPERATION
235
230
225
220
FERRY ROAD
RIVERBEND STREET
OMAND'S CREEK
AUBREY STREET
FURBY STREET
COLONY STREET
MAIN STREET
5X ADWF
4X ADWF
750ø
900ø
600
SIPHON
1200ø
1650ø
2-1200ø
1050ø
1500ø
1650ø
Q (cms) (5x DWF) 0.31 0.56 0.80 1.35 1.71 1.88 2.57
(4x DWF) 0.25 0.45 0.85 1.30 1.58 1.71 2.25
-2+50 2+50 7+50 12+50 17+50 22+50 27+50 32+50 37+50 42+50 47+50 52+50 57+50 62+50 67+50 72+50
Time of Simulation : 11 : 04 : 30 Cycle No. : 1329 / 1440
WESTL.dwg

FIG 2-21

The results of this analysis indicate that the interceptor can convey up to 5 times ADWF from the CSO Districts. It further indicates that the interceptor could overflow at other locations near the upstream end of the system before overflowing at the two major overflows.

2.8 SUMMARY OF FINDINGS

The Phase 2 analysis of the Winnipeg sewerage infrastructure, i.e., the interceptors and treatment plants, focused on WWF, particulary the WWF from the combined sewer districts. In this regard, the Phase 2 focus was on the Main Interceptor system and the NEWPCC which is tributary to 34 of the 42 combined sewer districts (9,200 of the 10,500 ha.). The conclusions developed during the Phase 2 analysis are as follows:

- The Main Interceptor system, including the CSO district diversions and secondary sewers, were designed to convey a minimum of 2.75 times ADWF from the combined districts (as established from sewer gauging data).

- The Northeast and Northwest Interceptors (tributary to the NEWPCC) were designed to accept 4 x DWF from the separate districts through the use of in-system storage (i.e., oversized sections). The conveyance and pumping facilities in the separate districts were designed based on WWFs equivalent to approximately 8 x ADWF. Since the areas tributary to these interceptors have not nearly reached full development, the Interceptors are conveying these flows to the WPCC.

- The rated capacity of the Main Interceptor and the common sewer receiving flows from the Northeast and Northwest interceptors is 6.14 cms and 6.83 cms, respectively (total flow capacity to NEWPCC is 12.97 cms or 1121 ML/d). The Interceptors have capacity to convey all diverted flows from minor storm events. For larger storm events, potential overflows can occur at St. John's and at other points on the upstream end of the system.

- The SEWPCC interceptors convey mainly separate sewage (4 combined districts are tributary) and were designed to convey 3 times DWF.

- The West End interceptors convey mainly separate sewage (only 3 CSO districts) and are designed to convey 3.5 times DWF.

- Visual spot inspections of the Main/NE/NW interceptors indicate generally good conditions with one exception on the Main interceptor at Sutherland Avenue where sulphide attack was noted.

- Dry weather overflows were reported at Cockburn, Tylehurst, Aubrey and Assiniboine districts. The elimination of these overflows should receive top priority as a means to control CSO impacts.

- A hydraulic model of the Main/Northeast/Northwest interceptors and the NEWPCC surge well and pumping was built to analyze the system under WWFs. The model was not calibrated to WWF data, but was deemed sufficiently representative to conduct the Planning Level, Phase 2 analysis of the system.

- The Main Interceptor can easily convey flows of 2.75 times DWF to the NEWPCC. The NEWPCC pumps can also handle these flows while also accepting 4 times DWF from the separate districts (NE and NW interceptors).

- There is a wide variation (from 1.3 to 26.2 x ADWF) in interception rates for the individual districts. Modifications to the C.S. district diversion facilities (i.e., removal of comminutors and alterations to the regulator valves) have had a significant impact on the diversion rates, raising many districts to greater than 2.75 times DWF. This is most evident in the gravity districts, since pumping capacity is a limiting factor in most of the districts where diverted flows are pumped to the interceptor system.

- Hydraulic analysis of the Main Interceptor system, under typical WWF conditions, indicates potential for overflows. A dynamic analysis of the system, including the C.S. trunks and diversion facilities, will be required to properly assess system performance.

- Flows in the SEWPCC and WEWPCC system are only moderately impacted by the tributary CSO districts (4 and 3 respectively). WWFs from the tributary separate districts should be assessed during the Phase 3 analysis.

- Monitoring programs for the Main/NE/NW Interceptors and the NEWPCC, as well as for the WEWPCC and SEWPCC systems were developed during Phase 2. Additional monitoring is recommending, including:

  - a review of the FAST alarm configuration in the CSO districts;
  - monitoring to determine the overflow volumes from the WEWPCC and SEWPCC interceptors; and
  - continuation of the sewer gauging program, with emphasis on winter gauging in the CSO districts and summer gauging in those districts experiencing dry weather overflows (Tylehurst, Cockburn, Aubrey and Assiniboine). This should be done with the aim of developing /implementing remedial measures.

- Hydraulic analysis of the Main Interceptor system (including the NEWPCC) indicate that there is sufficient capacity to convey up to 5 times DWF on an area-wide basis. This indicates potential for enhanced WWF operations (through extensive upgrades to the district diversion facilities) of the interceptor system.

2.9 PHASE 3 MODELLING

The Phase 2 modelling of the Main/NE/NW Interceptors, and the NEWPCC provided a preliminary understanding of the workings of the system under WWF conditions. The Phase 3 modelling is designed to further this understanding by developing a dynamic model of the system under WWFs. This will be accomplished by adding the district diversion facilities, including the combined trunks, diversion weirs, comminutor stations, pumping facilities (if applicable) and secondary sewers to the model. These additions will provide particularly relevant information into the workings of the system with regard to the contributions from the gravity districts which will be influenced (in some cases) by the levels in the Interceptor system.

The Main Interceptor model will be calibrated in Phase 3. This will entail a review of the monitored data gathered from the installation on the NE and NW Interceptors (in conjunction with the NEWPCC pumping data), as well as the level data obtained along the Main Interceptor.

In Phase 3, the model will be used to further investigate the potential for enhanced operation of the Main Interceptor system, i.e., up to 5 x DWF. This will include a review of the historical sewer gauging data to better determine ADWF from the individual districts, and the implications of such operations on the existing district diversion infrastructure. The model will also be used to examine the implications of increased interception rates from the DWO districts.

It is also proposed that hydraulic models of the SE and WE Interceptor systems be developed in Phase 3. This analysis would provide an understanding of the impact of WWFs from the separate districts on the Interceptor systems and WPCCs. With the data obtained from the monitoring installations on these systems, an understanding of the system flows, and the potential for overflows from the Interceptor systems to the rivers could be developed. In addition, the relative merits of extending the SE Interceptor system into the Baltimore, Cockburn and Metcalfe Districts would be investigated.

In addition to the Interceptor system modelling, three districts will also be selected for detailed system modelling in conjunction with proposed control alternatives. This will be done to assess the impact of the various alternatives on the level of basement flood protection. In this way, the study team will ensure that the proposed control alternatives do not have a negative impact on basement flood protection.

3.0 FLOOD PUMPING STATIONS

Flood pumping stations exist at the lower end of most combined trunk sewers, i.e., at the riverbank, and provide protection against basement flooding due to high river level. They are designed to pump WWF across closed gate valves into the river when the river stages are high, mainly in spring run-off. Most of these stations were constructed in the 1950's, after the major 1950 Red River flood. The stations have high, low-lift pumping capacity. The stations are discussed here because this existing infrastructure, especially the pumping capacity, is of potential relevance to some of the CSO control options.

3.1 PURPOSE

Under normal summer flow levels, the hydraulic capacity of the combined sewers is virtually unaffected by the river level in that the stormwater flow capacity is not restricted by the level of the river. In spring, the river levels are often high, due to spring snow melt and spring rains. At these times, the high river levels can reduce the normal carrying capacity of the trunk sewers, due to the reduced hydraulic head, thus making homes vulnerable to basement flooding from spring rains. The flood pumping station facility comprises: a flap gate, which automatically closes against high river levels to prevent river water from backing up into the sewer; a manually-operated positive gate, which closes off the river from the trunk sewer; and a large capacity pump to pump stormwater flows from the trunk sewer over the gate structure and into the river. The pumps are normally started manually. The flood pumping stations are usually only required in spring and even then, only sporadically, depending on the vulnerability of the tributary district.

During exceptional circumstances, such as those occurring in the summer of 1994, the river levels in the urban area can reach stages that restrict the carrying capacity of the combined sewers. While the flood pumping stations were designed for spring runoff conditions, which are less severe than summer thunderstorms in terms of urban run-off, these stations are still able to assist in controlling or limiting basement flooding in these circumstances.

3.2 POTENTIAL RELEVANCE TO CSO CONTROL

During the course of the investigations into CSO control alternatives, the potential of "end-of-pipe" storage/treatment devices (online or offline) was considered. The potential control options involve capture/treatment of CSO for subsequent conveyance to the NEWPCC. Since the flood pumping stations have significant pumping capacity to lift stormwater (combined wastewater), substantial savings in the capital costs of the offline storage or treatment devices, could be effected by using the stations to pump the flows into the devices. The savings would result from the substantial reduction in the quantities of excavation needed to install the storage/treatment facilities at or below the levels of combined sewers. Savings would also result from the possible elimination of the need to pump stored flows back into the interceptor. This potential use of the flood pumping stations is discussed in TM #3.

Available information on the capacities, and discharge levels, for some of the existing flood pumping stations was provided by the City of Winnipeg. Table 3-1 compares the capacity of the pumps in each district to the peak CSO, as determined by the runoff model for the 1992 summer period. 70% (13 of 18) of the flood pumping stations had capacity equal to the peak CSO for the 1992 scenario and 40% (2 of 5) of the remainder were close. This indicates that this existing infrastructure, while not designed for this use, has the potential to contribute to cost-effective CSO control measures.

The City of Winnipeg has engaged other consulting services to undertake a review of the flood pumping station control adequacy. This will comprise, in part, a review of the appropriateness of the design criteria for the flood pumping stations. The City has expanded this investigation to include the definition of pump capacities and discharge elevations for all of the stations. This information will be useful in assessing the feasibility and cost of offline control structures in Phase 3.

4.0 TREATMENT

4.1 INTRODUCTION

The focus of the treatment workstream is to provide information which will assist in the assessment of the existing wet weather flow impacts of the City's Water Pollution Control Centres' (WPCCs) effluents on the City's rivers and their potential role in various WWF control strategies. Details of the three Pollution Control Centres (NEWPCC, SEWPCC and WEWPCC) were provided in Technical Memorandum No. 3, produced for the Phase 1 workshop. For

TABLE 3-1

FLOOD PUMP CAPACITY VS. FLOWS - 1992

District	cu.m/hr	CMH	Flood Pump Capacity	District	cu.m/hr	CMS	Flood Pump Capacity
1	3,500	1.0	1.4	23	none		
2	2,600	0.7		24	10,500	2.9	
3	20,000	5.6		25	1,600	0.4	0.2*
4	7,500	2.1		26	3,400	1.0	2.4
5	none required			27	2,500	0.7	1.7
6	4,800	1.3		28	12,700	3.5	3.0*
7	4,500	1.2	2.1	29	1,500	0.4	1.0
8	2,700	0.8	2.8	30	2,400	0.7	2.4
9	2,000	0.6	0.8	31	1,500	0.4	
10	800	0.2		32	8,700	2.4	
11	9,000	2.5		33	1,800	0.5	2.2
12	3,000	0.8		34	9,600	2.7	1.8*
13	5,200	1.4		35	2,800	0.8	
14	1,000	0.3		36	6,000	1.7	
15	3,100	0.8	1.2	37	10,000	2.8	
16	550	0.2		38	none		
17	900	0.2		39	15,000	4.2	3.8*
18	1,000	0.5	1.8	40	500	0.1	
19	5,200	1.4		41	2,600	0.7	0.9
20	5,300	1.5	1.8	42	1,600	0.4	
21	7,000	1.9		43	8,200	2.3	
22	8,200	2.3	0.8*	44	1,200	0.3	

Notes:

- City currently having data collected on remaining flood pumping stations.
- 70% of stations of known capacity could meet needs for 1992 scenario.
  (* = those which could not)

convenience, schematics of these existing plants, along with their capacities, are provided on Figures 4-1, 4-2 and 4-3.

