



City of Winnipeg
Water and Waste Department

Combined Sewer Overflow Management Study

PHASE 3 Technical Memoranda

Appendix No. 5a

INFRASTRUCTURE MODELLING

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1. INTRODUCTION

This Appendix (5a) presents the Phase 3 hydraulic analyses of combined sewer systems and interceptors tributary to the North End Water Pollution Control Centre (NEWPCC). Specifically, the hydraulic behaviour of combined sewer and interceptors in response to rainfall, the potential to implement CSO control through inline storage in selected districts, and the conveyance capacity of the existing NEWPCC outfall are discussed in the following sections.

1.1 BACKGROUND – PHASE 2 ANALYSIS

The Phase 2 analysis of the Winnipeg sewerage infrastructure focused on wet weather flow (WWF) from the combined sewer districts. The main effort of this analysis concentrated on the Main Interceptor and NEWPCC, which is tributary to 34 of the 42 combined sewer districts and 9,200 of the 10,500 ha of drainage area. The complete list of conclusions developed during the Phase 2 analysis is contained in Section 2.8 of the Phase 2 TM #2. Some of these points, specifically those most relevant to the Phase 3 analysis follow:

- The rated capacity of the Main Interceptor and the common sewer receiving flows from the Northeast and Northwest Interceptors is 6.14 and 6.83 m³/sec., respectively (total flow capacity to NEWPCC is 12.97 m³/sec. or 1121 ML/d). The interceptors have capacity to convey all diverted flows from small rain events (less than 3 mm of runoff in all CS districts). For larger storm events, potential overflows can occur at St. John's and other points on the upstream end of the system.
- A hydraulic model of the Main/Northwest/Northeast Interceptors including the NEWPCC surge well and pumping facility was built to analyze the system under WWF's. The model was not calibrated to WWF data but was deemed sufficiently representative to conduct the planning level Phase 2 analysis.

- Hydraulic analysis of the Main Interceptor system, under typical WWF conditions, indicates potential for overflows. An analysis of the system, including the combined sewer trunks and diversion facilities, under continuous dynamic simulation will be required to properly assess system performance.

The Phase 2 hydraulic modelling of the Main/Northeast/Northwest Interceptors and the NEWPCC provided a preliminary understanding of the WWF response and behaviour of the system. Phase 3 modelling was designed to further this understanding by developing a more sophisticated model to more accurately simulate the dynamic performance of the system under WWF's by incorporating the individual district trunks and diversion facilities.

2. MAIN INTERCEPTOR MODEL

A preliminary hydraulic analysis of the Main Interceptor system was carried out in Phase 1. This analysis was carried out to provide an initial estimate of the hydraulic capacity in the system. However, it became apparent that a more sophisticated model would be required to analyze the system under WWF conditions and provide accurate system conveyance capacity and overflow response.

In Phase 2, an XPSWMM Extran model of the NEWPCC interceptor system was developed. The model included the Main, Northeast and Northwest Interceptor sewers. The model also included the NEWPCC surge well, raw sewage pumps, and the outfall. The model set-up, development and WWF analysis is documented in the Phase 2 Technical Memorandum No. 2, Infrastructure/Treatment.

The Phase 2 computer model was used to assess dynamic WWF response of the systems. The initial analyses were based on design conditions (i.e., flows of 2.75 times ADWF from the tributary combined sewer districts) and progressed to an evaluation of likely field conditions. From this analysis it was apparent that flows from the districts, particularly those districts with gravity flow connections to the interceptor, would be governed by the head differential between the combined trunk sewer and the interceptor sewer. Therefore, the next logical step in assessing the interceptor's WWF performance was to add the combined sewer trunks and diversion facilities to the model. Improvements to the XPSWMM model were carried out in Phase 3 and are documented in the following sections.

2.1 Phase 3 Development

The combined sewer district diversion facilities were added to the XPSWMM model of the interceptor sewer system developed in Phase 2 of the CSO study. This included:

- The downstream portion of the combined trunk sewer.
- The diversion weir and off-take pipe.

- Pumping facilities, if required.
- Outfall sewers to the river system.

A schematic representation of a typical combined sewer district in the model is shown on Figure 2.1.

The individual combined sewer district diversion facilities were “calibrated” before they were added to the model. This was accomplished by adjusting the diversion configuration (weirs, piping, etc.) in the model until the diversion rates matched the rates calculated during the Phase 2 analysis. This was done for incipient weir overflow and particularly, for the gravity districts, at various flow levels in the combined sewer trunk.

After adding the district diversion facilities the XPSWMM Extran model consisted of the following components:

- 286 conduits and 286 nodes
- 37 outfalls
- 25 pumps
- 17 storage junctions (pumping station wet wells)
- 11 orifices
- 32 weirs

The model was also updated with subcatchment data from the Combined Sewer District Runoff model. The development of the runoff model is described in detail in the Phase 2 Technical Memorandum, Problem Definition, and will not be covered further here. The runoff model consists of a single subcatchment for each of the combined sewer districts. These subcatchments were imported to the model and made tributary to the upstream node on the CS trunk. Typical runoff hydrographs (in cubic metres per second) for the storm of May 22, 1995, are shown on Figure 2.2.

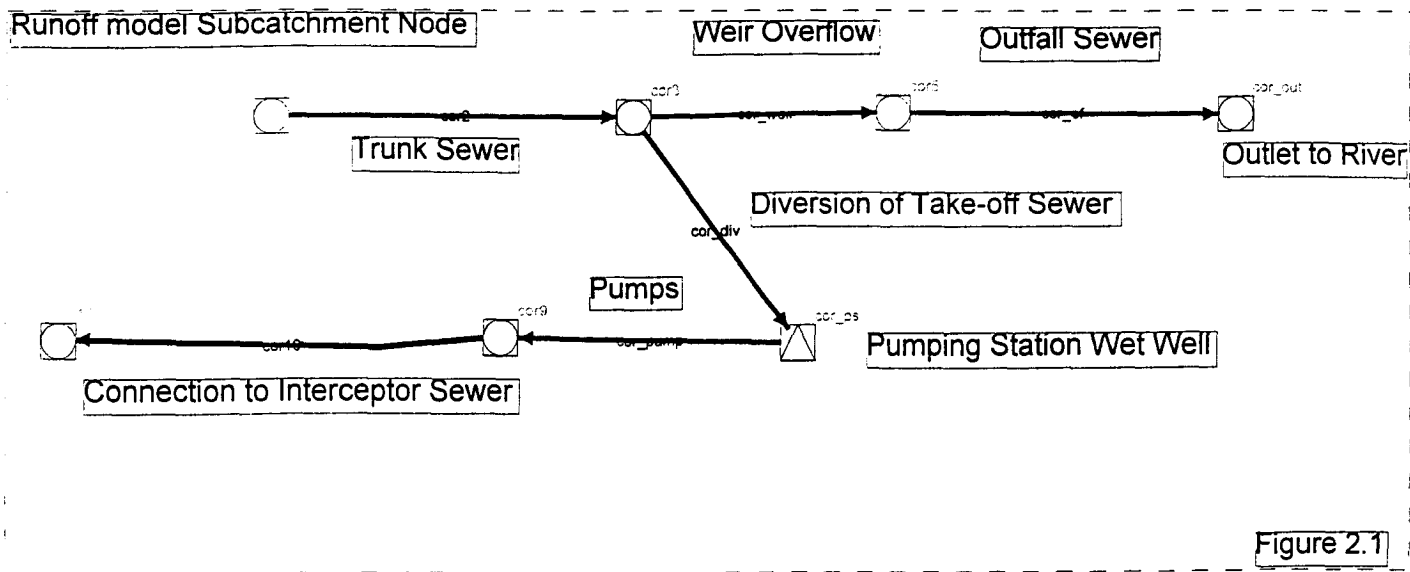
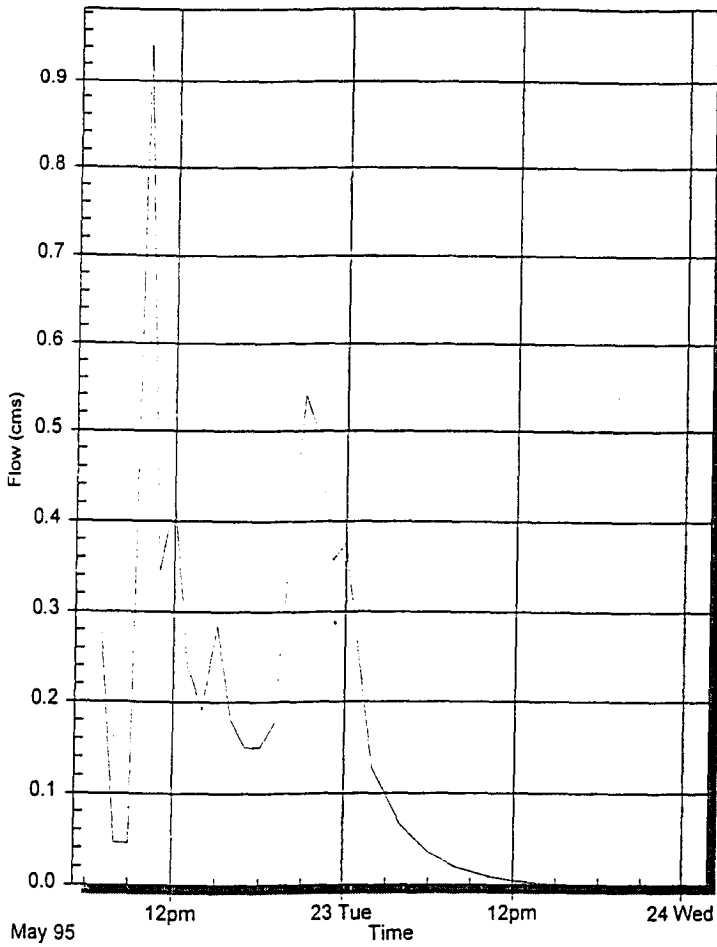
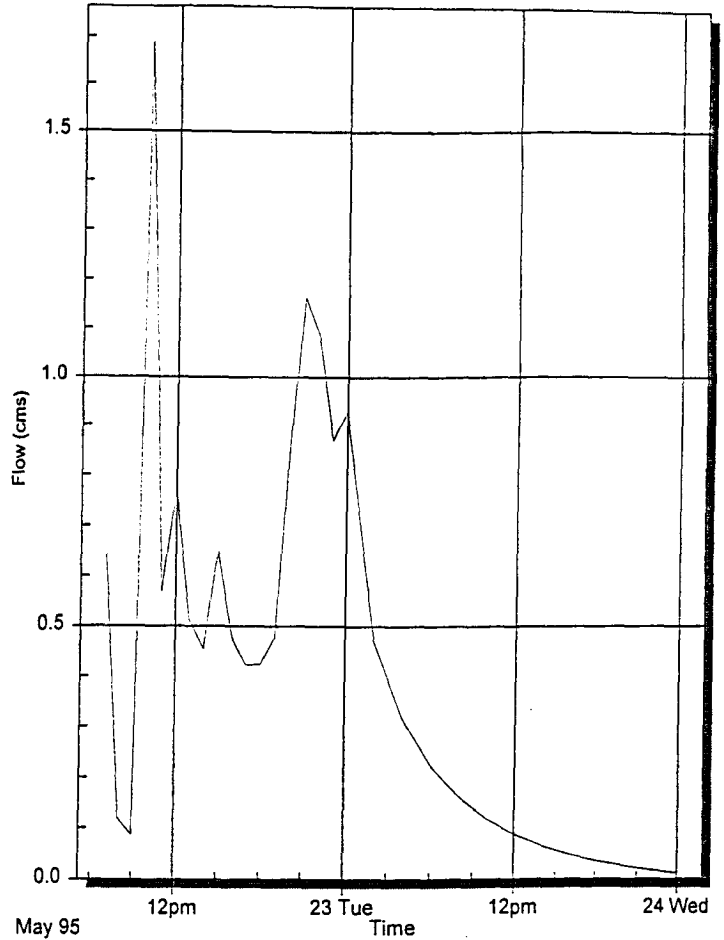


Figure 2.1

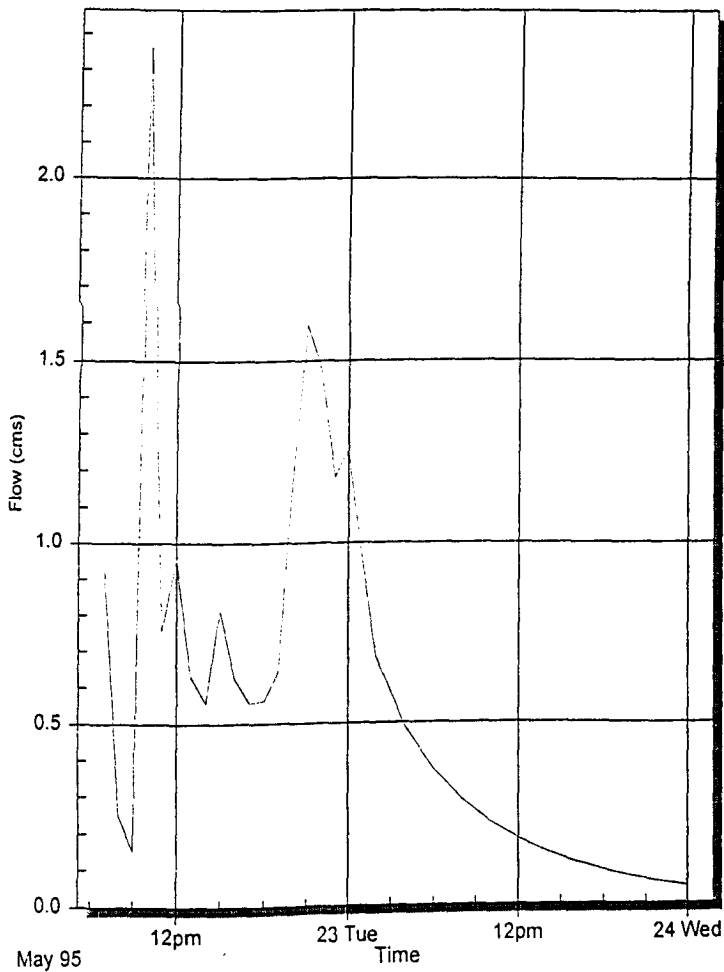
Node - new1
[Max Flow = 0.9432]



Node - lin1
[Max Flow = 1.6890]



Node - haw1
[Max Flow = 2.3635]



Node - alex1
[Max Flow = 2.9864]

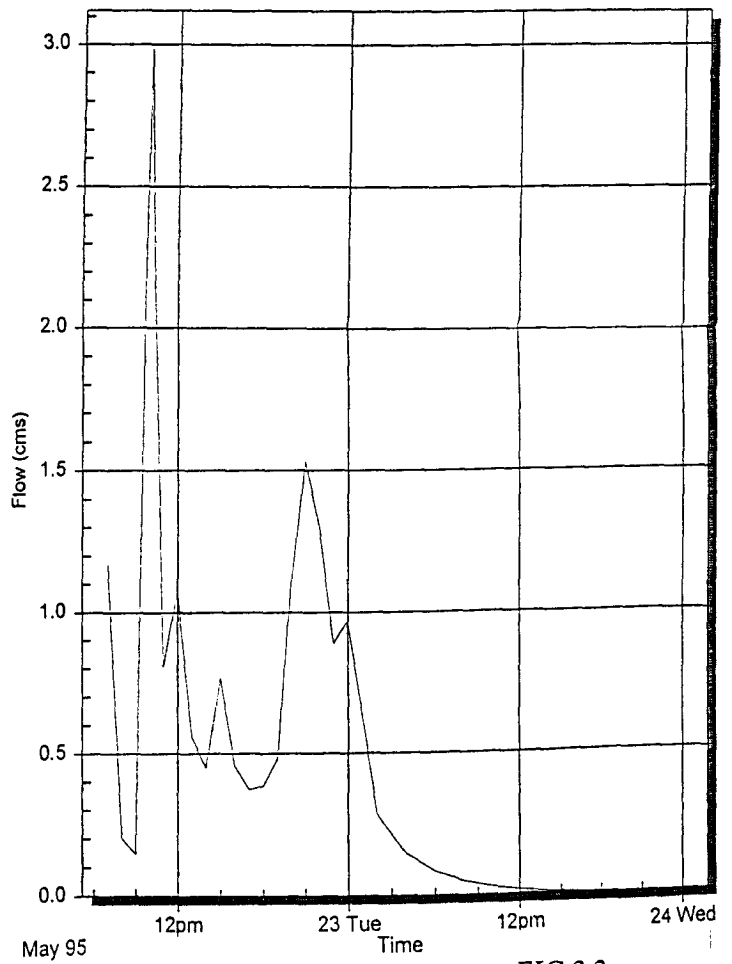


FIG 2.2

The hydrographs are for the following districts, listed in order from left to right, and from top to bottom:

- Newton (new 1)
- Linden (lin 1)
- Hawthorne (haw 1)
- Alexander (alex 1)

2.2 OVERVIEW

The Phase 3 model was set up for calibration of the interceptor sewer system to actual rainfall events. The rainfall data was obtained from the Winnipeg International Airport gauge. At this station rainfall, data is recorded in one-hour increments. While this is a longer period than is typically used for Extran simulations, the one-hour period is much less than the system's time of concentration. Furthermore, it was determined that the calibration events would be the long duration, low intensity events that were City wide as opposed to the more localized, high intensity, short duration events. On this basis, it was assumed that the one-hour increment rainfall data would be suitable for the calibration analysis.