Insofar as the current CSO study is concerned, the main focus of this technical memorandum is the North End Water Pollution Control Centre (NEWPCC), since over 90% of the City's combined sewer area discharge to the NEWPCC.

4.2 PHASE 1 WORKSHOP OVERVIEW

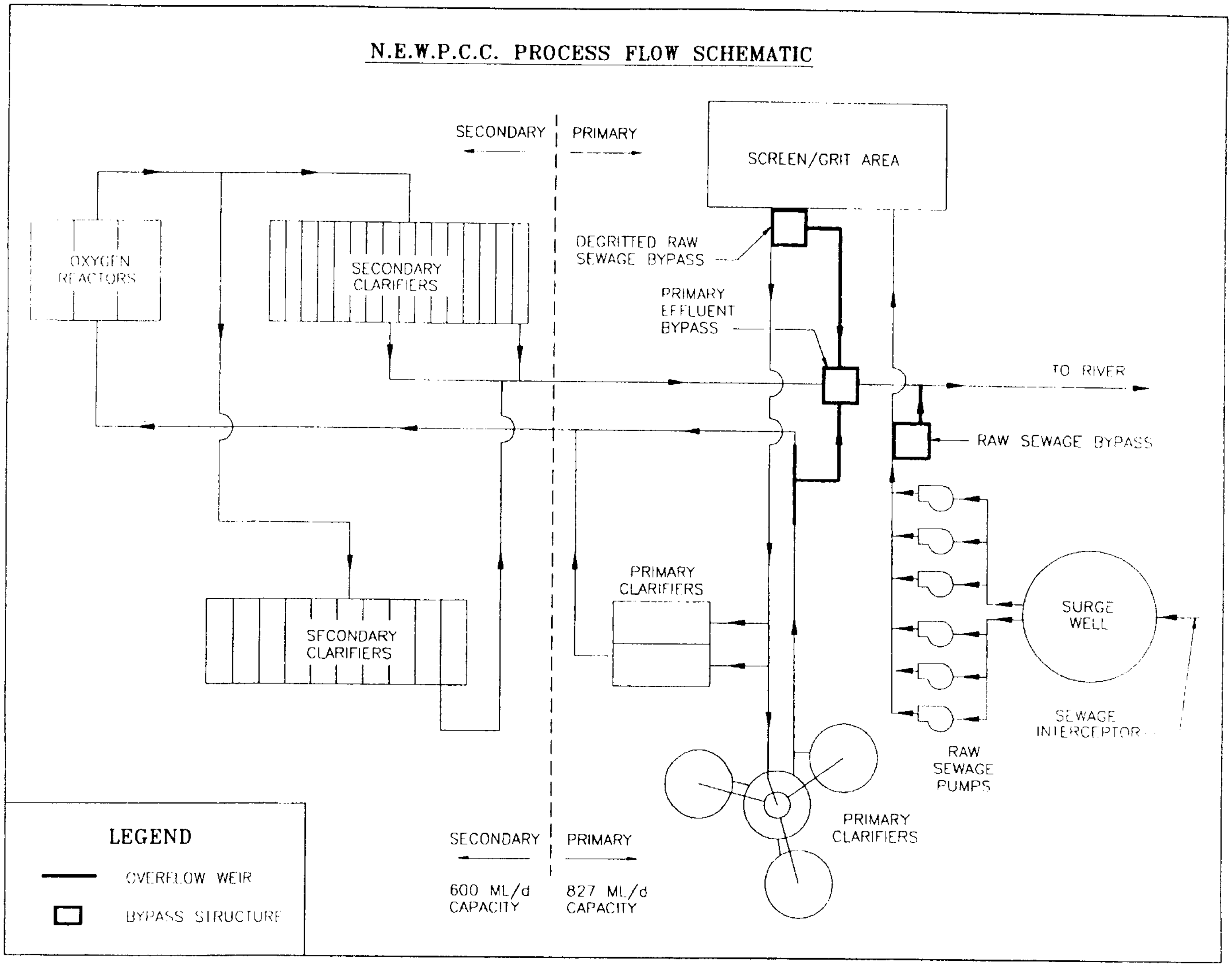
Key aspects which were brought out in the Phase 1 Workshop, and which relate to the treatment plants and the CSO study were:

- At present, no WPCC effluents are disinfected. These discharges result in coliform densities in the rivers which are greater than the Manitoba Surface Water Quality Objectives (MSWQO). Accordingly, any assessment of the benefits resulting from CSO control options on the rivers will include the effect of effluent (and possibly by-pass) disinfection.

- Flows to the plants should be maximized within the constraints of the existing infrastructure.

- Any significant reduction in CSO quantities, through diversion to the WPCC's, will likely dictate an increase in the capacity of the treatment plant to which the CSOs are tributary.

The following discussion reviews each of the WPCC's and their roles in overall WWF management, with the main focus on the NEWPCC.



NEWPCC Design Basis

• ADWF		= 300 ML/D
• AAF	= 1.11 x ADWF	= 332 ML/D
• PWWF	= 2.75 x ADWF	= 827 ML/D
• Primary Treatment Design Capacity	= PWWF	= 827 ML/D
• Secondary Treatment Design Capacity	= 1.8 x AAF	= 598 ML/D

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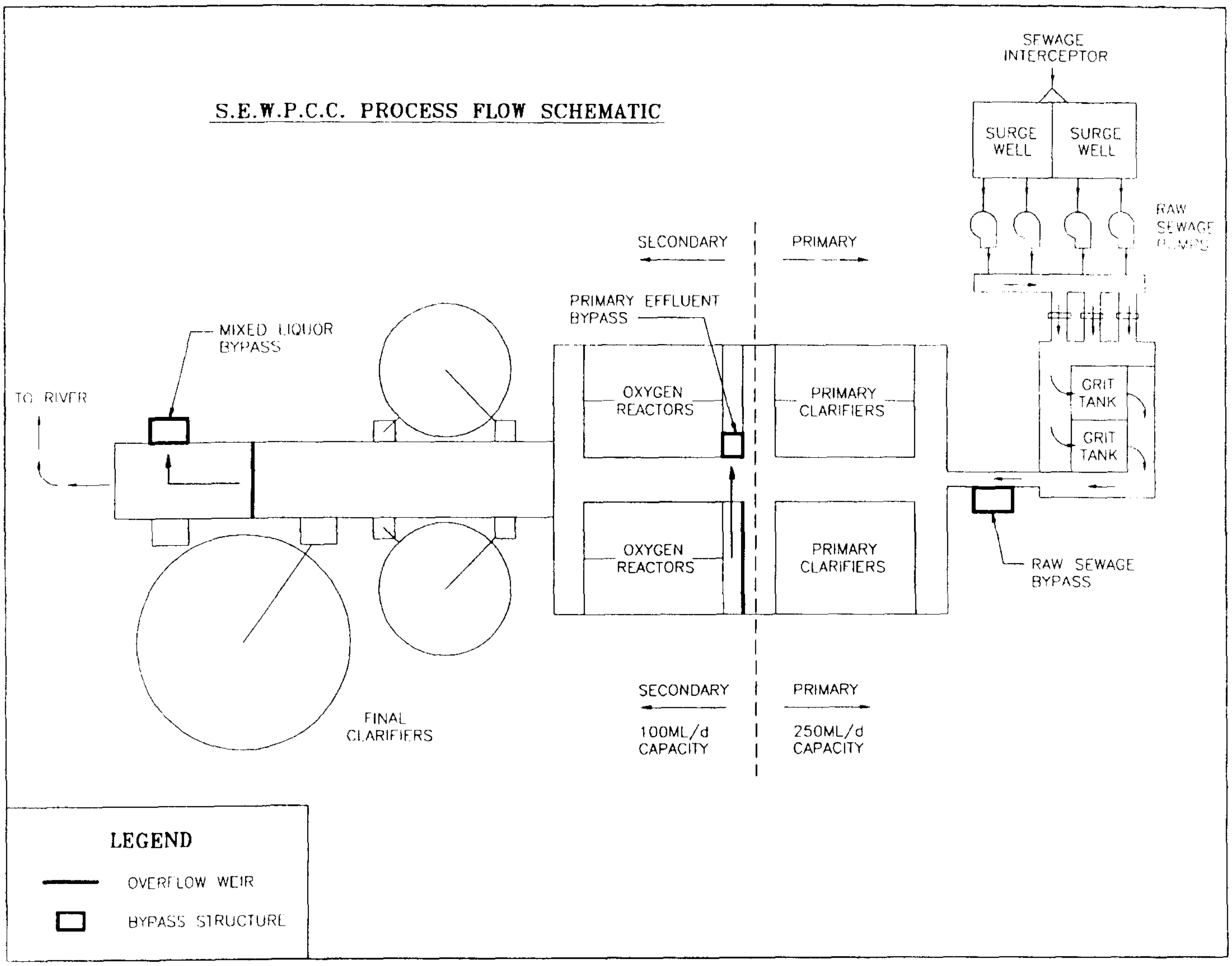
Figure 4-1

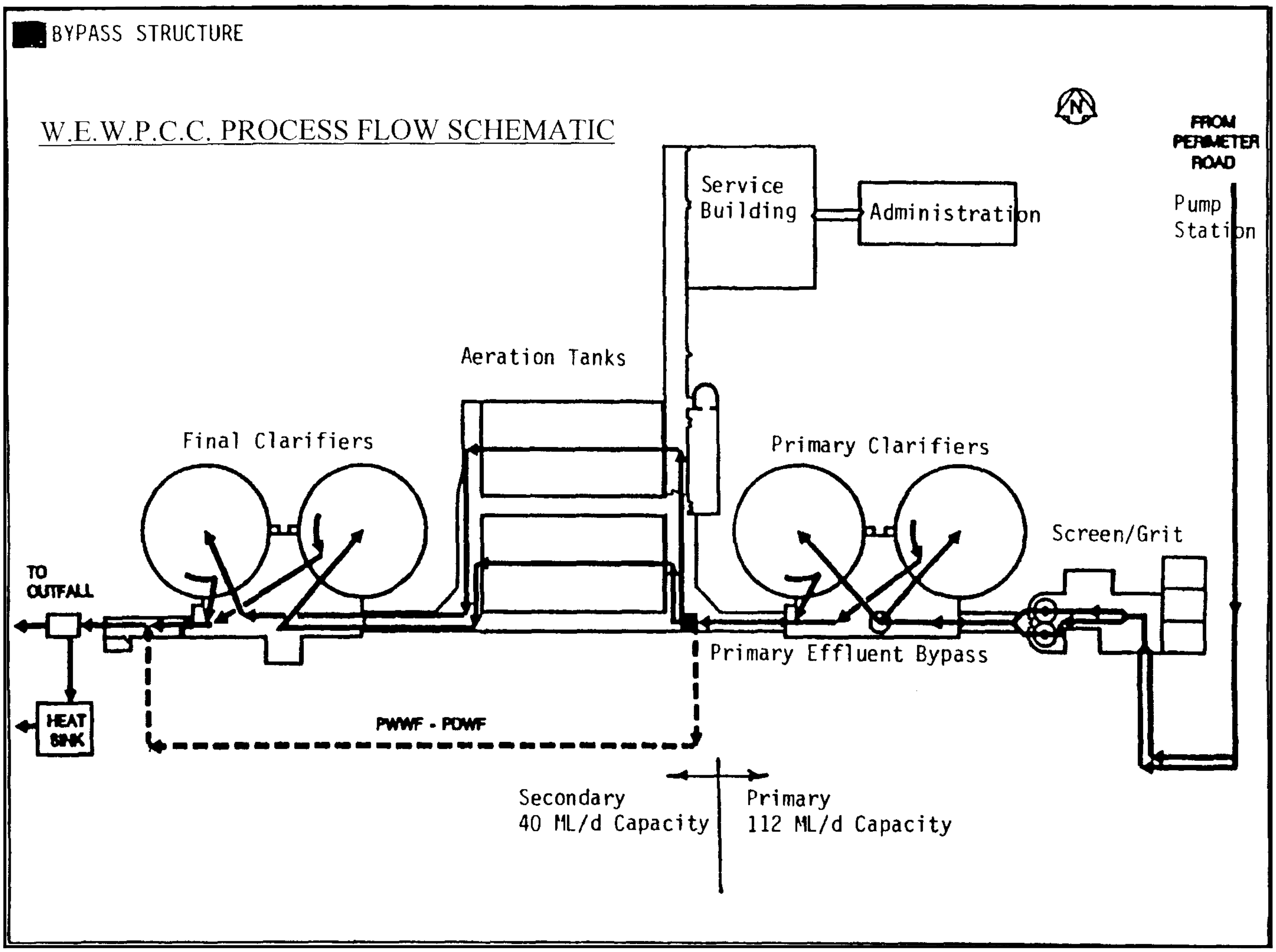


SEWPCC 1999 Design Basis*

• **ADWF**		**= 88 ML/D (59 ML/D)****
• **PDWF**	**= 1.7 x ADWF**	**= 150 ML/D (100 ML/D)**
• **PWWF**	**= 3.0 x ADWF**	**= 264 ML/D (176 ML/D)**
• **Primary Treatment Design Capacity**	**= PWWF**	**= 264 ML/D (176 ML/D)**
• **Secondary Treatment Design Capacity**	**= PDWF**	**= 150 ML/D (100 ML/D)**

* from SEWPCC Functional Design Report

** Stage 1 of 1999 Expansion Capacity Installed



WEWPCC Design Basis

• **ADWF**		**= 32 ML/D**
• **PDWF**	**= 1.7 x ADWF**	**= 54 ML/D**
• **PWWF**	**= 3.5 x ADWF**	**= 112 ML/D**
• **Primary Treatment Design Capacity**	**= PWWF**	**= 112 ML/D**
• **Secondary Treatment Design Capacity**	**= PDWF**	**= 54 ML/D** (currently restricted to 40 ML/D)

4.3 NEWPCC

4.3.1 Wet Weather Flow (WWF) Operation

Flows to the NEWPCC are conveyed by the Main Interceptor, servicing almost totally combined sewage areas, and the Northeast and Northwest Interceptors, serving separate sanitary sewer districts. On reaching the plant, all of the sewage is lifted by the raw sewage pumps. As discussed in Section 2.4.3 these comprise six pumps with a firm capacity of 870 ML/d, i.e., the capacity with one of the largest pumps out of service. Under current operations, flows are pumped up to the capacity of the six pumps. Flows in excess of treatment unit capacity overflow to the river(s) upstream of the treatment unit. Through a combination of fixed and variable capacity pumps, the firm pumping capacity is actually a range from 650 to 870 ML/d. The total installed pump capacity is 1060 ML/d.

Levels in the surge well are monitored on a continuous basis. These are currently being permanently recorded (at five minute intervals) along with total pumped flows, for purposes of interceptor calibration.

The pumped flows are conveyed directly to the coarse screens and aerated grit basins. These are designed to have a firm capacity of 827 ML/d, i.e., they are hydraulically able to accommodate the primary plant capacity with one of the four trains out of operation. Accordingly, the four trains have a total design capacity of 1100 ML/d, i.e., the total pumping capacity.

From the grit tanks, flows are conveyed to the primary clarifiers. These have a total design capacity of 827 ML/d. As indicated on Figure 4-1, discharges from the grit tanks in excess of the primary design capacity are by-passed over a weir to the plant outfall.

The primary clarifiers comprise five basins: two small and one large circular basins (outdoor) and two large rectangular basins (enclosed). The overflow rate at PWWF of 827 ML/d is 5 m/hr. During the summer months, the smaller basins are each removed from operation (consecutively) for about one month for preventive maintenance. With the large circular basin out of operation, the overflow role would be increased to 6.5 m/hr at PWWF.

On passing through the primary tanks, flows are conveyed to the secondary process, a high-purity oxygen activated sludge with enclosed secondary clarifiers. These have capacity to treat the PDWF of 600 ML/d. Flows from the primaries, in excess of PDWF, are diverted via an overflow weir to the plant outfall and thence to the river.

Effluent is conveyed to the river by the plant's outfall sewer. This 2.25 m diameter pipe is approximately limited to the current plant rated capacity (827 ML/d) at normal summer river levels. In fact, the capacity is limited by the hydraulic head available between the secondary clarifier weirs and the levels in the Red River at the outlet. For example, the capacity of the outfall, with high river levels at their six-year return frequency, limits the plant discharge to 770 ML/d (i.e., less than the plant rated capacity).

The NEWPCC is currently operating at an average dry weather flow (ADWF) rate of about 190 ML/d. Actual wet weather flow operating conditions are dictated by the quantity of flow intercepted in the combined sewer districts (Main Interceptor) and from the wet weather flow rates tributary to the Northeast and Northwest Interceptors (separate sewer districts). Combinations of potential flows from the tributary districts are shown in Table 4-1. The bases for the flows are, briefly:

- ADWF (in all interceptors) was calculated on the basis of 1.35 x water consumption in the tributary areas. This was the relationship between ADWF and water consumption at the NEWPCC for January 1993.

- 2.75 x DWF is the nominal design interception rate for the combined sewer districts (Main Interceptor). Incipient overflows, as discussed in Section 1.6 of this Technical Memorandum, represent a better estimate of the manner in which the system operates during small storms.

- 5 x DWF, for the flows tributary to the Main Interceptor, represents (approximately) the flows which could be conveyed down the interceptor without overflowing to the rivers. This was determined through the interceptor model analysis.

TABLE 4-1

CURRENT FLOW CONDITIONS (January 1993)
(based on 1.35*water consumption in tributary areas)

MAIN INTERCEPTOR ML/d				NORTHEAST/NORTHWEST ML/d			TOTALS AT PLANT ML/d				
	1	2	3		4	5	1+4	1+5	2+4	2+5	3+5
ADWF	2.75* DWF	INCIPIENT O/FLOW	5*DWF	ADWF	4*DWF	8*DWF	2.75*ADWF + 4*DWF	2.75*ADWF +8*DWF	INCIP.O/F + 4*DWF	I.O/F +8*DWF	5*DWF +8*DWF
153				36							
	420				145	290	565	710			
		600			145	290			745	890	
			770			290					1060

Notes:

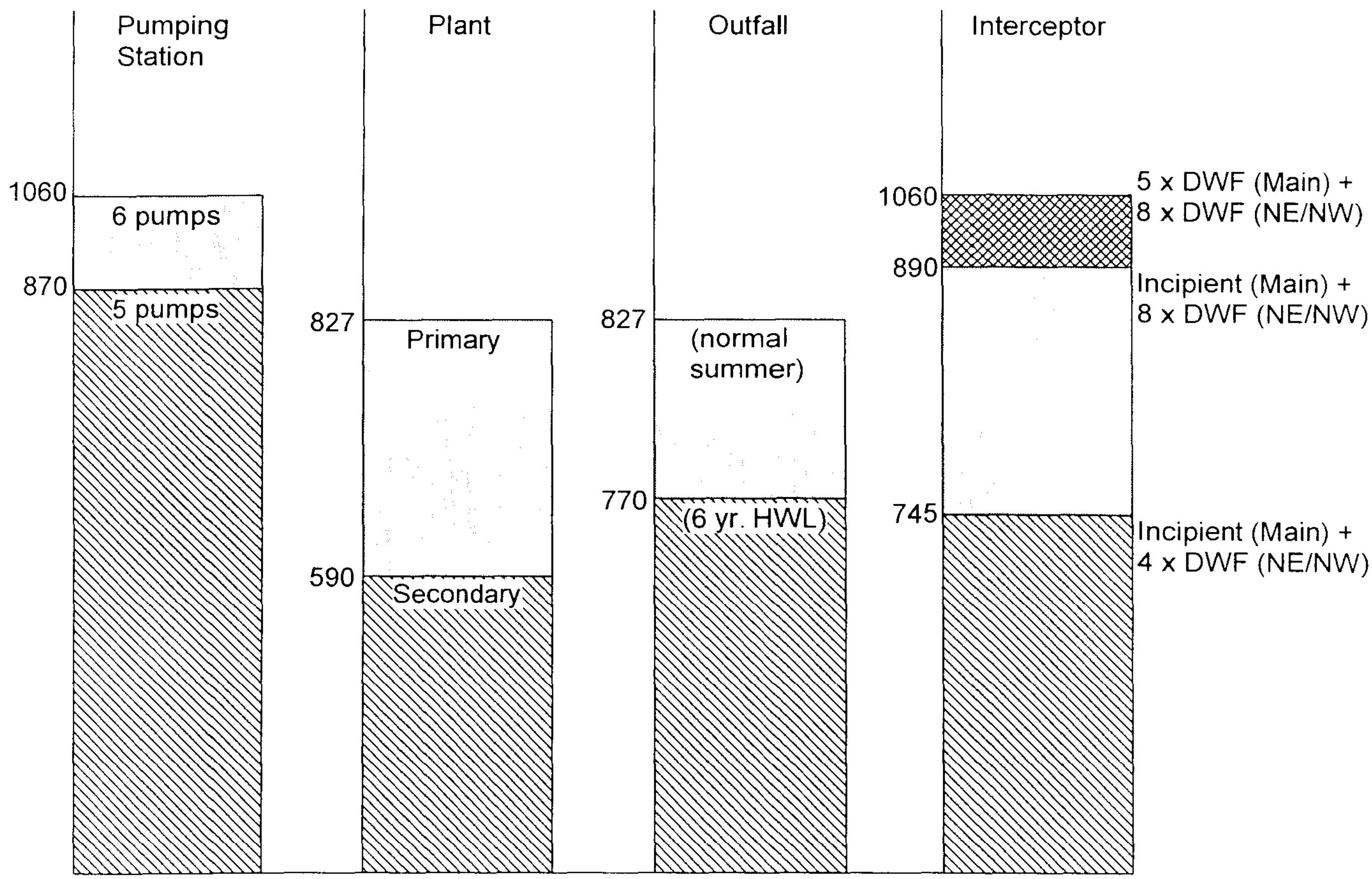
1. Current firm pumping capacity = 5 pumps with a range from 650 to 870 ML/d.
2. This information is presented graphically on Figure 4-4.

- 4 x DWF in the NE/NW Interceptors represents the design basis of these interceptors. For ultimate development of the areas tributary to these interceptors any excess flows was intended to overflow to the river.

- 8 x DWF was selected as being a high, but not an unrealistic, projection of the PWWF's tributary to the sanitary sewer collectors. Under current levels of development, the NE/NW have ample capacity to convey 8 x current ADWF to the NEWPCC. Whether or not the 8 x ADWF is appropriate is currently being investigated through flow monitors operating at the downstream ends of these two interceptors.