Rainfall data is read by the XPSWMM Rainfall utility and converted to an interface file. The runoff block reads and processes the interface file and develops another interface file containing the runoff hydrograph that is used as input for Extran. Extran routes the runoff hydrograph through the piping system to the diversion point. At the diversion, the flow is either diverted to the Interceptor (by gravity flow or pumping depending on the district) or to the river, according to the hydraulic rules in Extran. Flows diverted to the interceptor are routed to the NEWPCC. Flows from the Northeast and Northwest Interceptors are added to the flow in the Main Interceptor upstream of the surge well. The northeast and northwest flows to the interceptor were based on monitored results.



NORTHWEST INTERCEPTOR

NORTHEAST INTERCEPTOR

1650 1350

NEWPCC

LEILA ±

2250
1950

1800

1650

MAIN INTERCEPTOR

JEFFERSON

ST. JOHN'S

SELKIRK

1950

ASSINIBOINE

CLIFTON

1350 1800

RIVERBEND ±

LEGEND

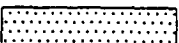
COMBINED SEWER DISTRICT BOUNDARY



INTERCEPTORS



SECONDARY INTERCEPTOR TO WEWPC OR SEWPC



WEIR (BUG) & LEVEL DETECTOR*



LEVEL DETECTOR*



LEVEL DETECTORS*

* SONIC OR PRESSURE TRANSDUCERS

FIG 2.3

2.2.1 Flow Monitoring

Monitored flow tributary to the NEWPCC was available from the following sources:

- Flow data from weirs and level recorders in the Northeast and Northwest Interceptors.
- Surge tank levels and raw sewage pumping at the NEWPCC.
- Levels on the Main Interceptor.

These monitoring locations are shown on Figure 2.3.

This section focuses on the level monitors placed on the Main Interceptor sewer. Monitoring at the NEWPCC and on the Northeast and Northwest Interceptors is discussed separately in Appendix 5b – NE/NW Monitoring.

Isco 4110 Ultrasonic Flow Loggers were installed at the following locations:

- 2250 mm Main Interceptor on Main Street at Hartford Avenue (Node 127 in the XPSWMM model).
- 2250 mm Main interceptor on Main Street, downstream of the Sutherland secondary sewer connection (Node 111).
- 1500 mm on west leg of Main Interceptor on Broadway at Donald Street (Node 233).

Levels were recorded over the summer months during 1995, 1996, and 1997. Over this period, there were approximately ten significant rainfalls over the combined sewer service area. Unfortunately, not all of the flow data was available from these events as a result of malfunction of the recorders. The malfunctions were primarily attributed to the hostile or extreme variations in the sewer system environment. The usable data was reviewed and used for the calibration exercise.

2.2.2 Main Interceptor Calibration

The first step in the calibration analysis involves selection of a storm event. This depended on the availability of information from a number of sources, including:

- Flow data from the Northeast and Northwest Interceptors.
- Pump and level data from the NEWPCC.
- Level data from the Main Interceptor.
- Rainfall data with a consistent, or nearly so, pattern over the tributary area.

After an event was selected, the XPSWMM model was set-up for the simulation. This entailed entering the flow hydrographs for the Northeast and Northwest interceptors and the rainfall data. Rainfall data was routed through the Runoff Block to develop runoff hydrographs, which were subsequently routed through the piping model (Extran). The results of the Extran simulation were subsequently compared to the monitored data to assess the accuracy of model simulations.

2.3 MODEL RESULTS

The calibration exercises were based on two rainfall events which occurred on May 22, 1995, and August 18, 1995. Data from 1996, was also. Although there were several significant storm events, they were not used for model calibration due to uncertainty regarding the flows in the Northeast and Northwest Interceptors.

The May 22, 1995, event had an approximate return period of two to three times per year and was fairly consistent over the combined sewer tributary area. The calibration data consisted of the flows from the Northeast and Northwest Interceptors and flow and level data from the NEWPCC. Level data from the Main Interceptor was not available for this event. However, the NEWPCC data allowed an end-of-pipe comparison.

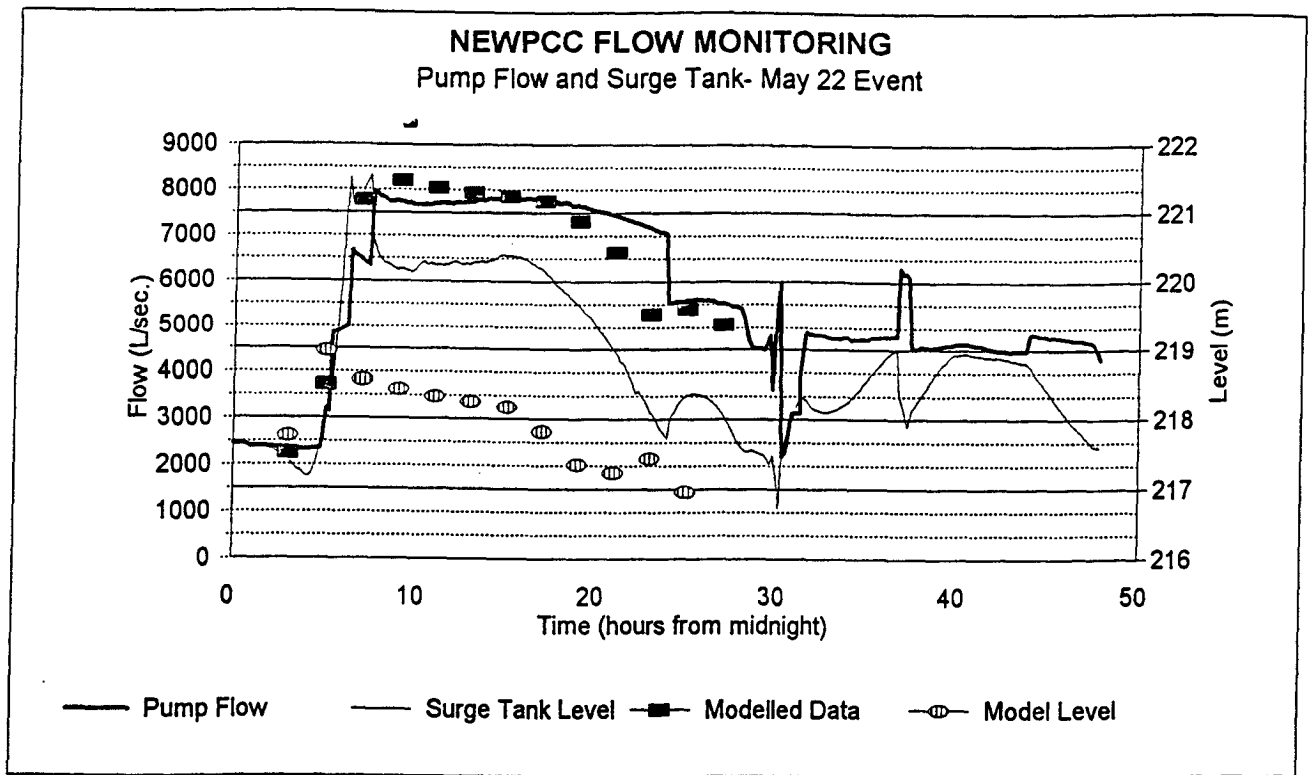


Figure 2.4

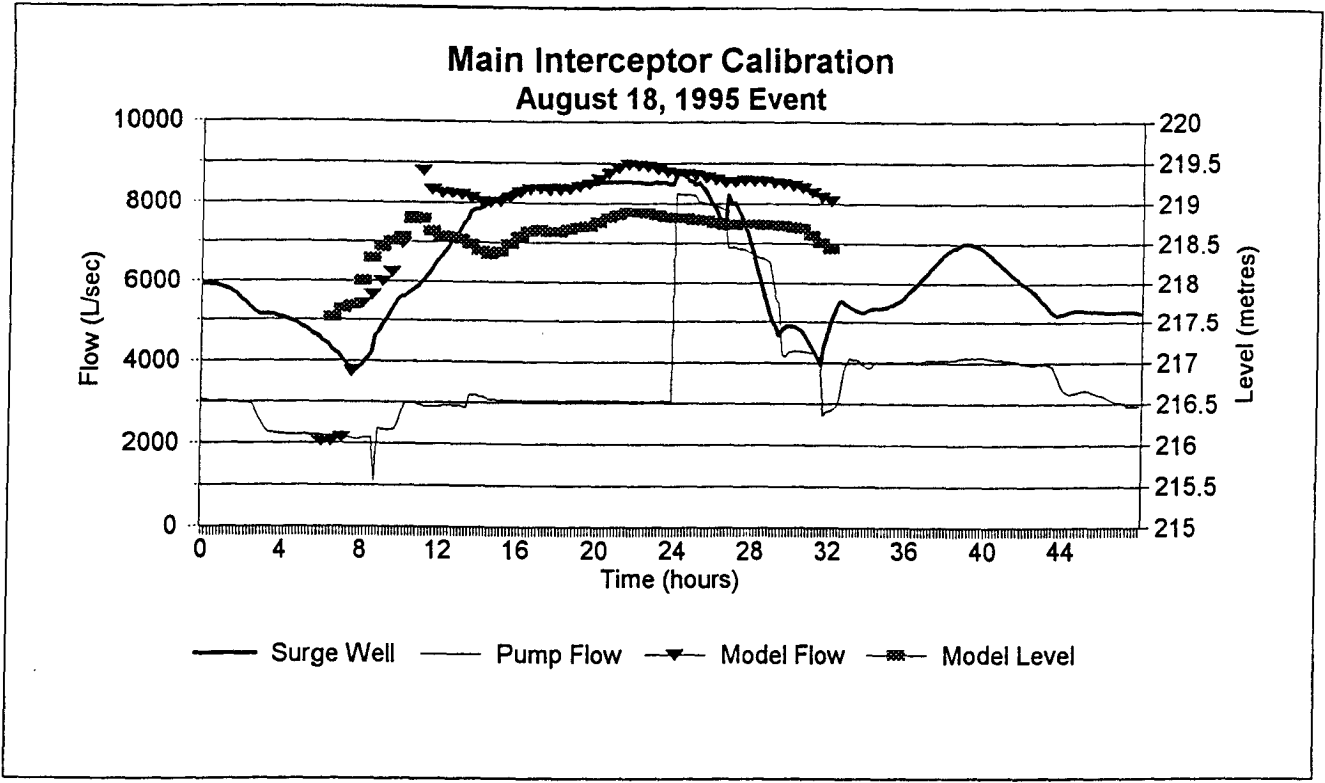


Figure 2.5

**Main Interceptor Calibration
August 18, 1995 - Node 111**

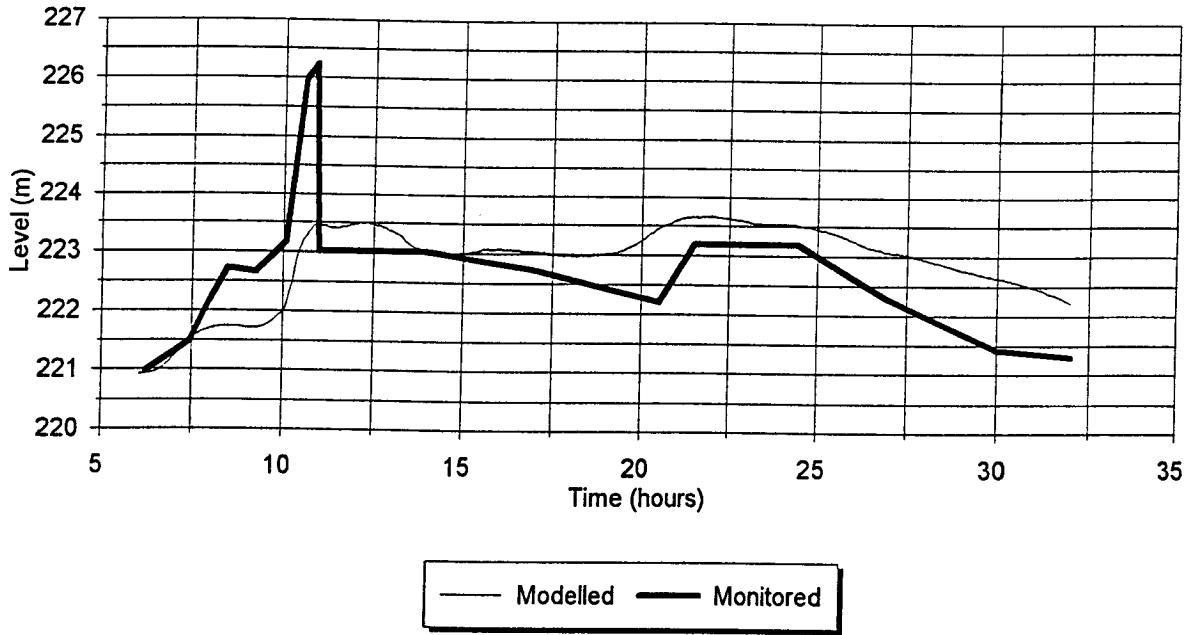


Figure 2.6

The results from this simulation are show on Figure 2.4. Predicted pump flows and levels at the NEWPCC were compared to the measured values. A very good correlation between the recorded pump flows and the XPSWMM model flows was observed. This indicates that model representation of the interception facilities and the travel time in the interceptor sewer are providing a good approximation of the actual WWF conditions. Conversely, a large discrepancy between the monitored and predicted surge tank levels was observed. Variations in predicted and monitored levels are likely due to operating protocols in effect at the time.

The August 18, 1995, storm event had a return period of approximately one year. Rainfall pattern was not consistent over the City, with higher accumulations of rainfall to the east; however, the rainfall pattern appeared to be fairly consistent over the combined sewer service area.

Monitoring data from the NEWPCC, the Main Interceptor and the Northwest Interceptor was available for this simulation. Flows from the Northeast Interceptor were not available for this simulation and required the flows to be approximated based on the interceptors response to previous storm events. It was determined that the approximated flows would be sufficiently accurate for the analysis since the WWF's in the system are dominated by the Main Interceptor.

The results of this calibration simulation are shown on Figures 2.5 and 2.6. Figure 2.5 compares the monitored and predicted flows and levels at the NEWPCC. As in the previous simulation, the model predictions closely correspond to the measured pump flow at the plant, except for the tail end of the simulation. This difference is likely due to the operations at the plant whereby the pumping rates were reduced to allow the levels in the surge well to rise. There is no correlation for levels in the surge tank. It is apparent from the graphic that the levels were maintained at an elevation of 216.5 m to maximize available storage in the interceptor system. After the storm, the levels were allowed to rise. Figure 2.6 shows the model comparison to the measured depths on the Main Interceptor, downstream of Sutherland Avenue (Node 111). This figure shows a good correlation to the monitored data with the exception of a spike near the start of the event. This spike was not apparent at the upstream monitoring location and is not reflected in the plant data.

Therefore, it was assumed that the spike was due to a short-term equipment malfunction. Accordingly, the model provides a good representation of flows in the interceptor for this event.

The result of the analysis of the 1995 monitoring data indicates that the model provides a good representation of flows in the Main Interceptor system.

3. NEWPCC OUTFALL ANALYSIS

In Phase 2 it was determined that the raw sewage pumping capacity at the NEWPCC was 1056 MLD (12.22 m³/sec.). In Phase 3 it was determined that the WWF operation of the Main Interceptor could be enhanced to deliver 970 MLD (11.23 m³/sec.) to the plant. The Phase 2 modelling also indicated that the NEWPCC outfall could convey 970 MLD with slight surcharge at the upstream end.

NEWPCC operational personnel questioned the capacity of the outfall. Flooding of the finger weirs in the secondaries had been observed when flows through the plant reached the design rate of 827 MLD (9.57 m³/sec.). It was their opinion that the outfall was surcharging and causing the flooding. The design rate is based on the secondary capacity of 600 MLD with 227 MLD bypassed after primary treatment.

The capacity of the outfall would impact any control alternative that would require enhanced WWF operation of the Main Interceptor (i.e., increased district interception rates). Therefore, it was decided to review the outfalls operation with the XPSWMM model and calibrate the model with flow and level measurements taken by plant personnel.

3.1 HYDRAULIC ANALYSIS

NEWPCC staff collected the flow data for the analysis over the period from May 2 to May 16, 1995. The various flow and level measurements were taken, as often as possible, under constant flow conditions through the plant. Flows ranged from 105 to 484 MLD. The data included:

- Flow through the plant in MLD.
- Depth of flow in outfall immediately outside the plant.
- Depth of flow in outfall at manhole approximately 200 m from the river.
- River level.

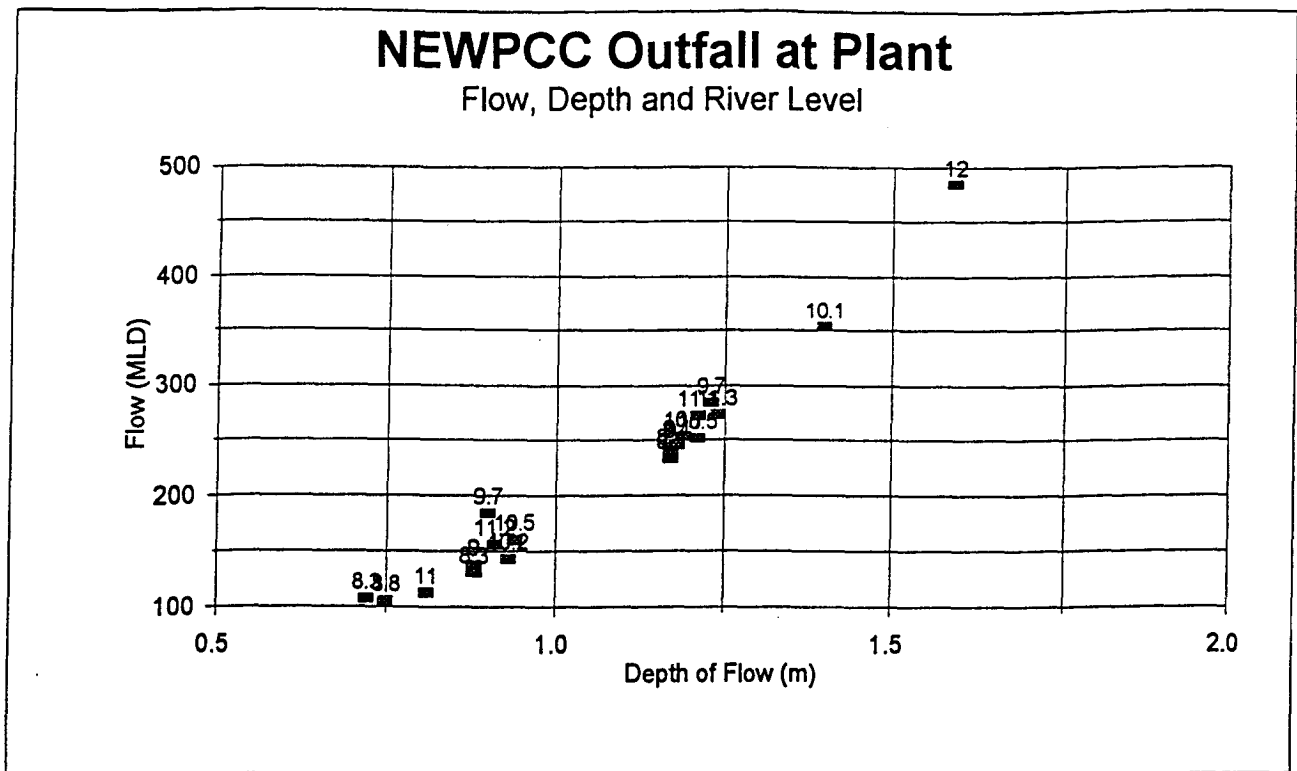


Figure 3.1

NEWPCC Outfall Analysis

Outside Plant

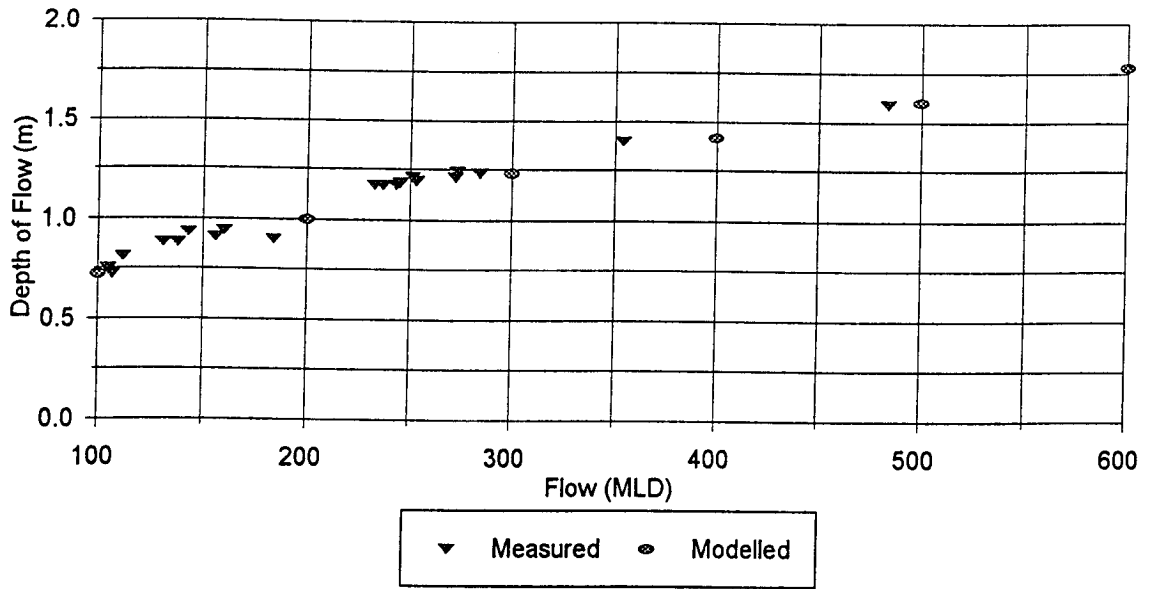


Figure 3.2

NEWPCC Outfall Analysis

Manhole approx. 200 m from River

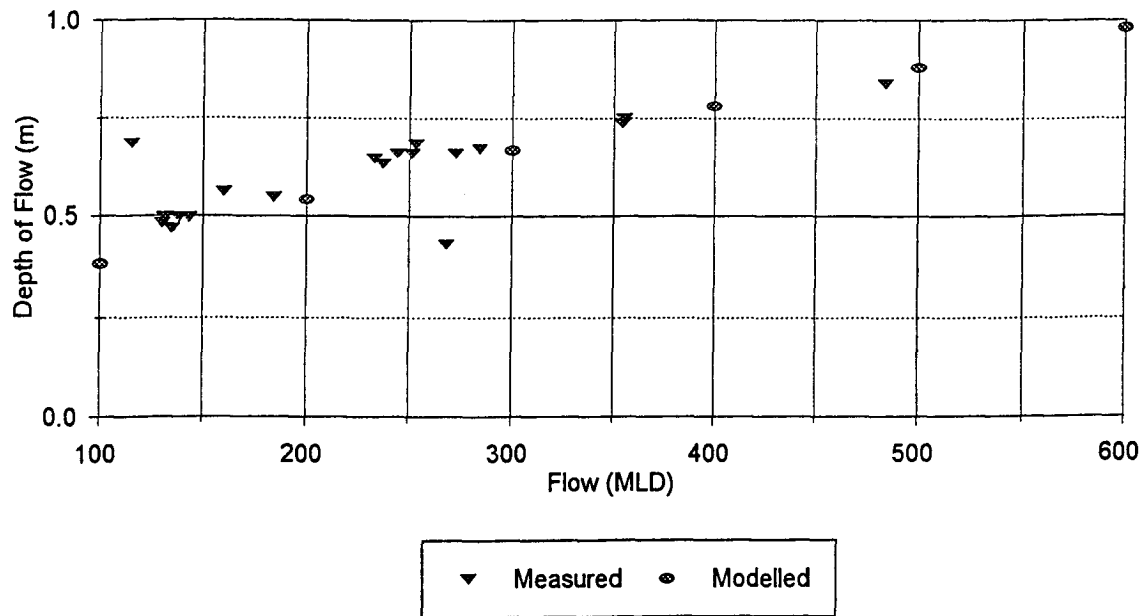


Figure 3.3

Levels recorded at the manhole outside the NEWPCC are plotted against the plant flows on Figure 3.1. The river levels, taken at the time of the level measurements, are plotted beside the data points.

3.1.1 Comparison to Flow Measurements

An XPSWMM model of the system was prepared and used to assess the hydraulic response of the system. The model included the downstream portion of the interceptor system, the surge well and pumping facilities, and the outfall. The outfall length, slope and diameter were obtained from the original 1936 construction drawings and the 1951 reconstruction drawings (for the replacement of the 2.29 mm diameter CMP end section with a 1.98 mm diameter pipe).

Hydraulic analysis consisted of entering a flow into the model and routing the flow through the system to the river. Model simulations were run long enough for the system to reach steady flow conditions in the outfall. The analysis started with a flow of 100 ML/d and increased in 100 ML/d increments. Normal river levels were used in all simulations. The XPSWMM model depths were then compared to the measured depths.

Comparison between measured and modelled flow depths in the outfall at the NEWPCC site is shown on Figure 3.2. An extremely good correlation between the model and the measured depths over the entire range of flows was achieved. However, a discrepancy in flow level at the manhole 200 m from the river was observed. The model consistently predicted levels approximately 0.8 m lower than the measured depths for a given flow. This required further investigation and analysis to explain why the difference did not increase with flow, as would be expected from any downstream obstruction with a corresponding head loss.

Additional site surveys were carried out to resolve the matter. The survey identified an error in the original manhole rim to pipe obvert measurement of 0.78 m (probably due to the manhole being offset from the pipe centreline). The original data was revised and the comparison to the model results is shown on Figure 3.3.

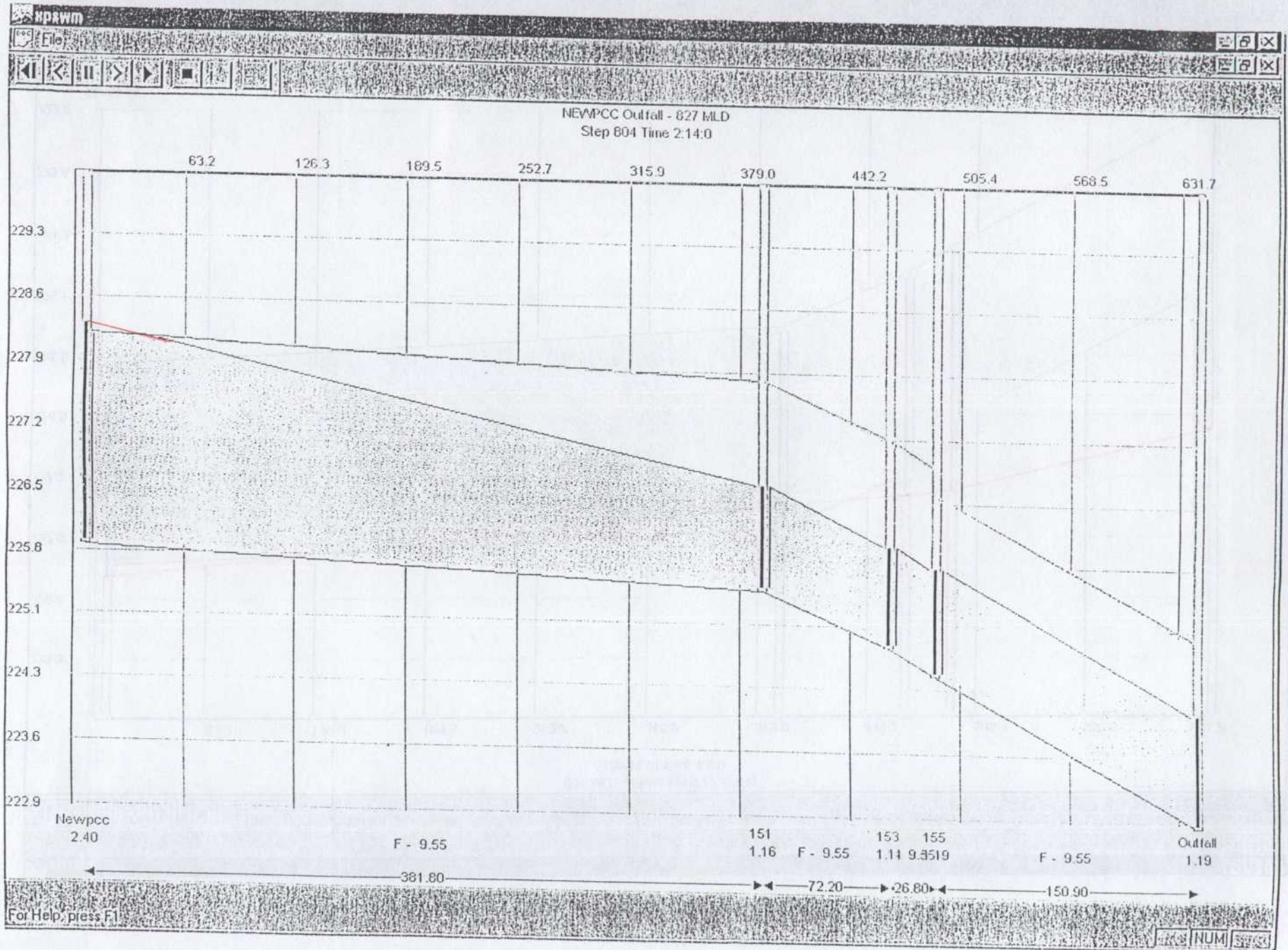


Figure 3.4

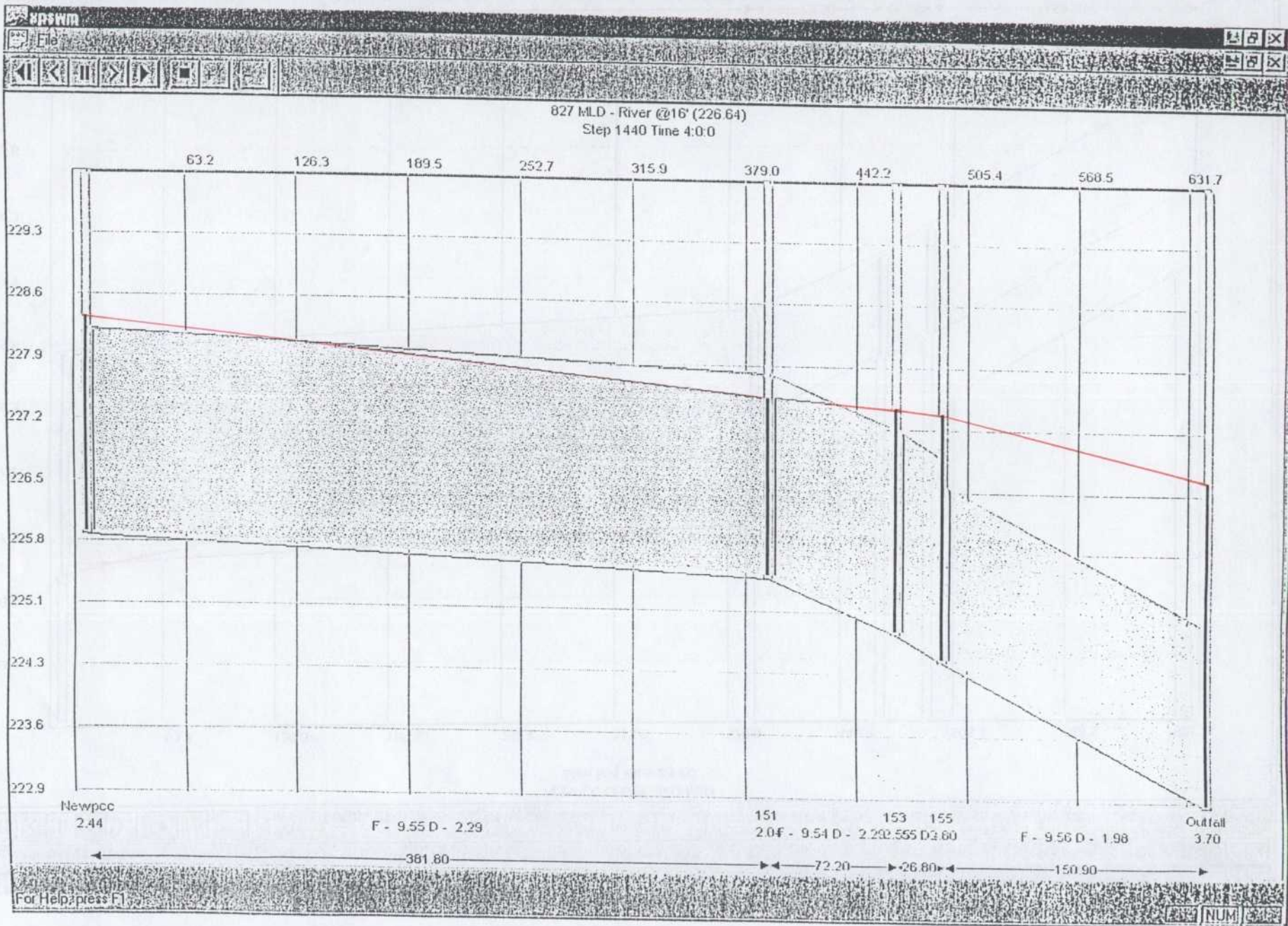


Figure 3.5

This calibration to the model resulted in an accurate hydraulic representation of actual flow conditions in the outfall. The calibrated and field verified model was then used to analyze the outfall under the design flow condition of 827 MLD and to determine the outfall's sensitivity to river level.

3.1.2 Outfall Under Design Flows

A design flow of 827 MLD was routed through the model. This simulation indicated that, for the most part, the outfall would be flowing less than full. A surcharge of approximately 0.2 m was predicted at the top end of the system. The hydraulic grade line predicted by the model is shown on the outfall profile in Figure 3.4.

Model predicted flow conditions were compared to the hydraulic calculations contained in the NEWPCC Secondary Expansion Functional Design Report. The design report also indicated that the top end of the outfall would be slightly surcharged under the design flow condition, which compares favourably to the model's predictions. There is a discrepancy in the geodetic elevation of the surcharge. The functional design report states a surcharge elevation of 228.08 m, compared to an elevation of 228.4 m predicted by the model. The difference in elevations could be resolved by a site survey to verify the elevation of the outfall at the plant. At this time the finger weir elevations could also be verified to determine if the outfall capacity does result in backup at the design flow rate.

3.1.3 Sensitivity to River Level

A hydraulic analysis was carried out by routing the design flow of 827 MLD through the outfall model with varying river levels. The river level was increased (in the model) until a change was noted in the top end of the outfall. This did not occur until the river level was set at 228.4 m (16 ft). Therefore, river level does not impact design flow conditions in the outfall until river stage exceeds an elevation of 228.4 m. The outfall profile and hydraulic grade line for this event is shown on Figure 3.5.