As can be seen from the table (and the graphical presentation of the results on Figure 4-4), and the above description of WWF operations, the NEWPCC has ample capacity to pump and provide primary treatment for total flows corresponding to incipient overflow (Main Interceptor) plus 4 x DWF from the NE/NW, 745 ML/d. That is, even though the combined sewer currently operates with PWWF in excess of 2.75 x DWF, the plant could accommodate the increase if the NE/NW operated under their design conditions for current development. There is, however, no control in the flows into the NE/NW Interceptor. Likewise, the NEWPCC can **almost** accommodate the WWF from incipient overflow (Main) plus 8 x DWF (NE/NW), i.e., 890 ML/d. With only five pumps running, the slight excess (20 ML/d) would overflow form the Main Interceptor, likely at St. John's overflow.

Analysis indicates (Section 2.7 of this TM) that the Main Interceptor could convey (with necessary modifications to CS diversions) 5 x ADWF without overflow to the rivers. As can be seen, this flow, in combination with the projected 8 x DWF from the NE/NW (1060 ML/d) would exceed the firm capacity of the main pumps, but could be lifted by the total installed pumpage (1060 ML/d). This fact is discussed later in this TM in association with control options.



NOTE - All volumes are in mega-litres per day (ML/d)

NEWPCC Comparative Flows

Figure 4-4

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4.3.2 Design Capacity/Constraints

From the above discussion, (Section 3.3.1), there are a number of constraints on the design capacity of the plants which would have to be addressed in order to accommodate any flows in excess of the NEWPCC design capacity (over 827 ML/d):

- Flow to the plants is limited by available pumping capacity - 870 ML/d firm, 1060 ML/d total.

- The screen/grit facilities could accommodate total pumping capacity, but only **with all four trains in operation**. Without all four available, the tanks would overflow.

- The primary sedimentation tanks are designed to accommodate 827 ML/d (PWWF) at an overflow rate of 5 m/hr - **with all five tanks in operation.** For a large part of the wet weather flow season, one of the three circular clarifiers is out of operation, which (if the flow is maintained at 827 ML/d) would raise the overflow rate to about 6.5 m/hr at PWWF. This is probably acceptable during PWWF.

- The hydraulics between the grit tank effluent channel and the primary tank outlet weirs is a concern, i.e., whether or not a flow increase (i.e. > 870 ML/d) can be accommodated with one clarifier out of operation, is questionable.

- The secondary plant operates at a PDWF of 600 ML/d. The impact of delivering more WWF would be to extend the period over which this flow would be sustained, i.e., as opposed to DWF diurnal variations or periods of storm runoff, the flows would continue beyond the storm duration until the inline or offline storage was drained to the plant.

- The current outfall capacity is more or less limited to current PWWF.

Any increase in PWWF would have to be accommodated within the above constraints (i.e., use **total** pump capacity) or would require structural modifications (eg., modify hydraulic capacity of primaries, if possible) or it would require installation of new facilities. In any event, it would likely require construction of a new outfall.

4.3.3 Expansion Considerations

One way to accommodate increased WWF to the NEWPCC is expansion of the front end of the plant (pumping, screening, grit removal and primary sedimentation). This section discusses the potential for such a plant expansion.

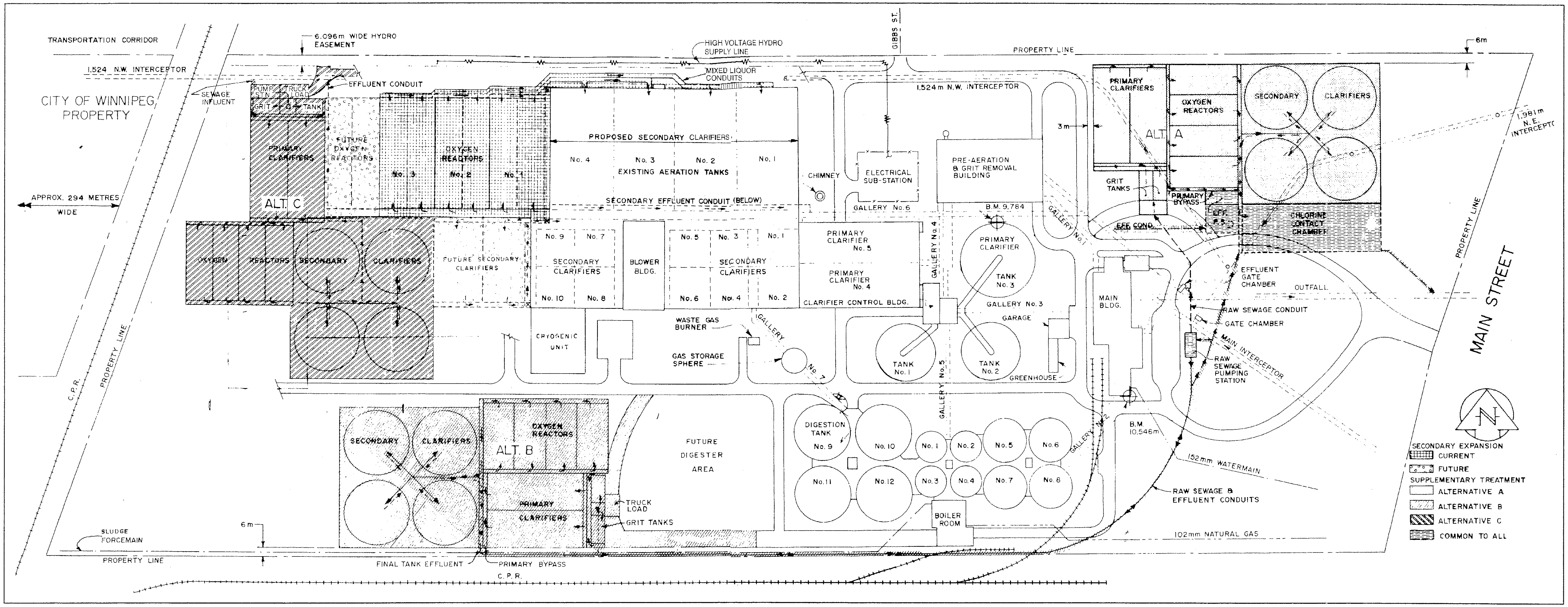
Expansion of the NEWPCC beyond its present capacity was considered as part of the "Functional Design Report on the NEWPCC Secondary Treatment Expansion" (Wardrop/MacLaren, 1981). The projected capacity increases upon which this expansion were based are:

	From (ML/d)	To (ML/d)
Primary Plant (raw sewage pumps, screens/grit tanks, primary clarifiers	827	1135
Secondary Plant (oxygen reactors, secondary clarifiers)	598	1000

The 1135 ML/d for the Primary Plant reflects the assigned capacity of the NE/NW Interceptors of 590 ML/d and the unsurcharged hydraulic capacity of the Main Interceptor of 531 ML/d. The actual future WWFs at the plant are uncertain. Most of this uncertainty relates to extraneous flows from the NE/NW separate sewerage system.

Figure 1-3 from the Wardrop/MacLaren report shows the location of three alternative sites for the expanded facilities on the existing NEWPCC property. Of the three, A and C were considered to be the preferred layouts for the supplementary treatment works.

The above expansions to the NEWPCC were intended to accommodate projected growth in tributary flows. The concerns of the current investigations are WWF. The latter would require additions to the primary plant facilities beyond the future expansion. On the basis of 5 times the current DWF + 8 times NE/NW current DWF, the additional WWF capacity needed now is 1060-827 or 230 ML/d, slightly less than double the projected future



Source: Wardrop/MacLaren, 1981

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Figure 1-3

expansion needs (1135-877 = 260 ML/d). By inspection of Figure 1-3, this could be best accommodated at Alternative A. Alternative C could also be adapted, but not as readily.

Sludge handling facilities would also require expansion. This would likely have the greatest impact on sludge digestion. There appears to be significant excess capacity in the sludge dewatering facility, which may be sufficient to accommodate the expansion contemplated.

As can be seen from the layouts, space has been allowed for disinfection. This was based on a chlorine contact chamber with a retention time of 30 minutes at ultimate average flow (555 ML/day). Preliminary indications are that the space allowed for this chlorine contact chamber would be sufficient for UV disinfection (MacLaren, 1986; Wardrop, 1992). It should be noted that the available head at the NEWPCC is limited. The addition of UV disinfection would necessitate pumping of the effluent, especially at higher river levels.

The hydraulics of the existing outfall to the river are such that it can convey 770 ML/day with the flood stage in the rivers at a one in six year return. The ultimate capacity for the projected supplemental treatment plant would require outflow of 1135 ML/day. This capacity could be achieved either through expanding the capacity of the gravity outfall (i.e., twinning) or effluent pumping. If UV is selected disinfection process, such effluent pumping could be accommodated at the same time as the installation of the disinfection facility. The effect of expanding the plant to accommodate the additional WWF (+230 ML/d) would be to increase the required outfall capacity, i.e. a larger pipe.

4.3.4 Maximizing WWF Treatment

If the flows from the Northeast and Northwest sanitary sewer districts in fact approximate the projected 8 x DWF from current development at the same time that intercepted combined sewer flows are raised to 5 x DWF, the total flows at the plant would be 1060 ML/day. The current installed capacity could pump these flows and it is expected that the screens and grit channels (with all trains in operation) could provide adequate treatment, so long as the screens are in good working order (i.e., the cleaning mechanisms can remove the screenings quickly enough).

Currently, flows in excess of the capacity of the primary tanks would overflow just downstream of the grit tanks and go directly to the outfall. It is possible that these flows could also be directed to the primary sedimentation basins and would receive a reduced level of treatment. This would be even further reduced with one basin out of service. In order to accommodate this increased flow, it would require changes to the diversion weir upstream of the primary tanks to increase the flow to the basins. The hydraulic capacity of the system, from the overflow weir downstream of the primary tanks through the tanks themselves to the feed channels, would have to be investigated. This would be part of Phase 3 investigation. Such a diversion would reduce the efficiency of solids removal in the tanks. This aspect will also be investigated in Phase 3. One constraint which might necessitate immediate construction (even if the rest of the system can be adapted to increased flows), is the capacity of the outfall from the plant. As noted, the current capacity is already limited by high river levels. In order to maintain the increased flows, a new parallel outfall would have to be constructed or effluent pumping would have to be installed. This would also be a matter for investigation in Phase 3 both from a risk and cost perspective.

If these increased WWFs were delivered to the NEWPCC, the secondary treatment facilities would be expected to operate at design capacity for much longer than normal periods of time. This is not expected to result in any operating difficulties.

The Northeast and Northwest Interceptors both include extensive reaches of oversized sewers. With appropriate control devices, these reaches could permit some degree of inline storage and hence, delay of peak flows at the plant. Further, the peaks from the sanitary sewer systems might well arrive at the plant later than the peaks from the combined sewer system. This would also delay the peak flows from the sanitary interceptors. The implications/benefits of such delays will have to be investigated in Phase 3 through the use of the interceptor model and the results of the current flow monitoring program which has been initiated on the Northeast/Northwest Interceptors.

The alternatives discussed above, if feasible, would allow the existing NEWPCC facilities to convey and treat the current projection of 5 x dry weather flow, plus 8 x dry weather flow from the two sanitary sewer interceptors. The condition discussed is based on 100% of the available equipment being in working order. Such a state carries with it risk that one or several of the units of operation are down for repair and/or maintenance. In this case, overflows would take place, probably at the St. John's overflow on the Main Interceptor.