4. CLIFTON DISTRICT

A review of the combined sewer district sewer infrastructure determined that in many of the districts, significant potential for developing inline storage exists. The inline storage can be used to minimize overflows to the rivers. This is particularly relevant to those districts in which a large diameter relief trunk sewer has been constructed for basement flooding relief. A hydraulic analysis needed to be performed to determine the extent of available storage that could be utilized without affecting the current level of basement flood protection. An XP SWMM model of the Clifton District was developed, calibrated and used to test various inline storage alternatives under a number of rainfall and operational scenarios to determine how much storage could be utilized and not reduce the current level of basement flood protection.

4.1 INTRODUCTION

A logical first step in CSO control is optimizing the existing infrastructure to minimize wet weather overflows. This optimization typically includes the efficient use of available inline storage. Such storage would be dewatered and conveyed by the existing interceptor system to the Water Pollution Control Centres after the wet weather period has passed. The utilization of inline storage has the potential to reduce both the number and volume of overflow to the Rivers.

Winnipeg's flat terrain and relatively shallow river system necessitated the use of large diameter sewers installed at a minimum grade. Accordingly, some of these trunk sewers can be easily adapted for inline storage with a regulating device, such as a gate or inflatable dam. It is fundamentally important that inline storage not impact the level of basement flood protection. Detailed hydraulic analysis was required to properly assess the effects of implementing inline storage and the possible reduction in CSOs. This was accomplished by detailed hydraulic modelling of the system to determine the response to wet weather flows in conjunction with system modifications for inline storage works.

Modelling was recommended for the following reasons:

- To maintain the current level of basement flood protection.
- To assess WWF conditions in the combined and relief sewers.
- If required, to set operating protocols for an inline storage pilot.

4.2 MODEL SELECTION AND DEVELOPMENT

The XPSWMM model had been previously selected and used to model the combined sewer and interceptor sewer system for the CSO Management study. Since the model could also approximate real time controls such as gates and weirs, it was an obvious choice for the inline storage analysis.

Modelling all 42 of the combined sewer districts was beyond the scope of the study and not required for a planning level assessment. Selecting and modelling one representative district was considered reasonable to make a preliminary assessment of the extent to which inline storage could be implemented without affecting the level of basement flood protection. A screening exercise was carried out to narrow the selection of candidate combined sewer districts for possible modelling of inline storage. As a result of this selection process, the Clifton District was determined to be the best candidate for detailed modelling. This district met the following selection criteria:

- Preliminary calculations indicated a significant amount of potential storage in the system.
- It had been previously modelled with SWMM.
- Relief sewers, including a major relief trunk sewer had been constructed.
- It had two major outfalls, the original combined sewer and the Storm Relief Sewer (CS and SRS) for testing control technologies in the event of pilot testing.
- The Clifton relief system was designed to permit the implementation of controls to access available inline storage for pollution abatement purposes.

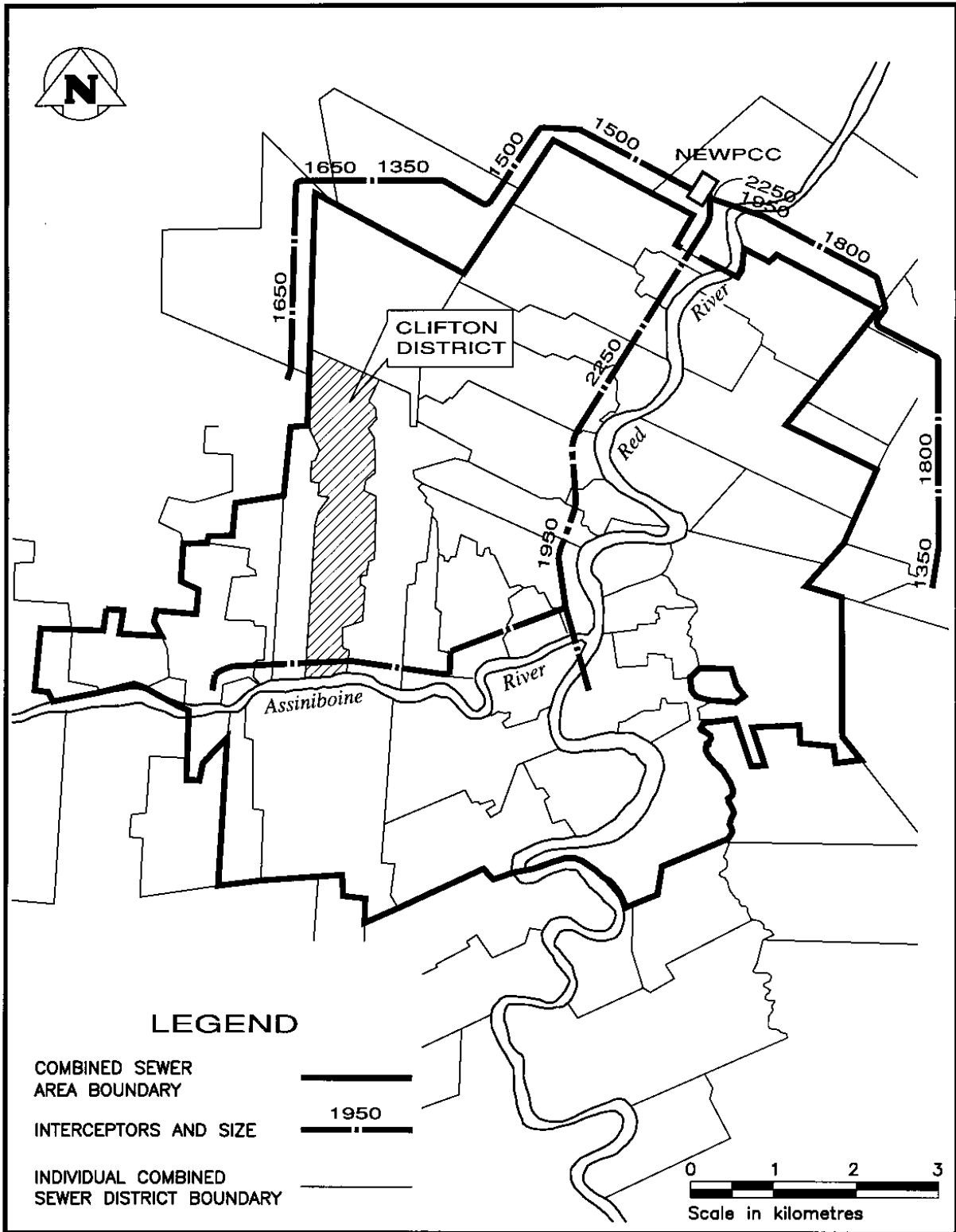


Figure 4.1 – Winnipeg's Combined Sewer Service Area

CLIFTON COMBINED TRUNK SEWER



Figure 4.2 – Profile of Downstream Portion of Combined Trunk Sewer

The Clifton Combined Sewer District is located in the west central portion of the combined sewer area and is identified on Figure 4.1. The combined trunk sewer is a 2.30 m x 2.97 m high egg-shape sewer at the outlet. Dry weather flow interception is effected with a weir that diverts the flow to a pumping station, which in turn pumps to the interceptor system. The diversion capacity is 236 litres per second, or a nominal 3.1 times average dry weather flow. The major relief trunk consists of a 2700 mm diameter sewer which discharges directly to the river. A profile of the Clifton combined trunk sewer is shown on Figure 4.2.

4.3 ORIGINAL CLIFTON MODEL

The Clifton District was previously modelled as part of the Basement Flood Relief (BFR) program. The analysis was carried out in 1978, using an earlier version of SWMM (MacLaren, 1978). The model was calibrated to existing and historical conditions and used for the analysis of the existing level of basement flood protection in the district and the design of the relief works to upgrade the district to a five year level of basement flood protection.

Land use in the district consists of approximately 70% single-family residential and 30% commercial and industrial. The original Runoff model partitioned the 415 ha contributing area into 400 subcatchment areas. Subcatchment areas ranged in size from 0.4 to 9.3 ha, but were typically about 1 ha in size. The overall district imperviousness was calculated at 44%. This relatively high value was attributed to the high percentage (77%) of residential rooftops discharging to the combined sewer system through directly connected downspouts and large paved commercial lots. The Extran model of the combined sewer network consisted of 459 conduits and 440 nodes.

4.3.1 XPSWMM Model Development

XPSWMM model development was hampered by a lack of detailed input data from the original model. A digital copy of the model could not be located. Therefore the XPSWMM

model was recreated based on hard copy information contained in the District's pre-design report (MacLaren, 1978).

The runoff data consisted of an output listing from the model. This provided specific data on the subcatchment runoff parameters with the exception that the area was rounded to the nearest 0.43 ha (1-acre). This was not considered sufficiently accurate for a detailed analysis. Fortunately, a small scale drawing of the catchment areas was available. This data was transposed onto a CAD drawing and the individual areas calculated. This provided a very good comparison to the individual catchment sizes as well as the overall contributing area of the Clifton District..

As noted previously, the original model was developed in 1978. The district was reviewed to determine if any significant changes had occurred which would impact the district hydrology. This was accomplished by a visual examination of the district, and a review of the available construction drawings. This resulted in two changes to the original data.

Over the years, the City of Winnipeg has carried out a program to inform and educate the residents of the benefits of downspout disconnection. The benefits of this program are evident. The amount of connected downspouts has dramatically decreased from 77% in 1978, to less than 5% in 1996. The residential area subcatchments were revised to reflect this change. In addition, subcatchments were revised, or removed from the model to account for minor stormwater separation, which had been implemented in the interim.

The available Extran data consisted of an output listing of the combined sewer system, however, this data did not include the storm relief sewers installed for basement flood relief. The latter information was obtained from the as-constructed drawings of the relief works. The relief sewers, and the associated interconnections to the combined sewer system, were added to the model.

CLIFTON COMBINED SEWER DISTRICT

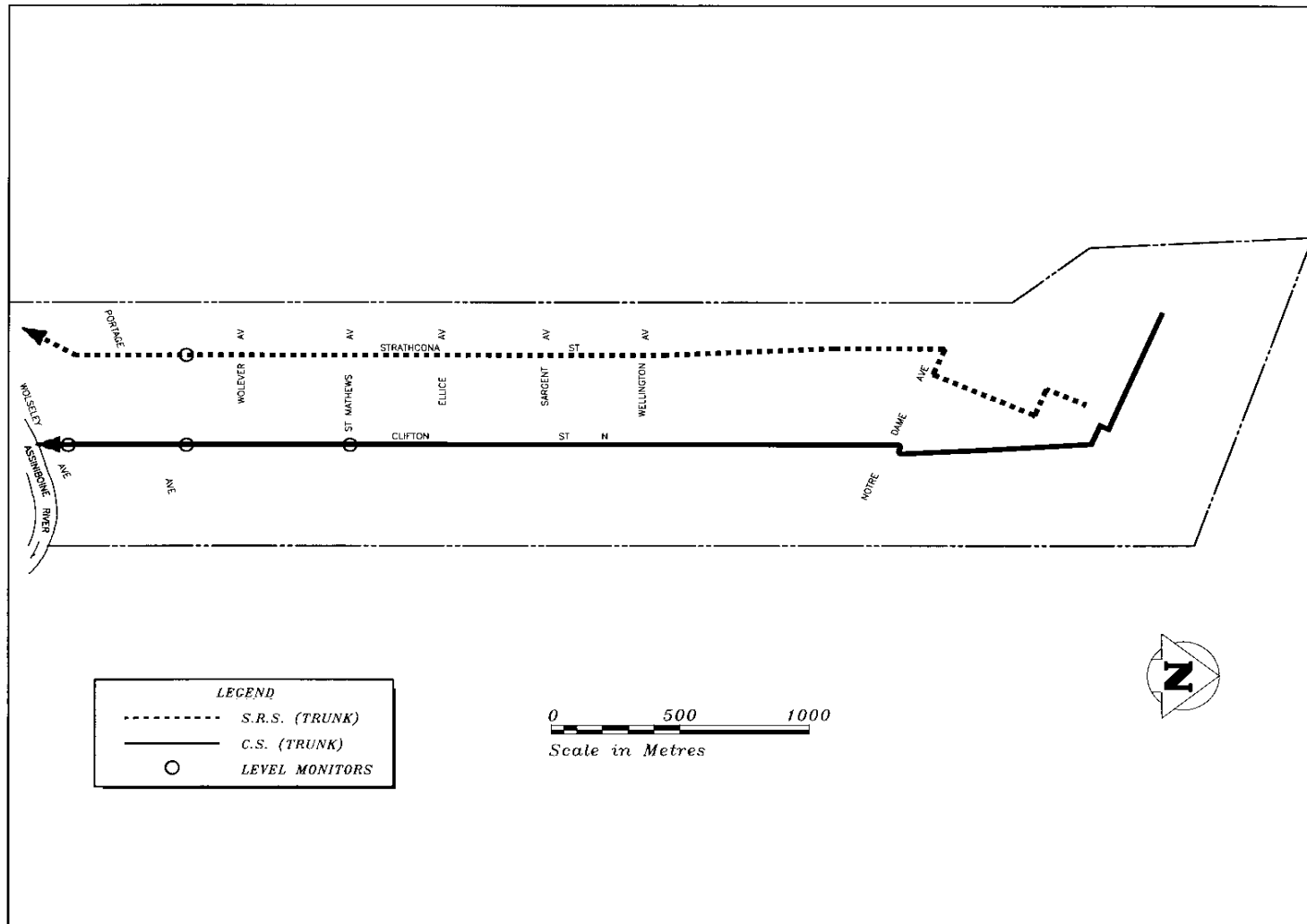


Figure 4.3 – CS and SRS Trunk Sewers – Level Monitoring Locations

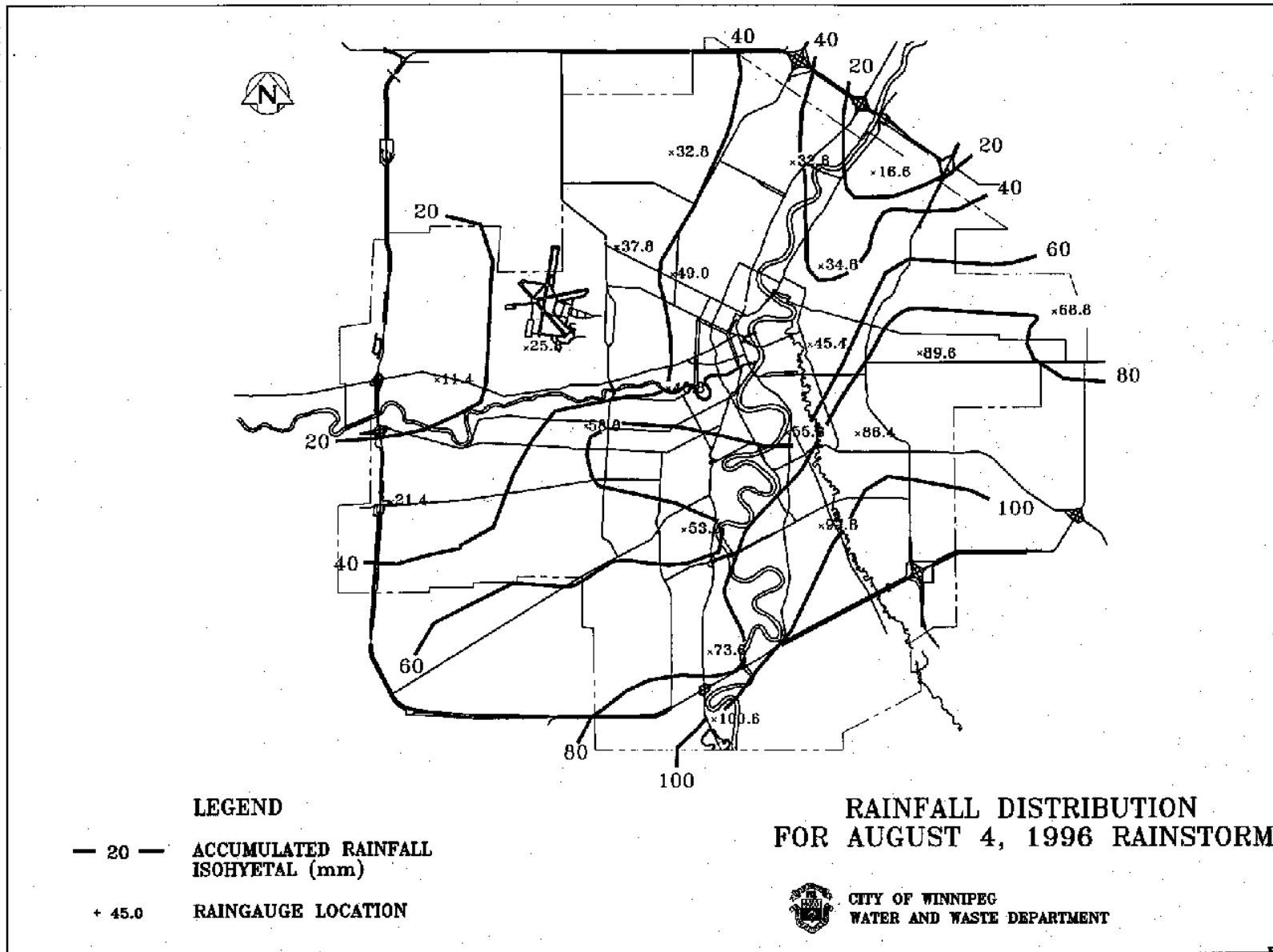


Figure 4.4 – Rainfall Data From Continuous Recording Stations

4.4 XPSWMM MODEL CALIBRATION

Calibration and verification of the SWMM model is necessary to develop a high degree of confidence in predictions. The 1978 model had been calibrated and used to determine the upgrades to the combined sewer system to achieve the required level of basement flood protection (i.e., five year). However, changes in district hydrology have occurred that necessitated recalibration of the model to ensure accurate simulation of system hydraulics for the current conditions.

The XPSWMM model was calibrated to monitoring data collected during the summer of 1996. Sewer monitoring data was collected at four locations: three locations on the combined trunk sewer and one location on the storm relief sewer. Figure 4.3 shows the water level monitoring locations along the combined sewer and storm relief sewer trunks. Continuous rainfall data was obtained from the City's network of fixed and portable rainfall gauge stations (Figure 4.4).

The first step in the calibration exercise involved the review of the City's isohyets, (accumulated rainfall contours) prepared for each significant rainfall event from the City's network of recording rain gauges. This was done to identify storm events that had a consistent rainfall pattern over the district, or as close as possible. Storms with a highly variable rainfall pattern over the district were rejected. Rainfall data for the selected rain events were reviewed and the most representative data was selected for the calibration exercise. This extensive screening was used to select rainfall data representative of the actual rainfall pattern over the district sewers.

The sewer monitoring program provided depth versus time data. The depths were entered into a spreadsheet and converted to geodetic elevations. This data was compared to the rainfall data as part of the initial screening process. Any corrupt or questionable data from the sewer monitors that did not match the rainfall pattern was discarded to conserve on modelling effort and subsequent analysis of results.

In the initial calibration works, the XPSWMM model consistently over predicted the volume of runoff from both small and large storm events. Adjustment of the usual calibration

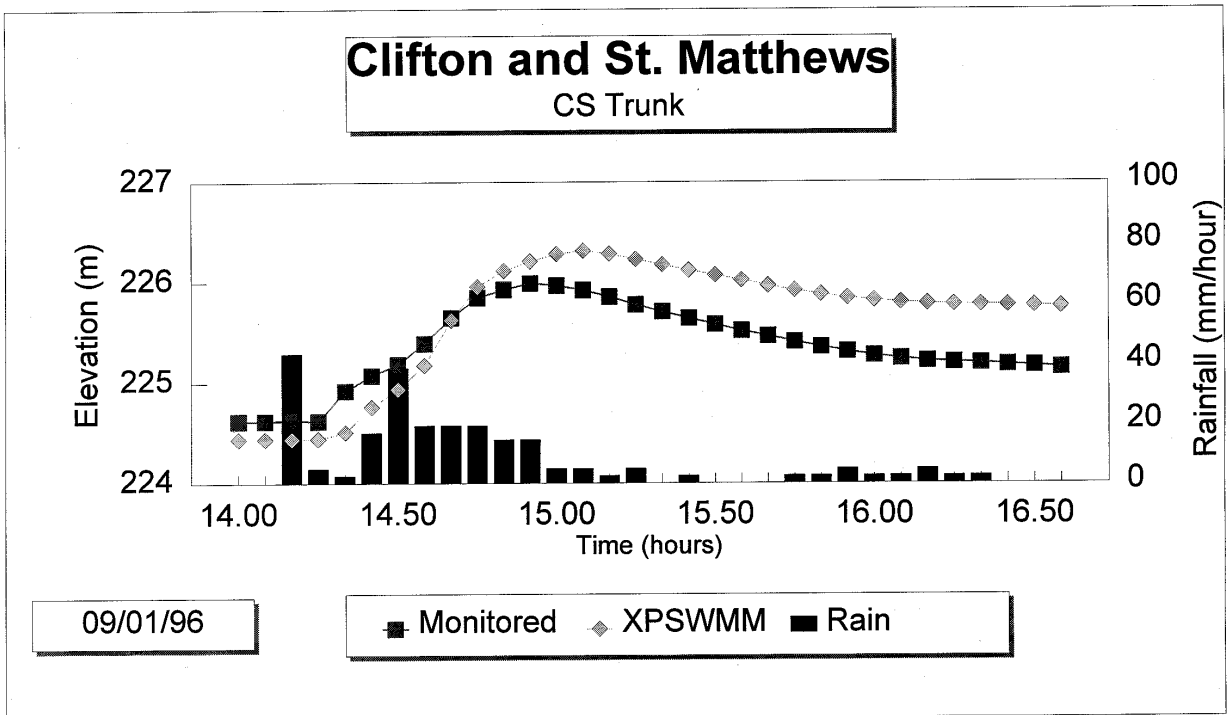
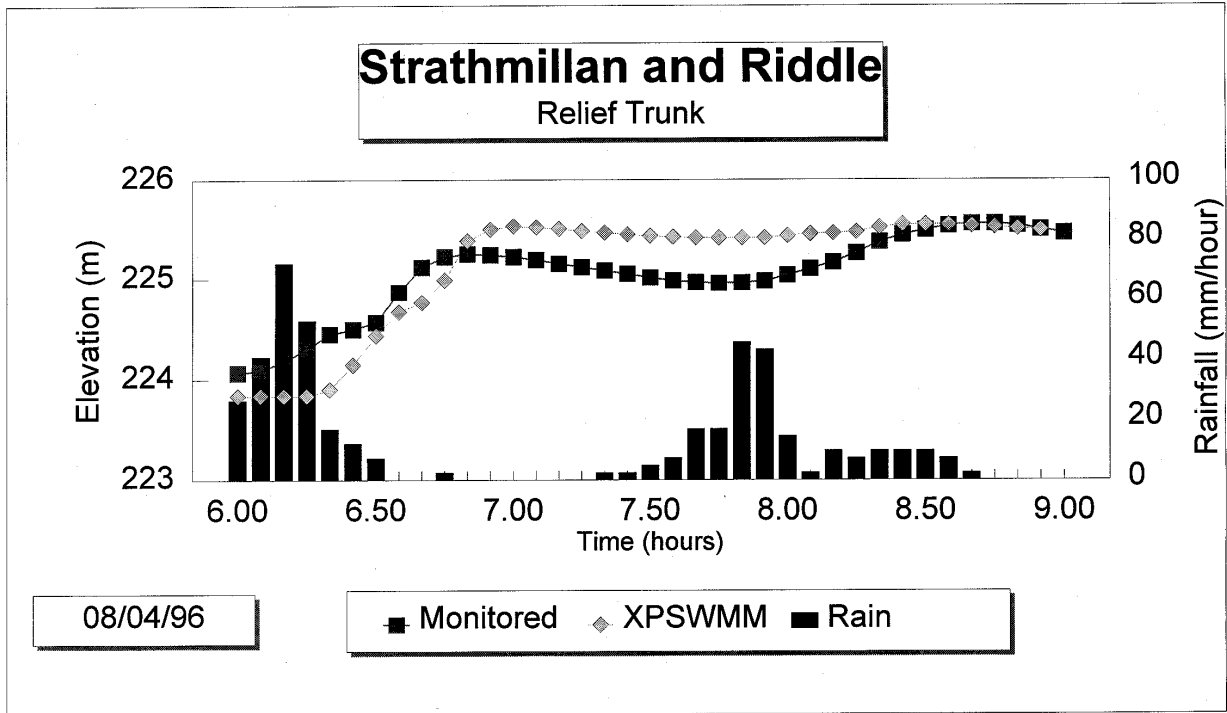


Figure 4.5 – Comparison of Level Monitoring and Model Predictions

Clifton District - Inline Storage Volumes

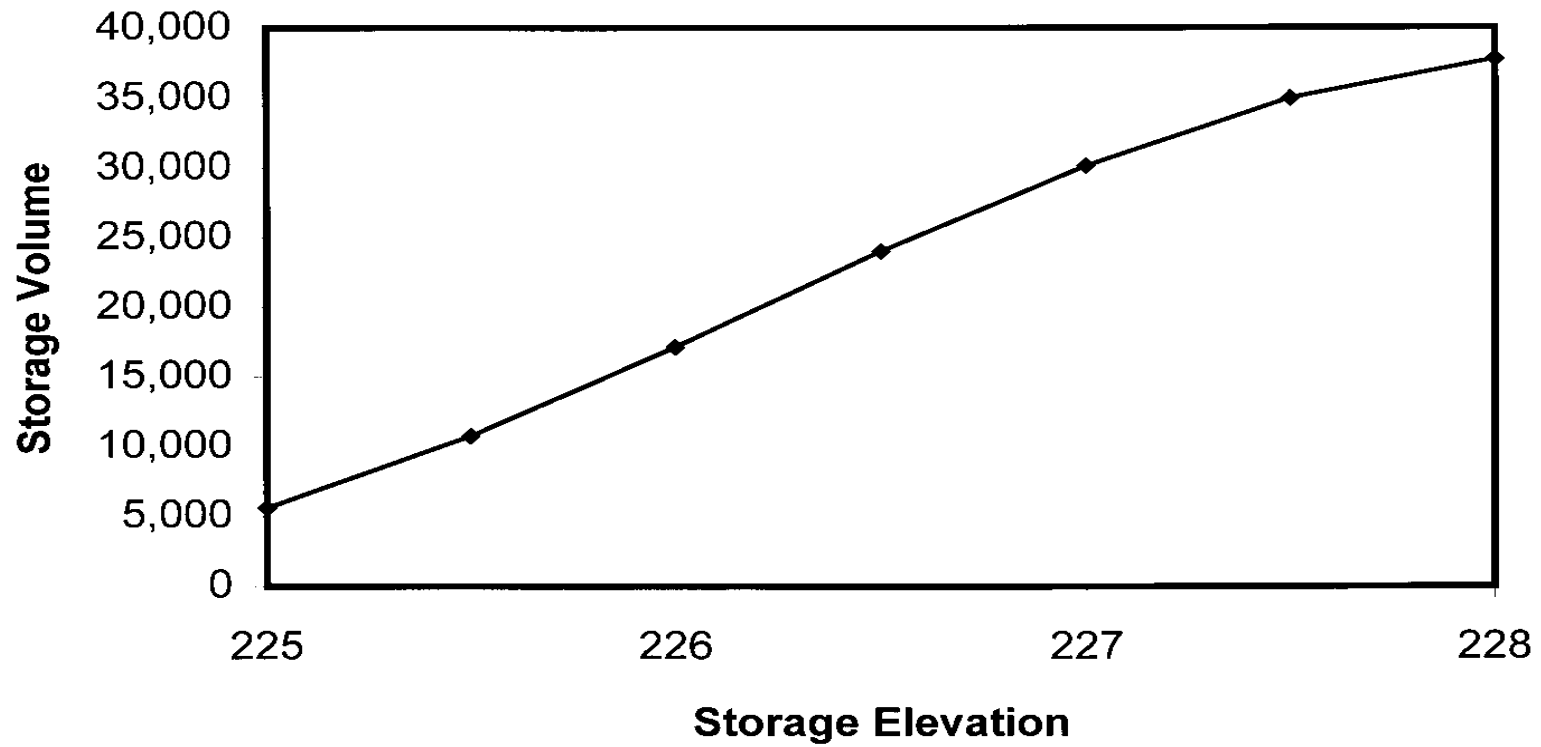


Figure 4.6 – Available Inline Storage Under a Given Elevation

parameters (i.e., width, infiltration, detention storage) within reasonable limits did not noticeably improve the calibration results.

In estimating the volume of runoff, the model is most sensitive to, in order of importance, rainfall and percent imperviousness. The initial screening process provided confidence that the rainfall data being used was representative of the actual rainfall on the district, therefore, it was concluded that the discrepancy in runoff volume was a function of subcatchment imperviousness. Unfortunately, it was not known how imperviousness had been calculated in the original model; if this parameter had been adjusted in the original calibration; or how the connected downspouts had been considered. Re-calibration was deemed necessary and involved resetting the remainder of the subcatchment parameters to the original values and adjusting the percent imperviousness.

The impervious area of each subcatchment was initially reduced by 5% (e.g., a subcatchment with 40% impervious area was reduced to 38%, not 35%) and then in 5% increments. The subsequent analysis indicated that the best match to the monitored data occurred with a 10% reduction in the impervious area, or an overall district imperviousness of 40%. Figure 4.5 shows a sample comparison of measured levels and the model's predictions for two different events. Based on the results from the calibration exercise, it was concluded that the model was representative of wet weather flow conditions in the Clifton district and could be used with confidence for the inline storage analysis.

4.5 INLINE STORAGE ANALYSIS

An analysis was performed on the existing sewer system to determine the amount of storage in the combined and storm relief sewers that would be available at varying elevations. The results of this analysis are shown of Figure 4.6 and indicate the available storage volume below a given elevation (for reference the average ground elevation in the district is approximately 233.0 and the lowest ground elevation is 231.2). This indicates the benefits from increasing the storage depth; however, it was also understood that increasing the depth increases the risk of basement flooding.

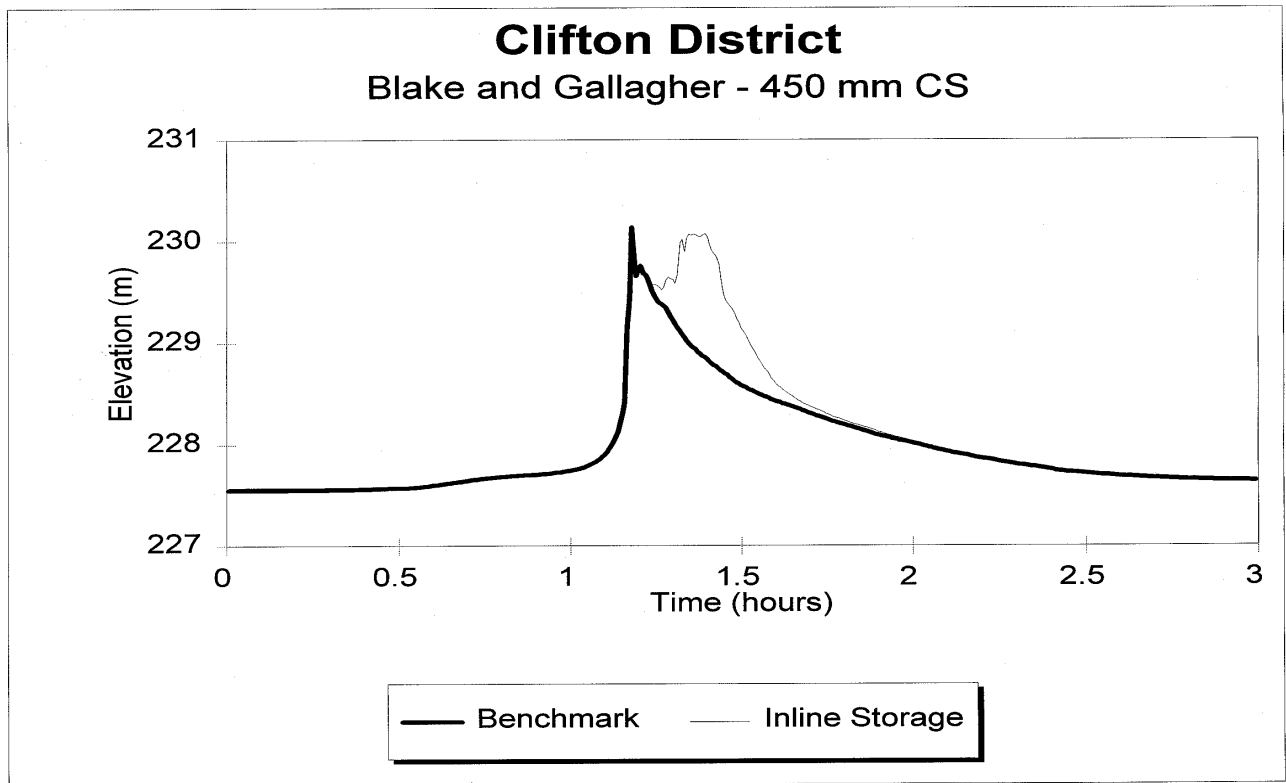
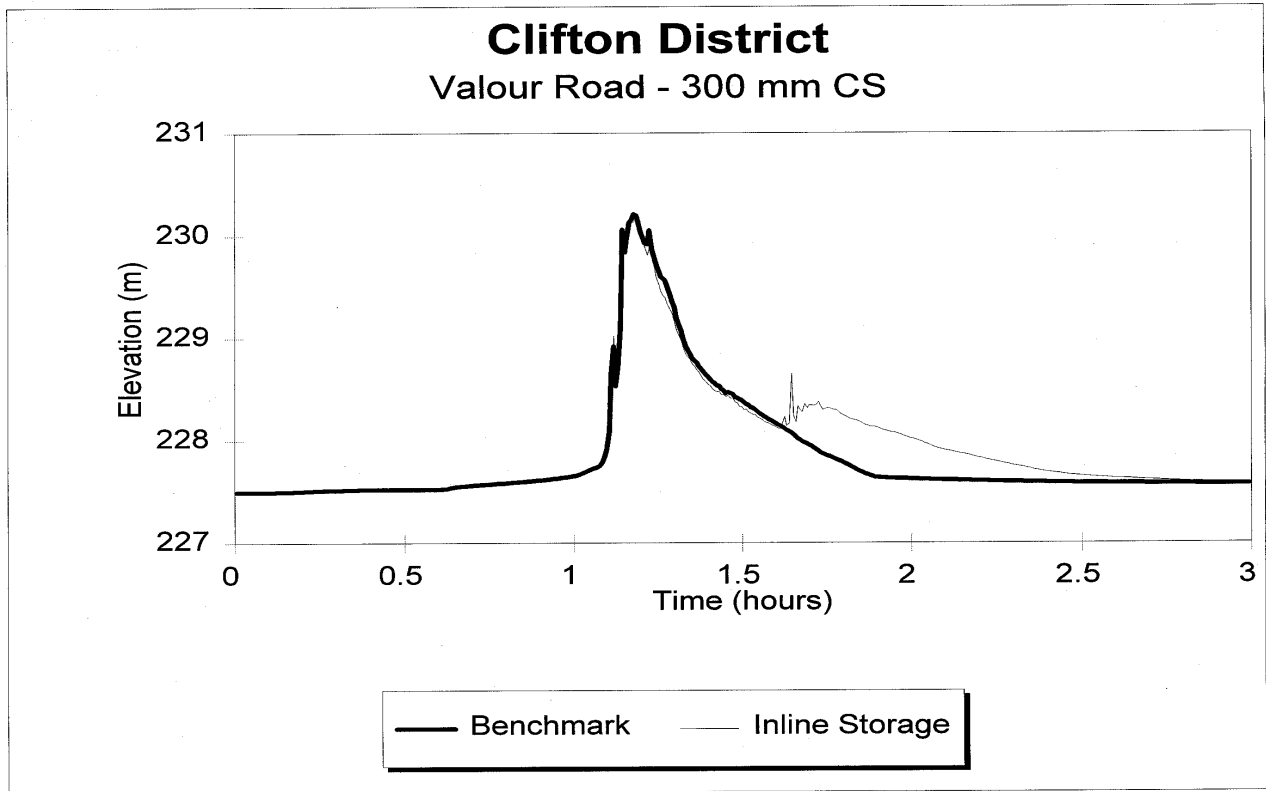


Figure 4.7 – Comparison of Levels With and Without Inline Storage

The City of Winnipeg five-year design storm was routed through the model for existing conditions. This was carried out to provide a benchmark condition to assess the impact of inline storage on basement flood protection (level of service). The comparison of inline storage model results included the assessment of hydraulic response in the trunk sewer, but would focus mainly on the upstream lateral sewers. To maintain the existing level of service, it was deemed prudent that not only should the inline storage not cause basement flooding, it should not raise levels in the lateral sewer systems higher than they would originally have been.