The only way in which to ensure that firm capacity is available to treat the projected flows would be to install a new raw sewage pumping station in conjunction with a separate set of grit tanks and primary clarifiers. Such an installation would initially be for the purpose of combined sewer overflow treatment. In the long term, these facilities could become part of the NEWPCC expansion and some of the costs incurred could be assessed against that expansion, i.e., some of the new facilities could be used routinely for normal dry weather sewage treatment and would not have to be kept in total reserve for wet weather flows. It is projected that the cost of such an expansion (to accommodate WWF) would be in the order of $20 to $30 million, plus any costs associated with sludge treatment.

4.4 SEWPCC

4.4.1 <u>Wet Weather Flow Operation</u>

Only about 20% of the total drainage area tributary to the SEWPCC is served by combined sewers. Accordingly, WWF's to the south end plant are not nearly as influenced by combined sewage flows as are those at the NEWPCC. Notwithstanding this fact, flows up to 6 x DWF have been recorded at the SEWPCC (Wardrop/Tetr*ES* 1993). This PWWF is apparently dominated by inflow/infiltration (I/I) in the sanitary sewer districts. Flows in excess of this amount overflow to the river via the D'Arcy, St. Mary's Road and Killarney overflows.

All flows tributary to the SEWPCC are pumped. There are four raw sewage pumps with a firm capacity of 250 ML/d and a total installed capacity of 364 ML/d. All pumped flows pass through the screens and grit tanks, and flows in excess of the primary clarifier capacity are bypassed over a weir to the plant outfall. The current design PWWF capacity for the screens is 370 ML/d (the projected PWWF capacity for the year 2039). The current design PWWF capacity of the grit tanks and primary clarifiers is 175 ML/d.

The design overflow rate of the primaries at PWWF is 3.5 m/hr. Since all the clarifiers are covered, they are maintained in the winter months and, hence, are normally in operation over the wet weather flow period, May to October.

The secondary plant has a current design capacity of 100 ML/d (PDWF). This is really not a factor in considering WWF operations. The outfall sewer is capable of conveying the project plant capacity of the year 2039.

4.4.2 Expansion Considerations

Since current (and future) tributary flows to the SEWPCC are dominated by sanitary sewage flows, the implications, on required plant capacity, of an increase in combined sewage interceptor rates from 2.75 times to 5 times DWF, are not as serious as is the case at the NEWPCC. These implications, along with related effects of peak sanitary sewage flows, will be a part of the Phase 3 investigations.

4.5 WEWPCC

4.5.1 Wet Weather Flow Operation

Less than 10% of the total drainage area tributary to the WEWPCC is served by combined sewers. Accordingly, tributary WWF's to the plant are only modestly influenced by combined sewage flows.

All flows tributary to the plant are pumped by the Perimeter Road Pumping Station, located just east of the plant. Overflows to the interceptor system are located at the Community Row Pumping Station and Dieppe Road and Parkdale Street sewers.

The current total capacity of the Perimeter Road Pumping Station is 112 ML/d, with a firm capacity of 91 ML/d. The capacities of the bar screens, grit chambers and primary clarifiers are 172 ML/d (two of each at 86 ML/d capacity). Notwithstanding the latter capacities, the current rated capacity of the plant is 112 ML/d for PWWF. As with the other two plants, the surplus of (PWWF minus PDWF) bypasses the secondary plant and discharges to the outfall.

As with the SEWPCC, tributary flows are dominated by sanitary sewage flows. The implications, therefore, of an increase in combined sewer interceptor rates (2.75 times to 5 times DWF) will be very small. Peak sanitary sewage (WWF) flows will have a larger impact and should be investigated in Phase 3.

At the time of writing, the effluent from the mechanical plant discharges to the existing lagoon system (total volume 1600 ML, approximately). If this practice is continued, it will result in large reductions in fecal coliforms in the final discharge to the Assiniboine River. The significance is currently being monitored. It would appear that the quality will meet MSWQO.

4.6 EMCs (WPCC)

The concentration of fecal coliforms in treated effluents from the NEWPCC, SEWPCC, and WEWPCC used initially in Phase 1 analysis were based on values report in the 1986 Disinfection Report (MacLaren 1986). Data on actual concentrations of fecal coliforms in treated effluent are sparse.

A recent report, the UV Disinfection Study (Wardrop 1992) indicated that the fecal coliform concentrations in the final effluent of all three plants were substantially lower than previously measured. A cursory analysis was performed, using the full history of bi-weekly monitored data upstream (Redwood Bridge) and downstream (North Perimeter Bridge) of the NEWPCC outfall, to estimate the concentration of fecal coliforms in NEWPCC treated effluent discharge. It was found that the long-term geometric mean from the NEWPCC was in the order of 2×10^5 organisms per 100 mL. This analysis is discussed in TM #3. This is about half of the value previously used in the 1986 Disinfection Report (MacLaren 1986). Details of this analysis are provided in Appendix A. Long term EMCs for SEWPCC effluent remain at 2×10^5 per 100 mL. The WEWPCC data is complicated by the fact that, until recently, the treated wastewater spent a considerable period of time in lagoons as part of its treatment and currently passes through the same lagoons for polishing. In both cases, the final effluent fecal concentrations are low. For modelling purposes, 2×10^5 organisms per 100 mL has been used for the effluent from the new mechanical plant.

It was recommended that the City conduct regular monitoring of the final effluents from the 3 WPCCs on a weekly basis, along with secondary bypass quality, to develop a database to characterize the density of fecal coliforms in the discharges. This activity is currently underway.

The EMCs, as adjusted in the river water quality assessment, are discussed in detail in TM #3.

4.7 EFFLUENT DISINFECTION

4.7.1 General

The City of Winnipeg is currently planning to implement a program of disinfection of the wastewater treatment plant effluents. This will result in a significant reduction in fecal coliform concentrations in the rivers under dry weather conditions.

The issue of plant effluent disinfection was reviewed in an extensive study (MacLaren 1986) which also reviewed the available technology for accomplishing disinfection. That study indicated that chlorination - dechlorination could be effective disinfection for all three plants and UV disinfection was potentially applicable to the SEWPCC and the WEWPCC. Pilot tests (Wardrop, 1992 UV-Disinfection Report) indicated that the SEWPCC plant effluent is amenable to UV disinfection with conventional low intensity lamps. The NEWPCC effluent was less amenable to cost-effective disinfection with conventional UV technology. UV disinfection will likely be the technology of choice, since it avoids the complications associated with chlorination, both from the perspective of a potential for fish toxicity, the avoidance of the production of THMs, and handling of a hazardous material.

UV disinfection of the NEWPCC effluent was not as successful because of the low light transmissibility of the plant flows, even under dry weather conditions. This situation was compounded under WWF conditions.

The WEWPCC has recently gone on stream as a completely upgraded mechanical (activated sludge) secondary treatment plant. The plant had previously operated as a small mechanical plant supplemented by a lagoon system. The new plant is currently being operated with the effluent continuing to pass through the lagoons for polishing purposes to utilize the lagoons as potential wetlands. So far, this extended storage period has resulted in a substantial reduction of coliform concentrations discharged to the river. It may be that, so long as the lagoons are maintained in the process stream, disinfection will not be needed in any form in order to meet the MSWQO.

4.7.2 WWF Disinfection

An element of uncertainty is whether or not disinfection facilities should include provision to disinfect wet weather flows as well as DWF. The river model developed for this CSO study could be used to evaluate the additional benefits which would be achieved through disinfection of WWF, as well as DWFs, and place these additional benefits in perspective with respect to residual CSOs and LDS overflows to the river.

If the current effluent fecal coliform concentration (EMC) at the NEWPCC is 2×10^5 per 100 ml, DWF disinfection could be expected to effect 3 log reduction, i.e., an EMC of 200 per 100 ml. The tests with UV disinfection indicate that, with WWF passing through the disinfection facility (sized for DWF), the reduction could be 1 log, i.e., an EMC of 2×10^4 per 100 ml. UV technology is evolving rapidly and this predicted performance will be reassessed in Phase 3.

4.8 MONITORING NEEDS

The City of Winnipeg has been monitoring effluent quality (fecal coliform) at each of the three WPCC in the city. Collection of this information began in May, 1995. Samples are being collected once a week at the NEWPCC and SEWPCC. Samples at the WEWPCC are collected three times a week. The City has been asked to sample by-pass flows (i.e., primary sedimentation effluent) in wet weather. This will be done at all three plants, where and when practicable. This will be useful in confirming fecal EMCs for plant effluents and bypass.

The City of Winnipeg commenced monitoring main pumping flows and associated surge well levels in June 1994 (on an hourly basis.) Problems, subsequently identified in the system, indicated that the data collected up to March 1995 were unreliable for wet weather flow. Accordingly, this data will only be used for confirming 1995 dry weather flow operating conditions. When data have been collected on a broad range of flows, the information will be used as part of the calibration of the NEWPCC interceptor model.

As discussed above, it will be of interest to know the delay between the arrival of the combined sewer peaks at the NEWPCC surge well and the Northeast/Northwest Interceptor peaks. The flow and level monitoring in the surge well and in the interceptors will be helpful in this regard. A necessary part of this analysis will be the nature and direction of the storm travel across the City. That is, the delay in peaks will not only be a factor of in-flow/infiltration at the time, but also the lag between storm peaks and durations.

The data obtained through the current monitoring program will be useful in the Phase 3 analysis.

4.9 SUMMARY OF PHASE 3 INVESTIGATIONS FOR TREATMENT

1. Investigate the capability of the NEWPCC to accommodate increased flows due to an increase in WWF interception in the CS district. This investigation would apply to pumping, screening and grit removal, primary sedimentation, outfall sewer and solids handling (sludge digestion).

2. Investigate implications of increased CSO interception rates to 5 times DWF on the SEWPCC and WEWPCc. This should include a review of the related effects of peak sanitary sewage flows on the plant capacities.

3. Investigate capacity of NE/NW Interceptor systems to store PWWFs with a particular emphasis on delaying peak flows to the NEWPCC (using results of current monitoring program).

REFERENCES

City of Winnipeg, Waterworks, Waste and Disposal Department. 1965. Polson Comminutor Station - Design Notes, 1965.

James F. MacLaren Associates. 1961. Report on Expansion of Sewage Works in the North District of Metropolitan Winnipeg. August 1961.

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Wardrop Engineering Inc. 1992. Wastewater Ultraviolet Disinfection Study - Phase II, June 1992.

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Wardrop Engineering Inc. and Tetr*ES* Consultants Inc. 1993. SEWPCC Expansion, Stage 1 Evaluation Study (Draft). December 1993.

Wardrop Engineering Inc. and Tetres Consultants Inc. 1995. Combined Sewer Overflow Management Study, Phase 2 - Priority Field Inspection Report, District Diversion Structures and NEWPCC Interceptor Sewers - Draft. May 1995.

W.L. Wardrop & Associates. 1979. Separate Sewer Relief Study, 1978 Program. June 1979.

APPENDIX A

APPENDIX A

The information contained in this Appendix is taken from the Phase 2 Priority Field Inspection Report, District Diversions Structures and NEWPCC Interceptor Sewers, Wardrop/Tetres, May 1995, Draft. The field investigations were conducted to gain a better understanding of the existing combined sewer system. The investigations were designed to:

- assist in determining the operation of the diversion structures;
- allow review of the C.S. system interactions with the interceptors;
- provide data for the Interceptor modelling; and
- provide a preliminary estimate of the structural condition of the combined trunk sewers (at the weir diversions) and interceptor sewers tributary to the NEWPCC.

Excerpts from the report include the following:

- executive summary,
- Table A-1, noting CSO District type (i.e., pumped or gravity connection to the interceptor), trunk size, weir height and type, and noting the pressure of dry weather overflows and leaking flat gates. The numbering convention used for the districts is the same as used throughout the body of the report.
- Table A-2 contains a summary of the structural condition of the district trunk sewers, as noted near the diversion weirs.
- Table C-1 contains summary information obtained during spot inspections of the Main Interceptor sewer, including pipe size and type, depth of sewer and noted hydraulic and structural conditions. The locations of the visual spot inspections are shown on Figures C-1 and C-2.
- Tables D-1 and E-1 provide summary information from the spot inspections of the Northwest and Northeast interceptors, respectively. The locations are shown on Figures D-1 and E-1.

EXECUTIVE SUMMARY

The priority field inspections were conducted to gain a better understanding of the existing combined sewer system in the City of Winnipeg. These inspections were undertaken as a part of the City of Winnipeg's Combined Sewer Overflow Management Study. They formed a part of Phase 2 of the study and were designed to:

- assist in determining the operation of the diversion structures;
- allow review of the system interactions; and
- provide data for the Interceptor model construction and calibration.

The detailed hydraulic analysis of the diversion facilities will be included with the Phase 2 Technical Memorandum.

The scope of the field inspections included a detailed examination of the diversion facilities of all 42 combined sewer districts in the City of Winnipeg, and spot inspections of the Main, Northwest and Northeast Interceptor Sewers (tributary to the North End Water Pollution Control Centre.)

This report covers the structural conditions noted during the inspections, and general observations concerning the hydraulic operation of the combined sewer district diversion structures. The inspection focussed on the existing weir diversions, and comminutor, regulator, and other structures that influence the hydraulic characteristics. A detailed description of the physical characteristics of each district's diversions is given. Pertinent information, such as weir heights, pipe sizes, gate sizes and operation, and the presence and operation of comminutors and hydraulic regulator valves, is included. Problematic areas such as leaking flap gates and dry weather overflows are noted.

The structural condition of the district trunk sewers were evaluated. Ten of the forty-two trunk sewers are in fair to poor or poor condition, and more detailed inspection or monitoring is recommended.

Spot inspections of the Main, Northwest, and Northeast Interceptor Sewers indicate that they are in generally good structural condition. One isolated location of severe exposed aggregate was detected in the Main Interceptor Sewer at the intersection with the Sutherland Secondary Sewer. Significant sludge buildup was observed in the Northwest Interceptor.

The Appendices contain inspection summaries with collected and tabulated data, detailed district information, schematics, and photographs taken during the inspections.

TABLE A-1
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
HYDRAULIC OPERATION SUMMARY

No.	District	District Type	Trunk Size	Weir Height	Weir Type	Dry Weather Overflow	Leaking Flap Gate	Comments
1	Alexander	gravity	1270	480	C	--	--	
2	Armstrong	gravity	2750	330	C	--	--	1220 cross connection to Newton trunk
3	Ash	pumped	3250 x 2520	690	C	--	--	
4	Assiniboine	gravity	2-1120	n/a	SP	--	--	overflow chamber present
5	Aubrey	pumped	2850 x 2190	790	C	--	--	
6	Baltimore	pumped	1830 x 1420	430	C	--	--	
7	Bannatyne	gravity	1520	n/a	C	--	--	
8	Boyle	pumped	920	200	C	--	--	
9	Clifton	pumped	2920 x 2310	840	W	--	--	
10	Cockburn	pumped	2700 x 2080	n/a	C	yes	--	discharges to Baltimore
11	Colony	gravity	1830 x 1420	840	C	--	--	
12	Cornish	pumped	1520	510	C	--	--	
13	Despins	pumped	1370	530	C	--	yes	
14	Doncaster	gravity	2290	200	C	--	--	discharges to Ash
15	Douglas Park	gravity	310	330	A	--	--	discharges to Ferry Road
16	Dumoulin	pumped	1070	230	C	--	--	
17	Ferry Road	pumped	3050 x 1980	360	C	--	yes	
18	Hart	pumped	2840 x 2180	330	C	--	yes	
19	Hawthorne	pumped	1830	410	W	--	--	discharges to Newton
20	Jefferson	gravity	4270 x 2840	460	C	--	yes	overflow chamber present
21	Jessie	pumped	2470 x 1910	710	C	--	--	
22	Laverendrye	gravity	810	690	W	--	--	discharges to Dumoulin
23	Linden	pumped	3430 x 2290	280	C	--	--	discharges to Newton
24	Mager Drive	pumped	3430 x 2290	990	C	--	--	
25	Marion	pumped	1680	530	C	--	--	

TABLE A-1
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
HYDRAULIC OPERATION SUMMARY

No.	District	District Type	Trunk Size	Weir Height	Weir Type	Dry Weather Overflow	Leaking Flap Gate	Comments
26	Metcalfe	pumped	1630 x 1070	310	C	--	--	
27	Mission	gravity	2970 x 1980	940	C	--	yes	discharges to Montcalm pumping station
28	Moorgate (Conway)	pumped	2550 x 1950	690	C	--	--	
29	Munroe	gravity	3200 x 2130	510	C	--	--	discharges to Polson
30	Newton	gravity	1830	200	C	--	--	1220 cross connection to Armstrong trunk
31	Parkside	gravity	610 and 760	150	C	--	--	discharges to Riverbend
32	Polson	gravity	2210 x 1780	690	C	--	--	
33	River	pumped	1500 x 1050	610	C	--	yes	
34	Riverbend	pumped	2290	150	C	--	--	
35	Roland	gravity	2110 x 1630	430	C	--	--	discharges to Montcalm pumping station
36	Selkirk	gravity	2030 x 1630	560	C	--	--	
37	Strathmillan	gravity	920	250	C	--	--	
38	St. John's	gravity	2030 x 1630	360	C	--	--	overflow chamber present
39	Syndicate	pumped	1070	250	C	--	--	
40	Tuxedo (Chattaway)	pumped	2290 x 1520	130	C	--	--	discharges to Doncaster
41	Tylehurst	pumped	2690 x 2080	360	C	yes	--	
42	Woodhaven	pumped	1220 x 940	150	W	--	--	

Weir Type Legend:
- C Concrete
- W Wood
- A Aluminum
- SP Special

Note: All dimensions in millimetres

TABLE A-2
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
STRUCTURAL CONDITION SUMMARY

No.	District	Pipe Type	Overall Condition	Little or no Deter.	Rubble in Invert	Invert Erosion	Exposed Agg.	Some Spalls	Heavy Spalls	Minor Cracks	Major Cracks	Wall Move.	Brick Deter.	Prev. Repairs Failed
1	Alexander	brick	good	✓										
2	Armstrong	concrete	good	✓										
3	Ash	concrete	fair-poor				✓			✓				
4	Assiniboine	brick	good-fair										✓	
5	Aubrey	concrete	poor		✓	✓	✓		✓					
6	Baltimore	concrete	good	✓										
7	Bannatyne	brick	good	✓										
8	Boyle	concrete	good	✓										
9	Clifton	concrete	poor		✓	✓		✓			✓			
10	Cockburn	concrete	good	✓										
11	Colony	brick	good	✓										
12	Cornish	brick	fair										✓	
13	Despins	concrete	good	✓										
14	Doncaster	concrete	good	✓										
15	Douglas Park	concrete	good	✓										
16	Dumoulin	concrete	good	✓										
17	Ferry Road	concrete	fair/ poor		✓	✓		✓		✓				✓
18	Hart	concrete	fair					✓						
19	Hawthorne	concrete	good	✓										
20	Jefferson	concrete	good-fair/ poor							✓				
21	Jessie	concrete	good	✓										
22	Laverendrye	concrete	good	✓										
23	Linden	concrete	good-fair							✓				

TABLE A-2
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
STRUCTURAL CONDITION SUMMARY

No.	District	Pipe Type	Overall Condition	Little or no Deter.	Rubble in Invert	Invert Erosion	Exposed Agg.	Some Spalls	Heavy Spalls	Minor Cracks	Major Cracks	Wall Move.	Brick Deter.	Prev. Repairs Failed
24	Mager Drive	concrete	good-fair							✓				
25	Marion	concrete	good	✓										
26	Metcalfe	concrete	fair					✓						
27	Mission	concrete	poor		✓	✓	✓							
28	Moorgate (Conway)	concrete	fair		✓	✓	✓							
29	Munroe	concrete	poor						✓	✓				
30	Newton	concrete	good	✓										
31	Parkside	concrete	good	✓										
32	Polson	concrete	poor		✓	✓			✓					
33	River	brick	good	✓										
34	Riverbend	concrete	good	✓										
35	Roland	concrete	fair-poor				✓			✓				
36	Selkirk	brick	fair										✓	
37	Strathmillan	concrete	good	✓										
38	St. John's	brick	good	✓										
39	Syndicate	brick	good	✓										
40	Tuxedo (Chattaway)	concrete	fair-poor		✓	✓				✓				
41	Tylehurst	concrete	fair		✓	✓				✓				
42	Woodhaven	concrete	fair							✓				

TABLE C-1
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
MAIN INTERCEPTOR INSPECTION SUMMARY

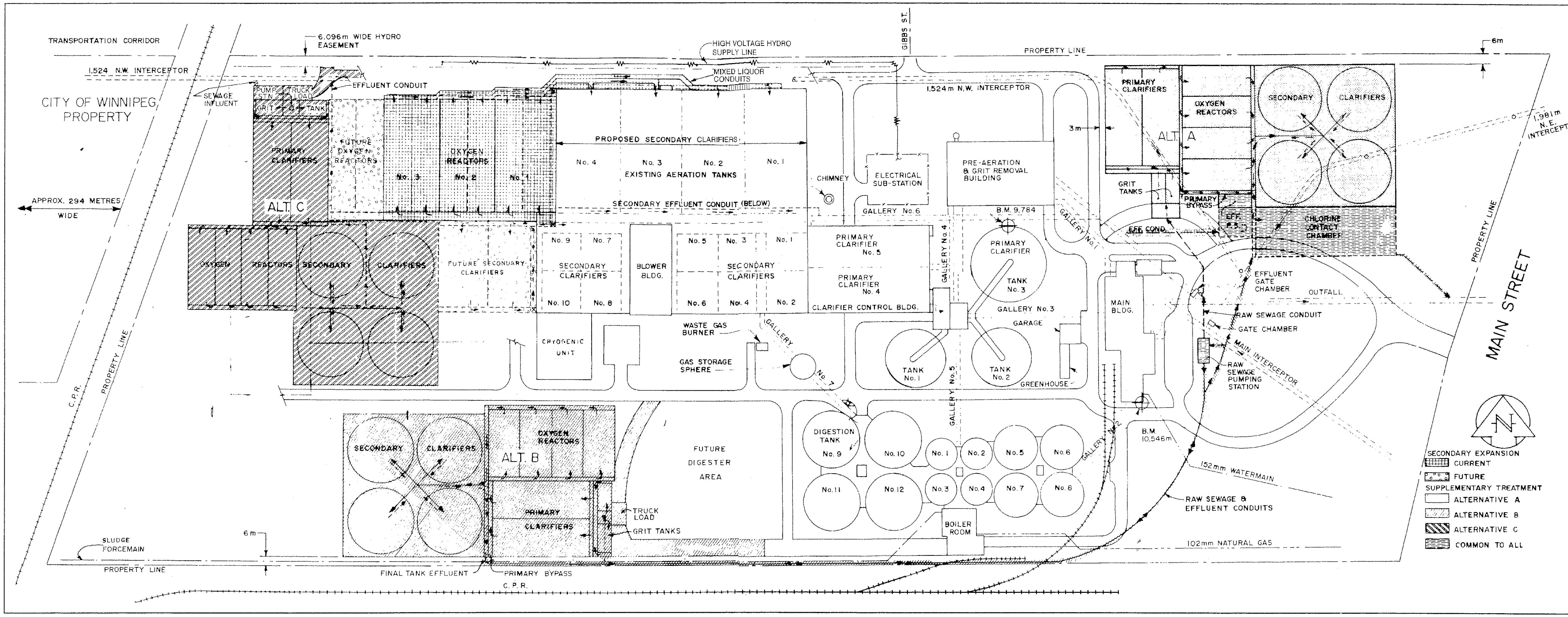
MH	Location	Date	Rim to Invert Depth	Water Depth	Water Vel.	Sed.	Pipe Size	Pipe Type	Struct. Cond.	Comments
2	Main Street, north of Seaforth Avenue	Aug 4	14960	2970	calm	none	2290	concrete	n/a	sewage level is 690 above obvert of pipe
2*	Main Street, north of Seaforth Avenue	Nov 29	14880	1250	slow	none	2290	concrete	good	
3*	Main Street, north of Templeton Avenue	Nov 29	14760	1270	slow	none	2290	concrete	good	
4	Main Street, north of Leila Avenue	Aug 4	13690	2210	swirl-ing	none	2290	concrete	n/a	sewage level is 75 from obvert of pipe
6	Main Street, south of Hartford Avenue	Aug 4	12600	1450	fast	none	2290	concrete	good	some exposed aggregate
7*	Main Street, north of Enniskillen Avenue	Nov 29	13210	710	very fast	none	2290	concrete	good	
8	Main Street, north of Carruthers Avenue	Aug 5	12930	915	very fast	none	2290	concrete	good	
8*	Main Street, north of Carruthers Avenue	Nov 29	12850	585	very fast	none	2290	concrete	good	
10	Main Street, north of Church Avenue	Aug 5	11760	865	very fast	none	2290	concrete	good-fair	some exposed aggregate
10*	Main Street, north of Church Avenue	Nov 29	11760	660	fast	none	2290	concrete	good-fair	slight exposed aggregate
12*	Main Street, south of Alfred Avenue	Nov 29	11530	535	fast	none	2290	concrete	fair	old patches present, exposed aggregate
13	Main Street, north of Selkirk Avenue	Aug 4	11530	915	very fast	none	2290	concrete	good-fair	some exposed aggregate