The inline storage analysis was conceptually designed based on motorized regulator gates at the combined sewer and storm relief sewer outfalls. These gates were modelled with the regulator link option in XPSWMM that controls the flow through the link or conduit. The regulator link uses water depth at a given control node as the operating protocol for the gate. The user is required to select the node that controls the flow and to specify the operating rules for the gate. The operating rules are tabulated and contain depths and corresponding multipliers that are applied to the link with the regulator. A multiplier of zero represents zero flow through the conduit and is consistent with a regulator gate in the fully closed position. A multiplier of one would correspond to a wide-open condition with no artificial control on the flow through the conduit. In this manner the gate opening and closing protocol was approximated.

The design storm event was routed through the model with controls on the combined sewer and SRS outlets to the river. The control node data was adjusted for each simulation to allow the water level to reach a target elevation before opening the gate. Initially, the gate was set up to open when levels reached an elevation of 228.0 m at the control node. Simulation results indicated the potential for basement flooding and that lateral sewers would experience higher surcharge levels than originally predicted as part of the basement flood protection analysis. Simulations were rerun with the gate opening elevation reduced in 0.25 m increments until the peak levels in the lateral sewers closely matched the levels from the benchmark run and no basement flooding was predicted. From this analysis, it was determined that an activation elevation of 227.0 m was the appropriate control elevation to safely utilize available inline storage. A comparison of the predicted levels to the levels from the benchmark simulation for selected locations are shown on Figure 4.7.

This data indicates that the peak flow passes through the lateral sewer before the peak levels are attained in the trunk sewers. A static water level of 227.0 in the Clifton sewer system represents approximately 31,000 m³ of inline storage.

The above analysis was based on the design event occurring when the system was empty. However, given the large volume of storage and the relatively low interception rate of 236 L/sec., it would take an extended period of time to fully dewater the system. This raised a concern as to how the system would react to back-to-back storm events and whether this type of event would have a negative impact on basement flooding. (As noted earlier, the primary concern with implementing inline storage was the impact on basement flood protection.) Therefore, to ensure that the level of basement flood protection would not be compromised, the system was analyzed under a more severe condition. This entailed having the design event occur when the system was full.

With an initial water level in the system set equal to 227.0 m, it was found that the system could not discharge the design storm without compromising basement flood protection. It was apparent from the model results that the sewer system could not be emptied fast enough to handle the peak flows entering the system. Additional computer simulations with the XPSWMM model indicated that the storage level in the system had to be reduced to an elevation of 226.0 m to convey the design event runoff starting from a full system and not compromise the existing level of basement flood protection. This condition represents approximately 17,000 m³ of inline storage in the sewer system. While this is still a significant amount, it amounts to 45% less than the 31,000 m³ available at elevation 227.0.

4.6 BENEFITS ON INLET CONTROLS

The long-term goal of the basement flood relief program is to upgrade the sewer system to a ten-year level of service through the implementation of catchbasin inlet controls. Such controls restrict flow to the sewer system. Flows in excess of the capacity of the inlet control are temporarily stored on the surface.

Subcatchment 220

1.32 ha. @ 30.5% Impervious

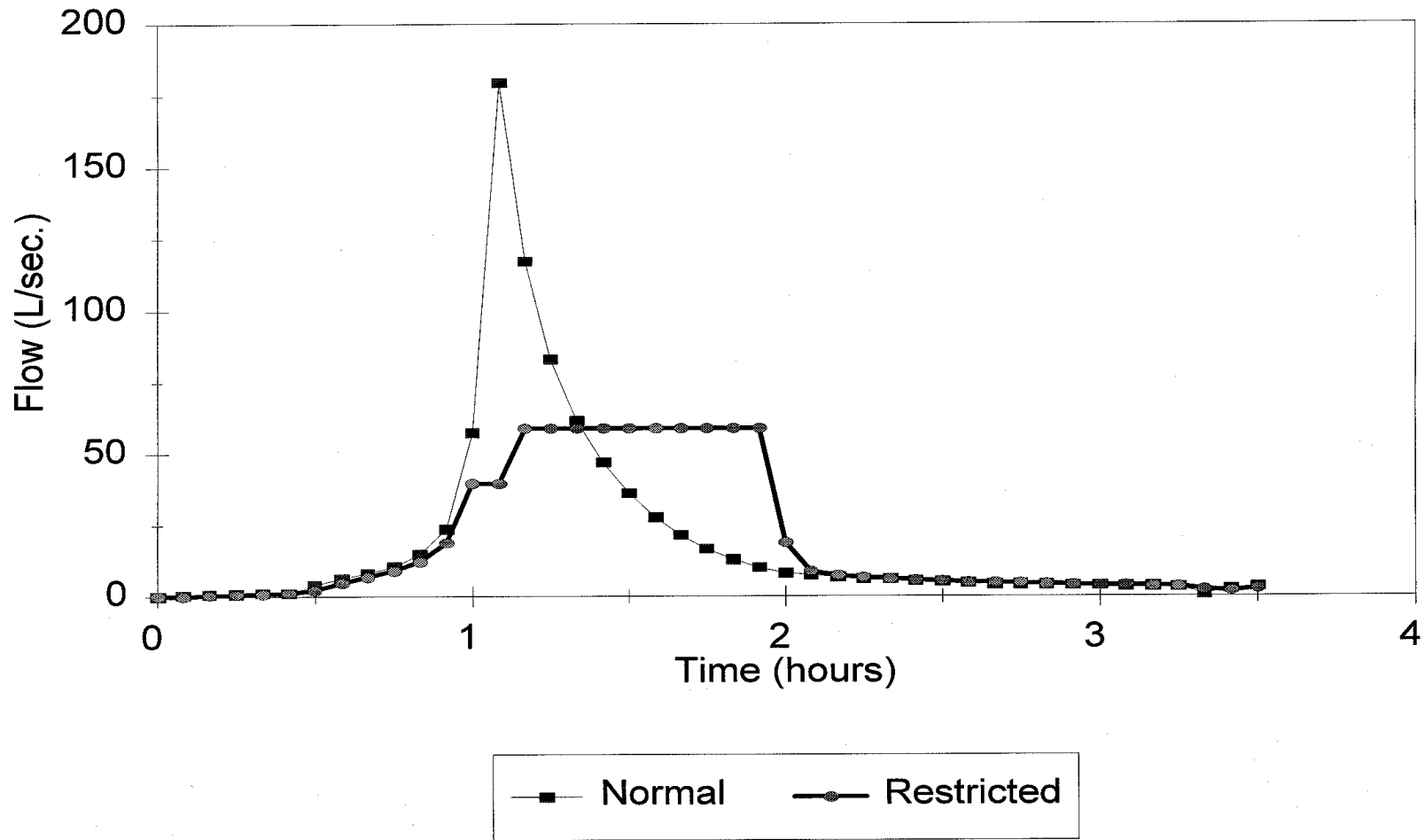


Figure 4.8 – Effect of Flow Restriction to Runoff from Typical Residential Subcatchment

CLIFTON COMBINED TRUNK SEWER

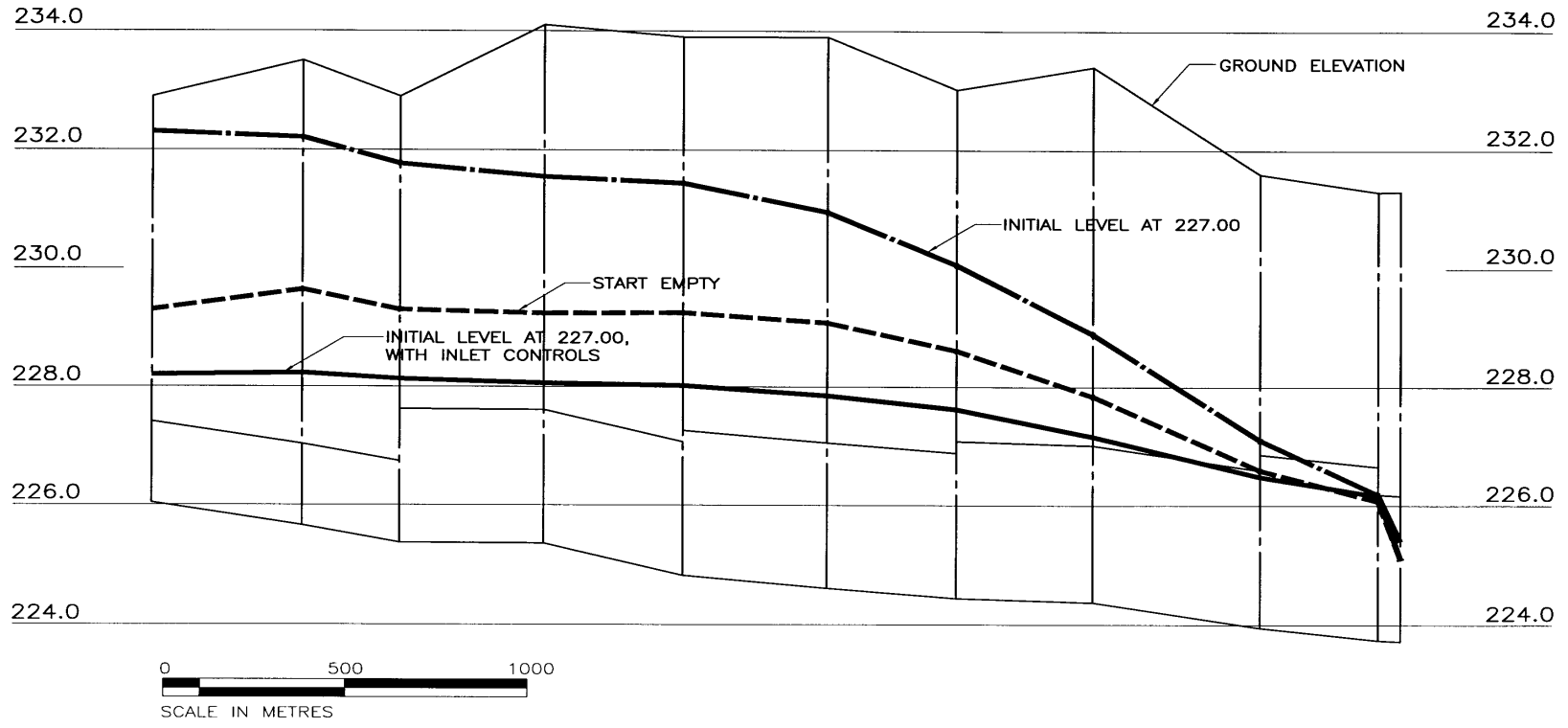


Figure 4.9 – Hydraulic Grade Line Comparison

The inlet control analysis was based on the assumption that flows to the sewer system would be restricted to 28 L/sec per inlet. The allowable peak flow from each subcatchment was calculated by counting the number of inlets within the area and multiplying by the peak flow rate. The restricted flows were modelled using channel pipes in XPSWMM Runoff. The effect of this restriction on flows for the five-year design event from a typical residential subcatchment in the Clifton District is shown on Figure 4.8.

Channel pipes were added to the XPSWMM model to assess the impact of inlet controls in conjunction with inline storage. The initial conditions for the simulation were identical to the initial trial to review the back-to-back event, i.e., the system full and the gate set to open at elevation 227.0. The five-year design storm was then routed through the model.

The results of the latter analysis indicated that inlet controls have a significant impact on reducing levels in the sewer system. Restricting the peak flows to the system results in less flow in the upstream sewers and allows the accumulated storage to be discharged from the trunk sewers. As expected, inlet controls resulted in lower levels in the system than were predicted for the five-year event in conjunction with an empty system. Figure 4.9 shows peak hydraulic grade line plots for the following conditions:

- System empty, gates start to open at 227.0.
- System full, gates start to open at 226.0.
- System full, inlet controls included and gates start to open at 227.0.

The inlet control analysis was conceptual in nature and no attempt was made to determine if the required volume of surface storage was available in any of the subcatchments. It is anticipated that the restricted flows would have to be increased in many areas, particularly in the highly impervious commercial areas to prevent exceeding the available surface storage. It is recognized that inlet controls must be designed on a site-specific basis to account for the site topography and the conveyance capacity of the local sewer system. Accordingly, the analysis represents a best case scenario and the actual benefits that could be realized through inlet controls would be less. Nonetheless, it is clear, that inlet controls are an important method of maintaining basement flood protection in conjunction with inline

storage in combined sewer systems and should be an integral part of Winnipeg's CSO control plan where inline storage will be utilized.

4.7 CONCLUSIONS

The calibrated XPSWMM model of the Clifton District Combined Sewer system provided a valuable analytical tool for the analysis of inline storage. The regulator link facility allows the approximation of actual control devices such as gates or inflatable dams. This facility allowed the review of inline storage and defined the amount of in-system storage that could be accessed without compromising the existing level of basement flood protection.

The results of this analysis indicate that there is significant potential for inline storage in the City of Winnipeg sewer systems as a result of the area's topography. This is particularly the case in areas such as the Clifton District where storm relief sewers have been constructed to upgrade the level of basement flood protection.

While the impact of inlet controls was not reviewed in detail, the conceptual analysis indicated that these controls can help mitigate basement flooding in sewer systems modified for inline storage. Furthermore, inlet controls can increase the amount of allowable storage in the sewer system, thereby further reducing CSO's to the rivers. On this basis it is recommended that inlet controls also be considered for any potential inline storage project to increase the pollution control benefits and help maintain basement flood protection.

5 HART DISTRICT

The previous analysis (Section 4.0 Clifton District) was carried out as a prerequisite to implementing an inline storage pilot program in the Clifton District. Unfortunately, there were several obstacles that would not permit the implementation of inline storage in the district in a timely manner. This led to the abandonment of plans to institute an inline storage pilot project in the Clifton Combined Sewer District. Further investigation of the combined sewer districts revealed that the Hart District was also a suitable candidate for pilot testing.

5.1 OVERVIEW

The Hart Combined Sewer District was previously modelled as part of the Basement Flood Relief (BFR) program. The work was carried out with SWMM Version 2. The model was built, calibrated and used for the conceptual and detailed design of storm relief sewers (SRS). The SRS were installed to provide a minimum five-year level of basement flood protection. The SRS, along with the existing egg-shape combined trunk sewer represents a significant amount of inline storage.

As in the Clifton analysis, modelling was required to assess the feasibility and operating characteristics of the proposed inline storage works in conjunction with real and synthetic storm events. The modelling was initially configured to follow three separate workstreams:

- **Calibration:** the original modelling was carried out with SWMM2. The inline storage analysis required the regulator link facility in XPSWMM. Therefore, the model had to be upgraded to SWMM4 format before being imported to the XP environment. This also required another calibration effort because of changes to the SWMM source code.
- **Inlet Controls:** the Clifton District analysis indicated the significant benefits of implementing inlet controls with inline storage. It was understood that inlet controls would be an integral part of the implementation works for the inline storage pilot test.

Inlet controls (in the form of 100 mm diameter pipe inserts in the catchbasin leads) had previously been installed in the Hart District to upgrade basement flood protection. It was believed that the controls had been installed on a district wide basis prior to 1980. It was not known if any of the controls had been removed with street renewal works, catchbasin steaming, etc. Therefore, a field survey was required to determine the size and location of the existing inlet controls.

- Inline Storage and Gate Control Protocol: this analysis would assess safe operating protocol for the proposed motorized gate to ensure that the existing level of basement flood protection would be maintained. This would also determine the volume of inline storage that could be safely accessed. This information would then be used, in conjunction with a continuous simulation, to determine the number and volume of overflows during a “typical” year.