TABLE C-1
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
MAIN INTERCEPTOR INSPECTION SUMMARY

MH	Location	Date	Rim to Invert Depth	Water Depth	Water Vel.	Sed.	Pipe Size	Pipe Type	Struct. Cond.	Comments
15**	Main Street, at Sutherland Avenue	Aug 4	n/a	n/a	turbu-lent	none	2290	concrete	fair	severe exposed aggregate
16	Main Street, south of Higgins Avenue	Aug 3	10640	840	very fast	none	1980	concrete	good	
18	Main Street, north of Portage Avenue	Aug 3	9960	890	very fast	none	1980	concrete	good	
19	Main Street, south of Graham Avenue	Aug 3	9525	815	very fast	none	1980	concrete	good	
21	Main Street, south of Broadway	Aug 3	9070	485	fast	none	1120	concrete	good	
23	Main Street, at River Avenue	Aug 3	3400	430	fast	none	1120	concrete	good	
24	Broadway, east of Donald Street	Aug 3	9020	760	very fast	none	1680	concrete	good	
27	Wolseley Avenue, east of Lenore Street	Aug 3	5410	760	fast	none	1070	concrete	good	
30	Wolseley Avenue, west of Clifton Street	Aug 3	3380	585	fast	none	1070	concrete	good	
35	Portage Avenue, at Ragland Road	Aug 3	5510	535	fast	none	1220	concrete	good	
57	Sutherland Avenue east of Austin Street	Aug 4	6600	430	fast	none	1120	concrete	fair	severe exposed aggregate
58	Newton Avenue, east of Main Street	Aug 5	9730	230	fast	none	1370	concrete	good	

*Inspection taken place during Main Interceptor draw-down on November 29, 1994
**Manhole at junction of Main Interceptor and Sutherland Secondary

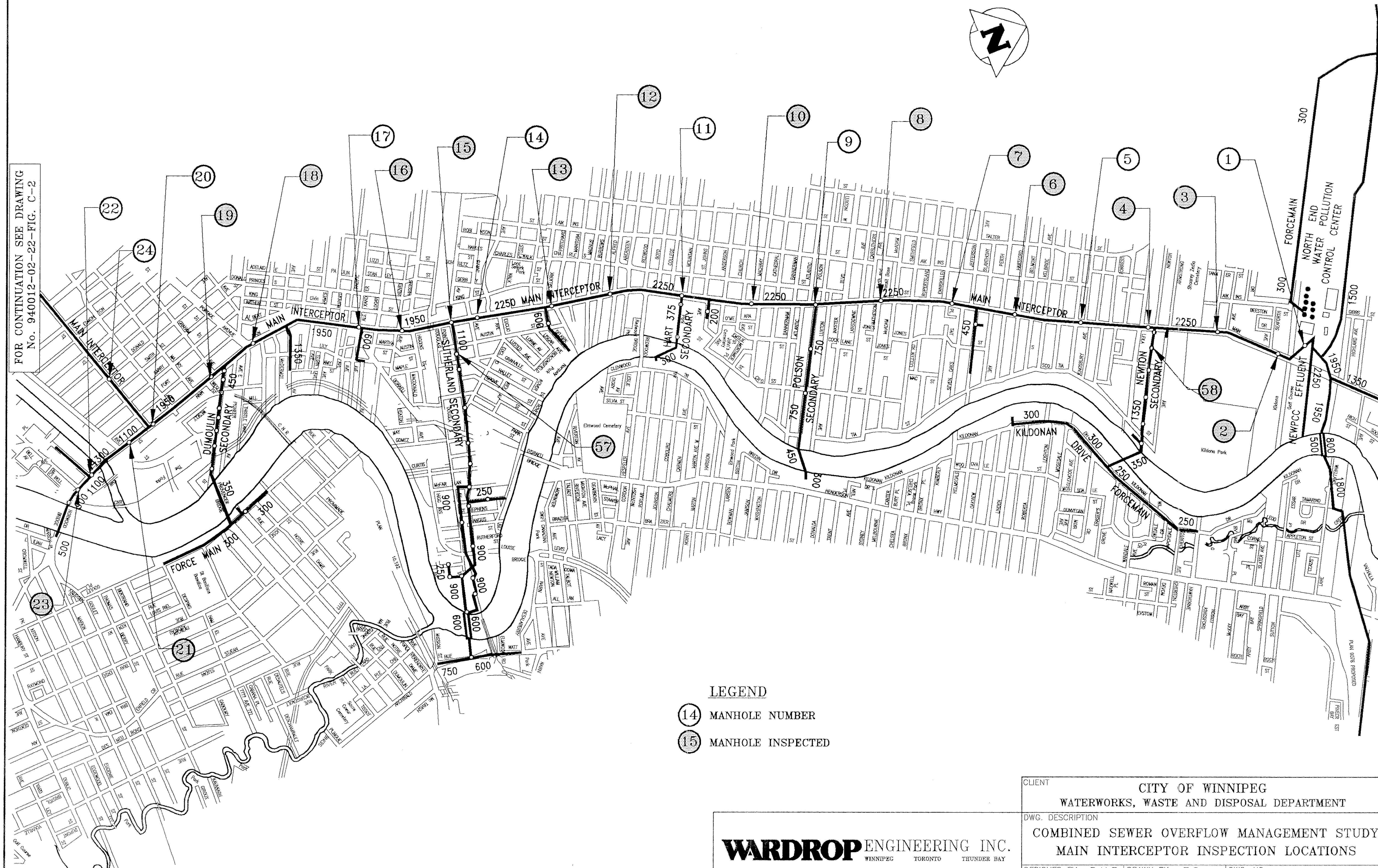
Note: All dimensions are in millimetres



Source: Wardrop/MacLaren, 1981

plantpln
01_files

Figure 1-3



LEGEND

(14) MANHOLE NUMBER

(15) MANHOLE INSPECTED

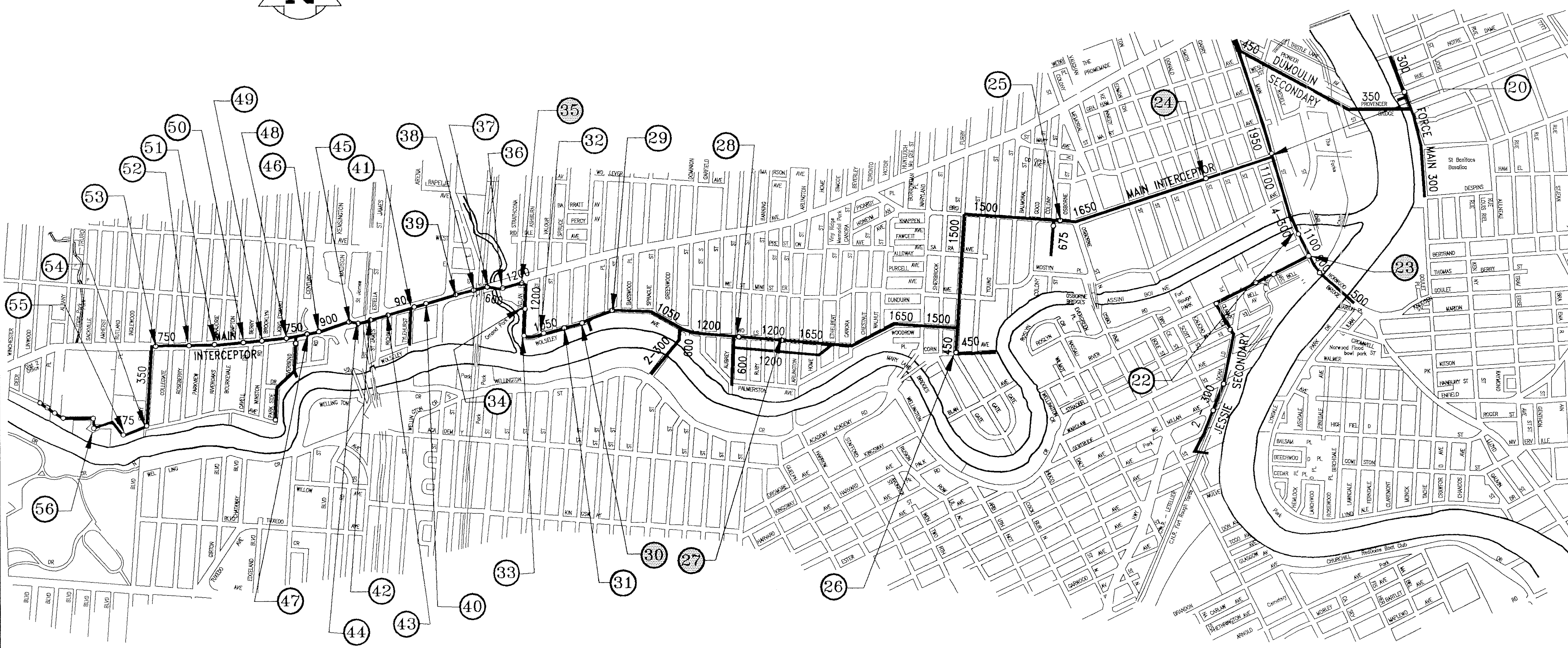
WARDROP ENGINEERING INC.
WINNIPEG TORONTO THUNDER BAY

CLIENT
CITY OF WINNIPEG
WATERWORKS, WASTE AND DISPOSAL DEPARTMENT

DWG. DESCRIPTION
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
MAIN INTERCEPTOR INSPECTION LOCATIONS

DESIGNED BY: D.N.B.	DRAWN BY: T.G.	DWG. NO.