The proposed inline storage pilot test for the Hart District was cancelled during the calibration works; however, it was decided that the model calibration work should be completed. In addition, the field investigations of the existing inlet controls were completed prior to the cancellation of the project.

5.2 MODEL SET-UP AND DEVELOPMENT

The original SWMM2 model files were retrieved from Wardrop’s archives. The models included:

- The original model used for the calibration works, i.e., pre-relief with inlet controls.
- The detailed design model including the storm relief sewer (SRS) piping based on a five-year storm event without inlet controls and the partial separation of the Stadacona area.
- The detailed model used for the ten-year analysis, i.e., five-year SRS piping and existing inlet controls.

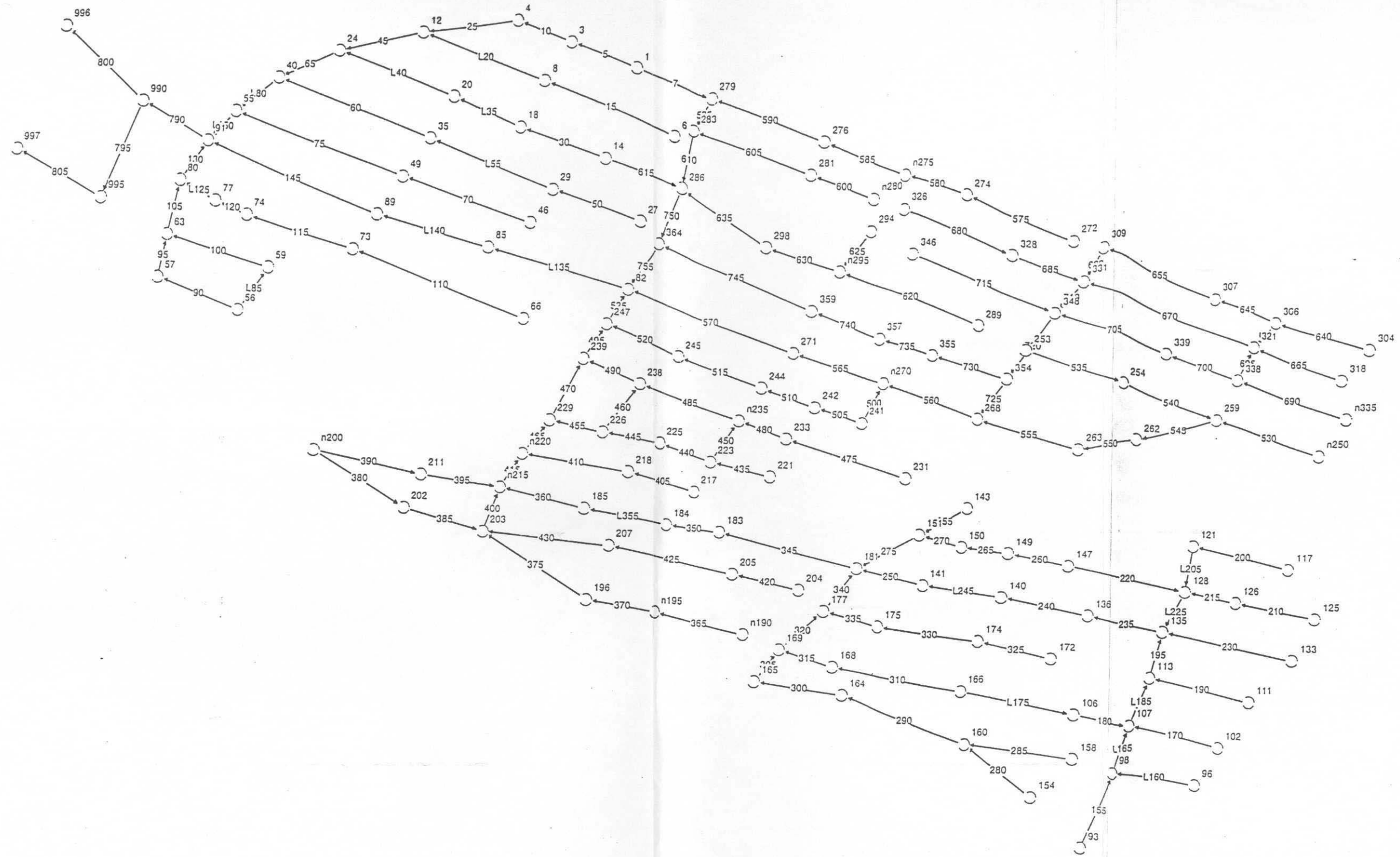


Figure 5.1

The SWMM 2 data files were revised to SWMM4 format and then imported to XPSWMM. Some minor changes were made to the node numbering system to correspond to the XP environment conventions. A schematic of the node and piping layout of the pre-relief conditions is shown on Figure 5.1.

5.3 CALIBRATION

It was initially thought that all of the original calibration data, rainfall and monitored sewer levels, would be available for this exercise. Unfortunately, this data could not be located. Therefore the available data was limited to what was contained in the Conceptual Design Report (Wardrop, 1985). This rainfall and sewer level data consisted of two events, August 5 and August 17, 1981. This consisted of time and level data for five locations and a number of approximate peak water levels in the system (from jar ladder data).

Significant effort was expended calibrating the SWMM4 model to the 1981 storm data. This included the adjustment of subcatchment values for percent impervious, width, and detention storage for pervious and impervious areas. While the model could be manipulated to closely match the monitored levels in one area, this would result in poor results in another area. In particular, it was extremely difficult to match the monitored levels in the Stadacona area without drastically adjusting the parameter values. This was notable in that a significant amount of the Stadacona area had been separated to land drainage sewers during the relief works. At this point, it was apparent that the calibration was hindered by the limited data and a significant effort would be required trying to calibrate the model to an area that was no longer connected to the combined sewer system. As such, it was deemed futile to continue with this calibration effort and more productive to concentrate on calibrating the model to the five-year event hydrographs calculated by the SWMM2 design model.

The initial simulation used the identical subcatchment parameters as the SWMM2 model. This resulted in an over estimation of the peak flow from the catchment as well as the total runoff volume. To calibrate the model a number of subcatchments were selected for

Table 5.1
SWMM2 Runoff Parameters for Selected Subcatchments

Subcatchment	4	29	185	207	245	254
Area (ac.)	3.7	5.75	5.43	1.89	3.39	2.35
Width (ft.)	1080	1926	1764	990	1206	720
% Impervious	30	30	50	50	50	30
Slope	.0025	.0025	.0025	.0025	.0025	.0025
Impervious "n"	.02	.02	.02	.02	.02	.02
Pervious "n"	.25	.25	.25	.25	.25	.25
Impervious Storage (in.)	.4	.3	.3	.2	.3	.4
Pervious Storage (in.)	.7	.5	.5	1	.5	1
Maximum Infiltration (in./hr)	2	2	2	2	2	2
Minimum Infiltration (in./hr)	.1	.1	.1	.1	.1	.1

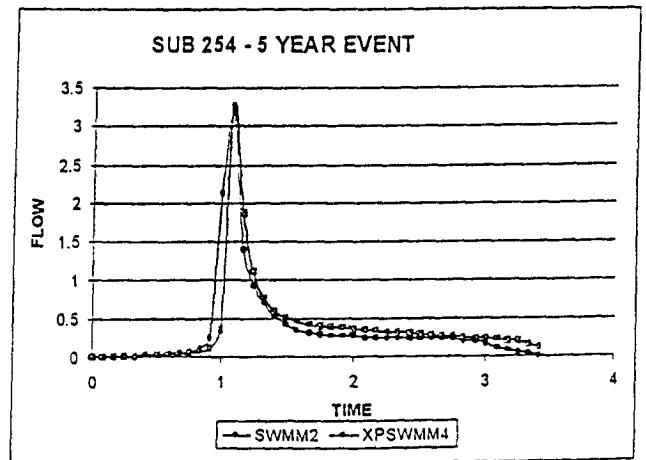
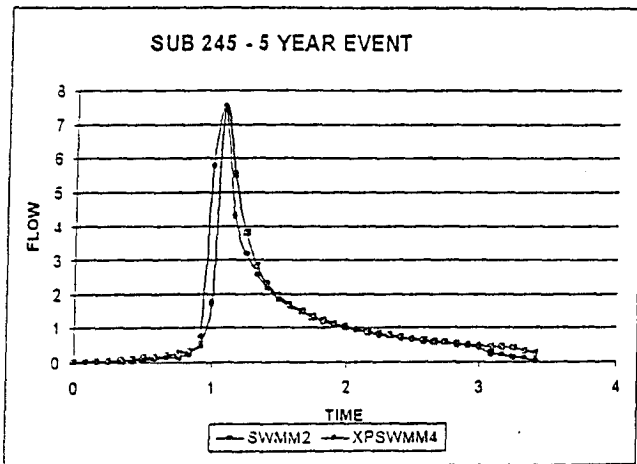
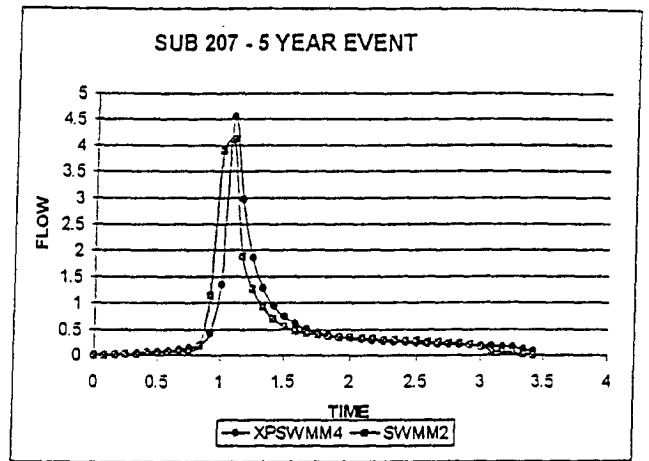
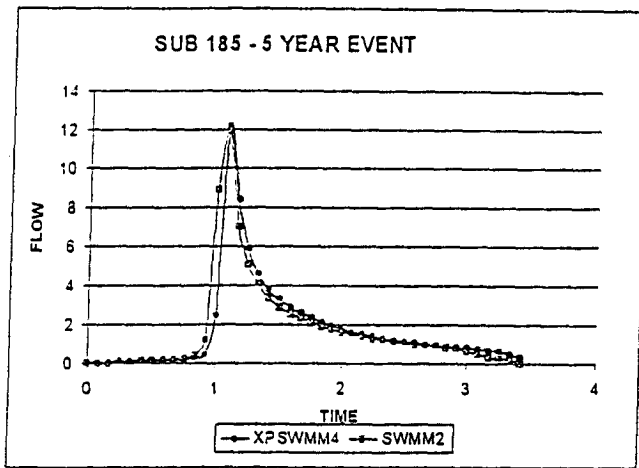
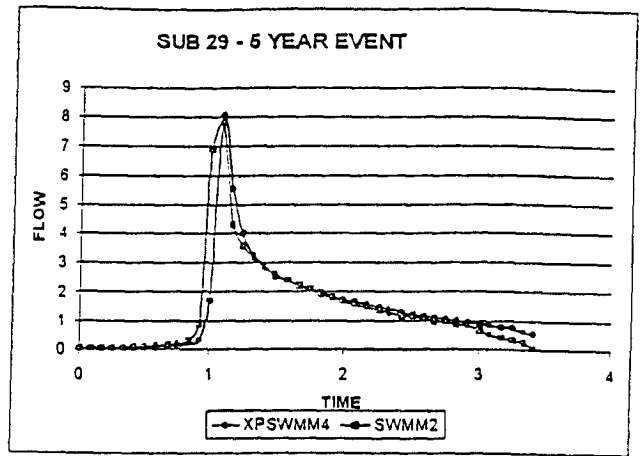
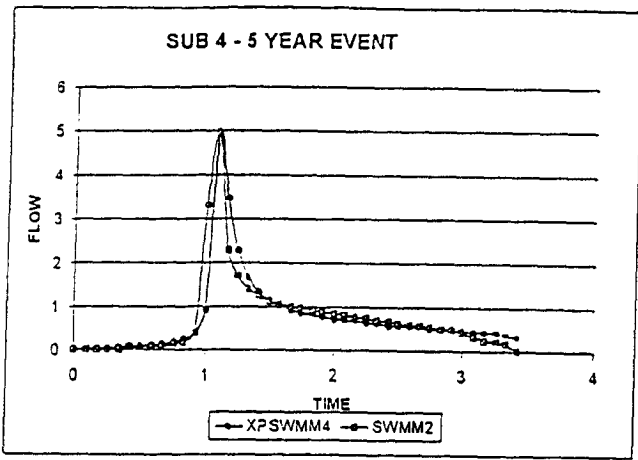


Figure 5.2

detailed comparison. These subcatchments and the associated SWMM2 runoff parameters are shown in Table 5.1.

The original SWMM2 runoff parameters were adjusted for the SWMM4 calibration exercise. This included:

- Reducing the percent imperviousness of the subcatchment.
- Adjusting the width.
- Adjusting the amount of impervious area with zero detention storage.
- Adjusting the pervious detention storage depth.
- Adjusting the maximum infiltration rate.

The parameters from each of the selected subcatchments were adjusted until the SWMM4 predicted peak flow and runoff volume closely matched the SWMM2 hydrograph. Then the individual adjustments were reviewed to determine a common adjustment that could be applied on a district wide basis. This review resulted in the following revisions to the Hart District SWMM2 runoff parameters for the SWMM4 analysis:

- Width * 0.75
- Percent Impervious * 0.9
- Pervious detention storage * 0.75
- Impervious Area with zero detention storage = 25%

The five-year event was routed through the SWMM4 model and the results compared to the SWMM2 analysis. The comparison of the runoff hydrographs for the selected subcatchments is shown on Figure 5.2. The district wide volume comparison is shown in the following table.

	SWMM2 (ft³)	SWMM4 (ft³)
Precipitation	1,727,280	1,727,289
Infiltration	613,852	697,907
Runoff	523,092	537,731
Water remaining in Surface Storage	572,010	479,570

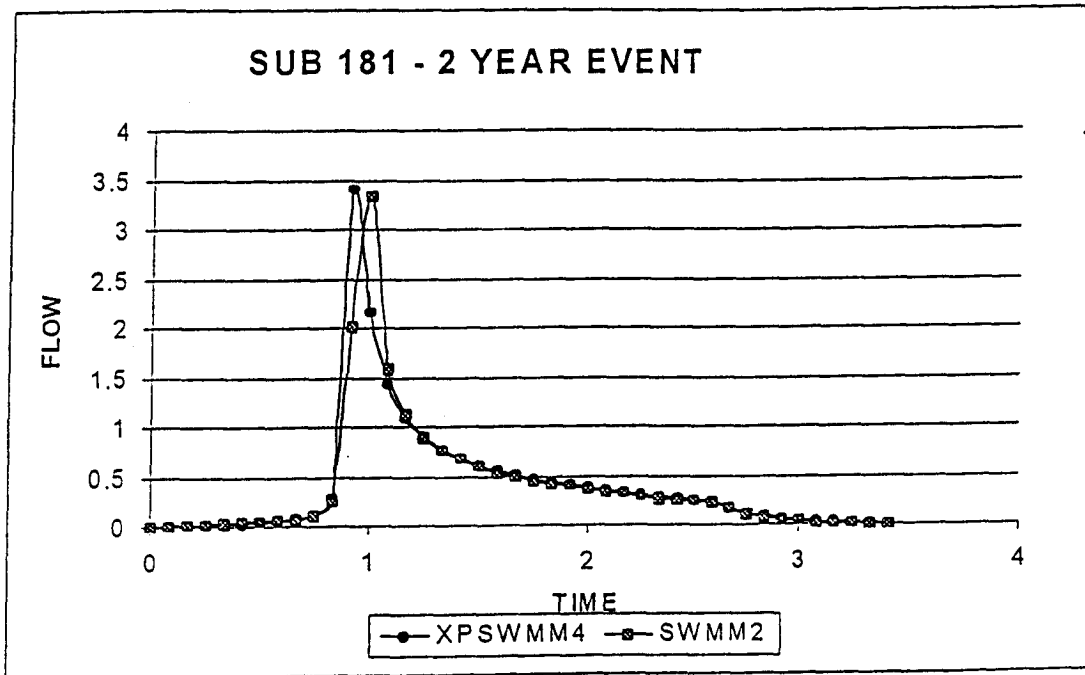
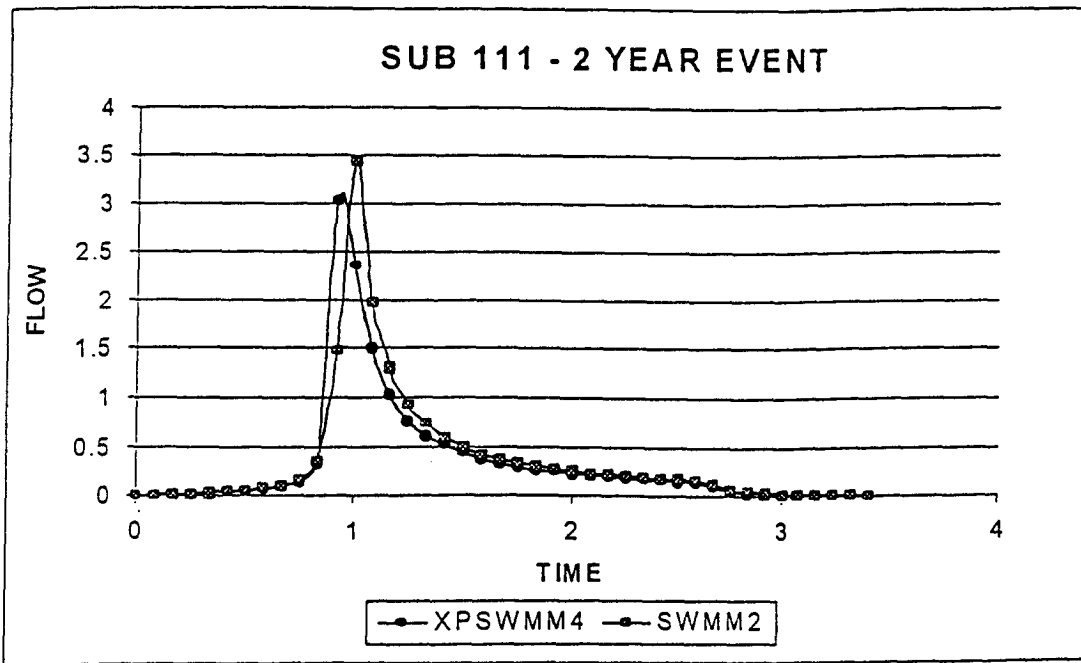