FOR CONTINUATION SEE DRAWING
No. 940012-02-22-FIG. C-1



LEGEND

44 MANHOLE NUMBER

35 MANHOLE INSPECTED

WARDROP ENGINEERING INC.
WINNIPEG TORONTO THUNDER BAY

CLIENT
CITY OF WINNIPEG
WATERWORKS, WASTE AND DISPOSAL DEPARTMENT

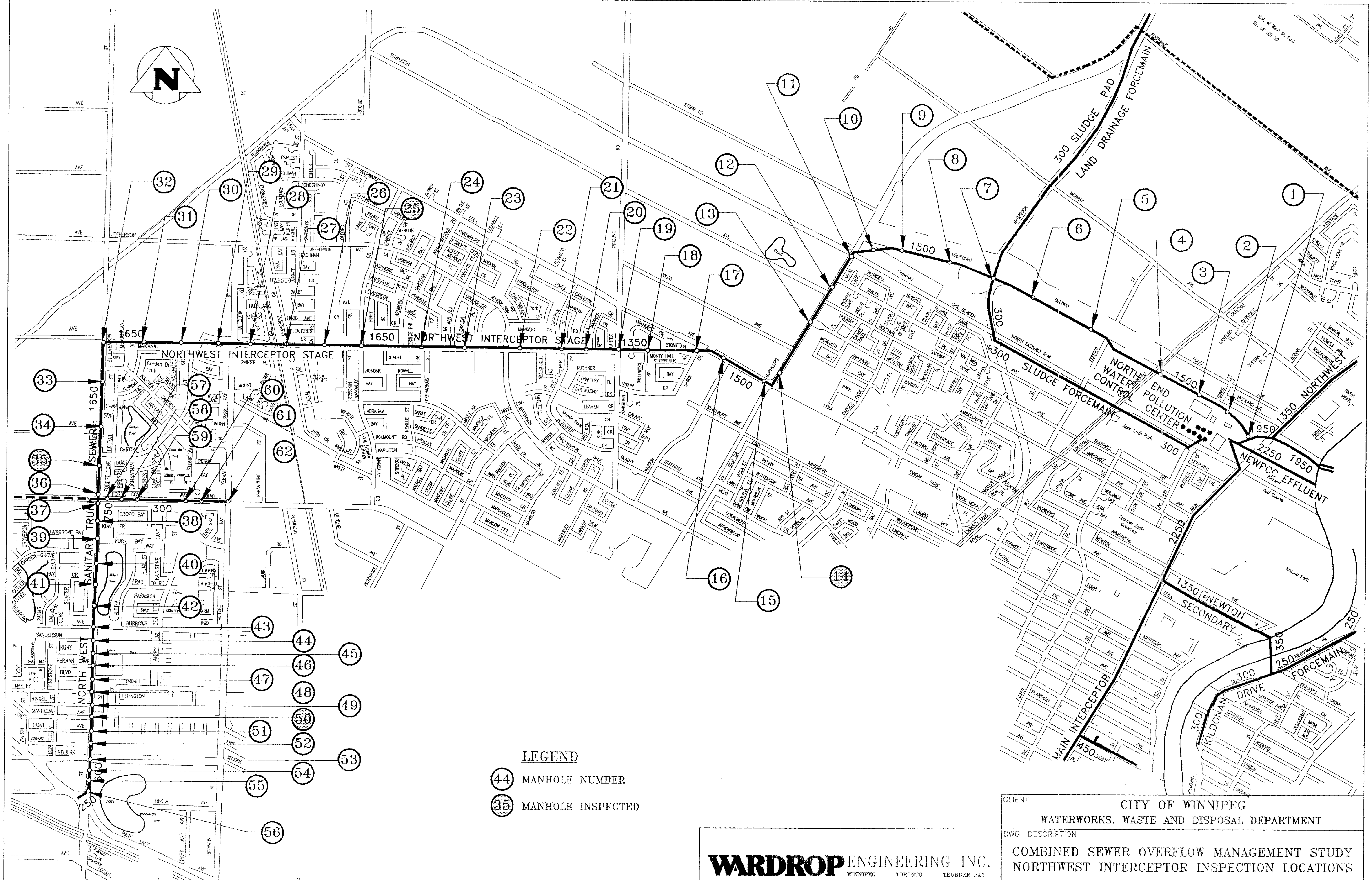
DWG. DESCRIPTION
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
MAIN INTERCEPTOR (WEST LEG) INSPECTION LOCATIONS

DESIGNED BY: D.N.B. | DRAWN BY: T.G. | DWG. NO.

TABLE D-1
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
NORTH WEST INTERCEPTOR INSPECTION SUMMARY

MH	Location	Date	Rim to Invert Depth	Water Depth	Water Vel.	Sed.	Pipe Size	Pipe Type	Struct. Cond.	Comments
14	McPhillips Street, north of Leila Avenue	Aug 5	8840	355	med-ium	25	1520	concrete	good	
25	Adsum Drive at Mandalay Street	Aug 5	6600	180	slow	255	1680	concrete	good	sludge buildup
35	King Edward between Inkster and Garton	Aug 5	6530	180	slow	380	760	concrete	good	heavy sludge buildup
50	King Edward at Manitoba	Aug 5	4170	180	slow	none	610	concrete	good	forcemain outfall in MH chamber

Note: All dimensions are in millimetres



CLIENT

CITY OF WINNIPEG
WATERWORKS, WASTE AND DISPOSAL DEPARTMENT

WARDROP ENGINEERING INC.
WINNIPEG TORONTO THUNDER BAY

DWG. DESCRIPTION

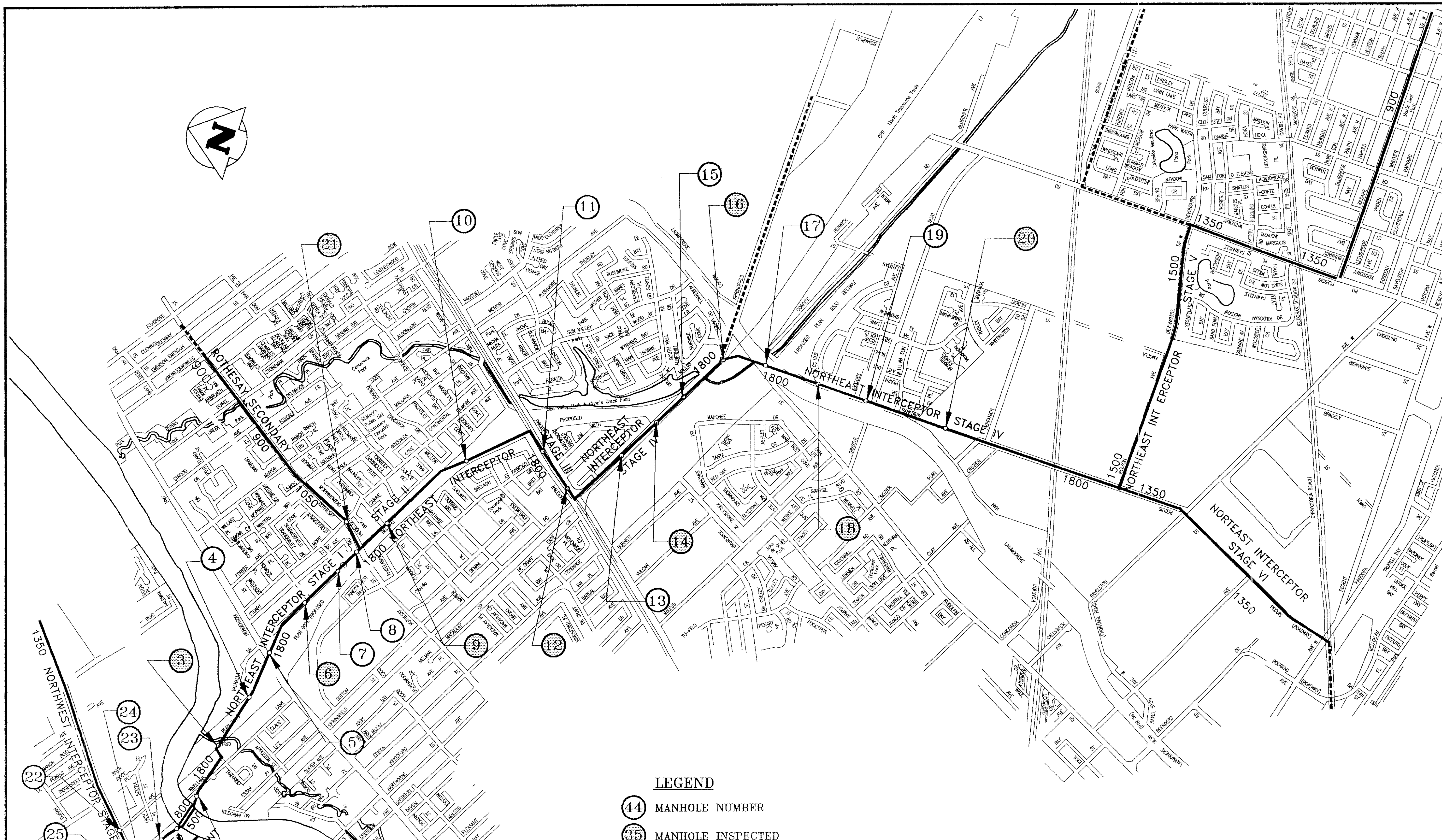
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
NORTHWEST INTERCEPTOR INSPECTION LOCATIONS

DESIGNED BY: D.N.R. DRAWN BY: T.C. DWG. NO.

TABLE E-1
COMBINED SEWER OVERFLOW MANAGEMENT STUDY
NORTH EAST INTERCEPTOR INSPECTION SUMMARY

MH	Location	Date	Rim to Invert Depth	Water Depth	Water Vel.	Sed.	Pipe Size	Pipe Type	Struct. Cond.	Comments
1	West bank of Red River	Aug 2	n/a	n/a	n/a	n/a	1830	concrete	good	river crossing outlet chamber
2	East bank of Red River	Aug 2	9750	560	turbulent	none	1830	concrete	good	river crossing inlet chamber
3	North of the Kildonan Corridor	Aug 2	9300	460	fast	125	1830	concrete	good	
6	Douglas Avenue, south of Pentland Street	Aug 2	10190	255	fast	none	1830	concrete	good	
9	Douglas Avenue, north of Rothesay Street	Aug 2	9630	330	medium	50	1830	concrete	good-fair	some exposed aggregate
12	Gateway Road, north of Springfield Road	Jul 18	8260	330	fast	none	1830	concrete	good	
14	Springfield Road, west of Gateway Road	Jul 18	8180	205	medium	75	1830	concrete	good	
16	Springfield Road, east of Bunns Creek	Jul 18	8380	305	slow	none	1830	concrete	good	
18	Ham Street, north of Cordite Road	Jul 18	7930	305	slow	50	1830	concrete	good	
20	Ham Street, south of Grassie Blvd	Jul 19	8480	280	slow	none	1830	concrete	good	
21	Rothesay Street at Gilmore Avenue	Aug 2	8610	230	slow	none	1070	concrete	good	

Note: All dimensions are in millimetres.



CLIENT

CITY OF WINNIPEG
WATERWORKS, WASTE AND DISPOSAL DEPARTMENT

DWG. DESCRIPTION

COMBINED SEWER OVERFLOW MANAGEMENT STUDY
NORTHEAST INTERCEPTOR INSPECTION LOCATIONS

WARDROP ENGINEERING INC.
WINNIPEG TORONTO THUNDER BAY

DESIGNED BY: D.N.B.	DRAWN BY: T.G.	DWG. NO.	
CHECKED BY: G.S.	DATE: FEB. 1995	940012-02-22	FIG. E-1