Figure 5.3

Table 5.2
Hart District - SWMM2 VS SWMM4
Water Level Comparison
5year Event

JUNCTION NUMBER	SWMM2	SWMM4	DIFFERENCE S4-S2	JUNCTION NUMBER	SWMM2	SWMM4	DIFFERENCE S4-S2	JUNCTION NUMBER	SWMM2	SWMM4	DIFFERENCE S4-S2	JUNCTION NUMBER	SWMM2	SWMM4	DIFFERENCE S4-S2
1	743.86	744.49	0.63	136	745.5	756.07	10.57	235	745	745.9	0.9	359	741.45	741.58	0.13
3	741.8	741.92	0.12	140	744.17	757.17	13	238	743.2	745.77	2.57	364	740.23	740.44	0.21
4	740.27	741.56	1.29	141	744.05	757.5	13.45	239	740.59	743.3	2.71	990	737.29	737.64	0.35
6	744.62	744.61	-0.01	143	746.86	746.48	-0.38	241	743.97	743.97	0	995	714.23	711.68	-2.55
8	741.17	741.54	0.37	147	749.01	749.31	0.3	242	742.59	742.4	-0.19	996	734.12	734	-0.12
12	739.47	740.49	1.02	149	746.99	747.25	0.26	244	740.55	740.7	0.15	997	714.22	711.67	-2.55
14	743.88	743.91	0.03	150	745.73	747.61	1.88	245	740.46	743.55	3.09	700	740.64	740.81	0.17
18	740.27	740.37	0.1	151	744.9	746.77	1.87	247	739.8	742.55	2.75	705	739.64	739.85	0.21
20	739.45	739.96	0.51	154	749.17	749.35	0.18	250	746.09	745.99	-0.1	701	742.9	742.57	-0.33
24	738.82	739.68	0.86	158	747.52	748.09	0.57	253	748.36	748.35	-0.01	710	747.86	749.69	1.83
27	744.47	744.47	0	160	745.83	757.2	11.37	254	746.87	746.9	0.03	715	747.11	748.59	1.48
29	741.95	742.02	0.07	164	744.31	753.07	8.76	259	739.76	739.77	0.01	720	744.77	746.71	1.94
35	739.7	739.63	-0.07	165	744.62	754.77	10.15	263	739.71	739.74	0.03	725	741.2	741.52	0.32
40	738.29	738.78	0.49	166	749.13	749.49	0.36	268	739.65	739.7	0.05	730	740.85	741.11	0.26
46	740.82	740.96	0.14	168	745.56	753.8	8.24	270	739.5	739.63	0.13	751	745.9	753.77	7.87
49	738.85	738.93	0.08	169	744.55	753.97	9.42	271	739.37	739.56	0.19	752	742.07	757.87	15.8
55	738.12	738.55	0.43	172	745.74	748.11	2.37	272	751.55	751.69	0.14	753	740.79	744.19	3.4
56	744.13	744.25	0.12	174	744.59	758.17	13.58	274	748.68	750.58	1.9	754	739.93	740.24	0.31
57	741.77	749.27	7.5	175	744.19	757.47	13.28	275	747.96	749.97	2.01	1143	744.16	747.61	3.45
59	741.98	742.19	0.21	177	744.3	756.97	12.67	276	745	748.11	3.11	800	739.2	739.84	0.64
63	741.37	741.91	0.54	181	743.94	748.8	4.86	279	743.51	745.2	1.69	805	738.99	739.78	0.79
66	745.89	746.02	0.13	183	743.74	752.67	8.93	280	750.07	750.47	0.4	806	738.62	739.79	1.17
67	745.01	745.01	0	184	743.58	755.37	11.79	281	748.75	750.73	1.98	807	738.3	738.79	0.49
68	743.99	744.06	0.07	185	743.23	755.47	12.24	283	743.29	744.62	1.33	810	738.22	738.58	0.36
73	741.56	741.5	-0.06	190	744.38	752.5	8.12	286	741.13	741.52	0.39	815	738.47	738.78	0.31
74	740.22	740.32	0.1	195	744.12	747.6	3.48	289	747.48	747.52	0.04	820	739.92	740.07	0.15
77	739.54	740.27	0.73	196	744.03	747.51	3.48	294	743.75	743.77	0.02	8510	740.05	742.72	2.67
80	739.33	739.77	0.44	200	744.78	748.54	3.76	295	741.8	741.83	0.03	811	739.54	740.56	1.02
82	738.98	739.19	0.21	202	744.48	748.19	3.71	298	741.77	742.44	0.67	878	740.15	741.77	1.62
85	738.57	738.83	0.26	203	743.18	746.36	3.18	304	747.38	747.47	0.09	818	739.91	741.91	2
89	738.17	738.48	0.31	204	746.47	747.46	0.99	306	744.24	746.49	2.25	829	740.95	741.64	0.69
91	737.8	738.2	0.4	205	745.04	750.89	5.85	307	743.76	748.15	4.39	846	738.74	740.71	1.97
93	747.97	758.97	11	206	743.9	751.49	7.59	309	742.78	751.68	8.9	859	739.48	740.16	0.68
96	748.37	758.57	10.2	207	743.78	747.82	4.04	318	744.25	744.88	0.63	966	742.74	743.24	0.5
98	747.74	757.67	9.93	208	745.77	751.3	5.53	321	742.86	757.57	14.71	9200	744.58	748.35	3.77
102	747.53	756.07	8.54	211	743.37	747.01	3.64	326	748.06	748.09	0.03	9205	744.58	750.47	5.89
106	747.5	753.34	5.84	215	742.67	745.57	2.9	328	745.18	758.17	12.99	9217	744.28	747.88	3.6
107	747.46	753.52	6.06	217	746.1	748.38	2.28	331	742.74	748.37	5.63	9555	743.36	746.98	3.62
111	749.38	756.22	6.84	218	743.93	747.04	3.11	335	747.33	747.42	0.09	9195	743.93	748.42	4.49
113	746.91	754.34	7.43	220	742.26	745.16	2.9	338	744.39	744.72	0.33				
117	746.64	757.87	11.23	221	744.89	745.05	0.16	339	744.35	747.21	2.86				
121	746.67	757.97	11.3	223	745.14	747.09	1.95	346	747.58	747.74	0.16				
125	747.58	754.74	7.16	225	743.4	745.41	2.01	348	742.29	745.56	3.27				
126	746.65	753.81	7.16	226	743.05	745.41	2.36	350	741.63	741.45	-0.18				
128	746.56	753.53	6.97	229	741.78	744.64	2.86	354	748.04	746.79	-1.25				
133	746.71	757.07	10.36	231	748.36	748.34	-0.02	355	746.36	747.41	1.05				
135	746.17	754.44	8.27	233	744.03	744.22	0.19	357	743.04	743.06	0.02				

This indicates that the revised SWMM4 model is representative of the SWMM2 model's hydrographs for the two-year event.

The two-year runoff hydrographs were then routed through the SWMM2 and the SWMM4 models. The maximum water levels predicted by each model were reviewed and compared. Table 5.2 shows the peak water levels from the models and the difference in the levels. The table also indicates whether the difference was within 1 ft, and if not, which model predicted the higher level. The final column indicates if the node was surcharged.

The results indicate, that for the majority of the district, the models predict nearly identical peak water levels. It is also noted that surcharging was evident for each location where the SWMM4 levels were over 1 foot higher than the SWMM2 levels. This analysis supports the earlier assumption that the SWMM2 model may have over predicted node storage during surcharge conditions.

5.3.1 Two-Year Event

The SWMM4 model, with the revised subcatchment parameters for the five-year event, was reviewed in detail to ensure it would still be representative for the two-year event. A sample comparison of runoff hydrographs at two nodes is shown on Figure 5.3. The overall district volume comparison is contained in the following table.

	SWMM2 (ft3)	SWMM4 (ft3)
Runoff	1,042,600	1,113,140
Infiltration	611,987	689,903
Water remaining in Surface Storage	746,144	625,714

This indicates that the revised SWMM4 model is representative of the SWMM2 model's hydrographs for the five-year event.

The five-year runoff hydrographs were then routed through the SWMM2 and the SWMM4 models. The maximum water levels predicted by each model were reviewed and compared.

Table 5.3
Hart District - SWMM2 VS SWMM4
Water Level Comparison
2 year Event

JUNCTION NUMBER	SWMM4	SWMM2	Difference \$4-S2	Levels within one foot?	Surcharged Node	JUNCTION NUMBER	SWMM4	SWMM2	Difference \$4-S2	Levels within one foot?	Surcharged Node	JUNCTION NUMBER	SWMM4	SWMM2	Difference \$4-S2	Levels within one foot?	Surcharged Node
1	743.57	743.58	-0.01	ok		168	745.52	745.52	0	ok		298	741.11	741.08	0.03		
3	741.53	741.61	-0.08	ok		169	741.13	741.27	-0.14	ok		304	746.92	746.9	0.02		ok
4	739.43	739.61	-0.18	ok		172	745.72	745.7	0.02	ok		306	744.22	744.15	0.07		ok
6	744.42	744.42	0	ok		174	744.50	744.56	0.02	ok		307	743.32	743.65	-0.33		ok
8	741.06	741.04	0.02	ok		175	743.45	743.46	-0.01	ok		309	740.96	740.4	0.56		ok
12	738.32	738.56	-0.24	ok		177	740.88	741.09	-0.21	ok		318	744.01	744.06	-0.05		ok
14	743.72	743.72	0	ok		181	740.76	740.91	-0.15	ok		321	742.81	742.84	-0.03		ok
18	740.17	740.19	-0.02	ok		183	740.33	740.5	-0.17	ok		326	748	747.99	0.01		ok
20	739.05	739.08	-0.03	ok		184	740.03	740.27	-0.24	ok		328	744.84	744.62	0.22		ok
24	737.75	737.98	-0.23	ok		185	739.78	740.01	-0.23	ok		331	742.89	739.7	3.19	SWMM4 is Higher	yes
27	744.47	744.47	0	ok		190	742.87	742.67	0	ok		335	747.18	747.17	0.01		ok
29	741.79	741.71	0.08	ok		195	741.73	742	-0.27	ok		338	743.71	743.77	-0.06		ok
35	739.31	739.4	-0.09	ok		196	741.78	741.77	0.01	ok	yes	339	744.84	742.61	2.23	SWMM4 is Higher	yes
40	736.7	736.83	-0.13	ok		200	743.22	743.32	-0.1	ok		346	747.46	747.36	0.1		ok
46	740.71	740.77	-0.06	ok		202	742.42	742.03	0.39	ok	yes	348	743.79	739.19	4.6	SWMM4 is Higher	yes
49	738.56	738.64	-0.08	ok		203	740.22	740.27	-0.05	ok	yes	350	749.63	738.22	11.41	SWMM4 is Higher	yes
55	736.33	736.5	-0.17	ok		204	746.47	746.47	0	ok		354	747.88	746.97	0.91		ok
56	744.09	744.08	0.01	ok		205	746.94	744.88	2.06	SWMM4 is Higher	yes	355	744.79	744.97	-0.18		ok
57	739.93	739.84	0.09	ok		206	743.32	743.38	-0.06	ok		357	742.8	742.78	0.02		ok
59	741.94	741.91	0.03	ok		207	743.02	742.53	0.49	ok	yes	359	741.19	741.23	-0.04		ok
63	739.92	739.61	0.31	ok		208	745.84	745.71	0.13	ok		364	738.13	738.32	-0.19		ok
66	745.84	745.88	-0.04	ok		211	740.25	740.45	-0.2	ok		990	735.23	735.44	-0.21		ok
67	745.03	744.95	0.08	ok		215	739.45	739.68	-0.23	ok		995	711.54	714.11	-2.57	SWMM2 is Higher	
68	744.03	743.96	0.07	ok		217	745.66	745.9	-0.24	ok		996	734	734.08	-0.08		ok
73	741.3	741.37	-0.07	ok		218	743.93	743.78	0.15	ok		997	711.54	714.11	-2.57	SWMM2 is Higher	yes
74	740.06	740.09	-0.03	ok		220	739.23	739.43	-0.2	ok		700	740.52	740.38	0.14		ok
77	738.74	738.85	-0.11	ok		221	744.84	744.72	0.12	ok		705	738.05	738.26	-0.21		ok
80	738.95	738.11	0.84	ok	yes	223	744.21	744.15	0.06	ok		701	742.12	742.48	-0.36		ok
82	737.18	737.33	-0.15	ok		225	742.75	743.04	-0.29	ok		710	748.12	747.88	0.24		ok
85	736.73	736.89	-0.16	ok		226	741.71	741.48	0.23	ok		715	748.65	746.18	0.37		ok
89	736.28	736.45	-0.17	ok		229	738.95	739.14	-0.19	ok		720	741.98	742.2	-0.22		ok
91	735.88	736.08	-0.2	ok		231	748.17	748.21	-0.04	ok		725	738.23	739.15	-0.92		ok
93	747.97	747.97	0	ok		233	744.06	743.89	0.17	ok		730	738.06	738.76	-0.7		ok
96	748.24	748.28	-0.04	ok		235	743.62	743.72	-0.2	ok		751	741.08	741.23	-0.15		ok
98	746.35	746.44	-0.09	ok		238	740.95	741.16	-0.21	ok		752	739.86	740	-0.14		ok
102	747.47	747.45	0.02	ok		239	738.3	738.42	-0.12	ok		753	739.3	739.41	-0.11		ok
106	745.95	745.94	0.01	ok		241	743.97	743.97	0	ok		754	738.21	738.31	-0.1		ok
107	745.14	745.23	-0.09	ok		242	742.1	742.11	-0.01	ok		1143	739.44	739.74	-0.3		ok
111	745.91	745.99	-0.08	ok		244	740.54	740.47	0.07	ok		800	736.53	736.45	0.08		ok
113	744.17	744.22	-0.05	ok		245	739.87	739.83	0.04	ok		805	736.61	736.49	0.12		ok
117	746.47	746.47	0	ok		247	737.78	737.92	-0.14	ok		806	736.56	736.51	0.05		ok
121	744.28	744.4	-0.12	ok		250	745.7	745.75	-0.05	ok		807	736.53	736.47	0.06		ok
125	746.49	746.44	0.05	ok		253	748.15	748.13	0.02	ok		810	738.84	737.02	-0.18		ok
126	744.69	744.65	0.04	ok		254	746.62	746.69	-0.07	ok		815	737.73	737.83	-0.1		ok
128	743.78	743.87	-0.09	ok		259	737.55	737.85	-0.3	ok		820	738.15	736.43	-0.28		ok
133	746.77	746.71	0.06	ok		263	737.56	737.82	-0.26	ok		8510	736.54	736.54	0		ok
135	743.18	743.34	-0.16	ok		268	737.59	737.79	-0.2	ok		811	736.42	736.45	-0.03		ok
136	743.18	742.55	0.63	ok	yes	270	737.43	737.69	-0.26	ok		878	738.39	738.54	-0.15		ok
140	741	741.26	-0.26	ok		271	737.35	737.62	-0.27	ok		818	737.55	737.66	-0.11		ok
141	740.83	741.04	-0.21	ok		272	751.42	751.35	0.07	ok		829	739.24	739.39	-0.15		ok
143	746.31	746.39	-0.08	ok		274	746.55	746.85	-0.3	ok		846	738.09	738.22	-0.13		ok
147	749.07	748.97	0.1	ok		275	745.52	745.73	-0.21	ok		859	738.78	738.88	-0.1		ok
149	746.97	746.88	0.09	ok		276	744.61	744.58	0.03	ok		966	741.9	741.96	-0.06		ok
150	745.74	745.78	-0.04	ok		279	742.05	742.03	0.02	ok		9200	740.43	740.93	-0.5		ok
151	744.82	744.65	0.17	ok		280	749.96	749.99	-0.03	ok		9205	742.67	742.67	0		ok
154	749.1	749.01	0.09	ok		281	746.99	746.99	0	ok		9217	742.45	742.58	-0.13		ok
158	747.53	747.49	0.04	ok		283	741.28	741.42	-0.14	ok		9555	739.51	741.1	-1.59	SWMM2 is Higher	
160	745.61	745.6	0.01	ok		286	739.74	739.84	-0.1	ok		9195	740.13	740.27	-0.14		ok
164	743.62	743.65	-0.03	ok		289	747.19	747.34	-0.15	ok							
165	741.99	741.98	0.01	ok		294	743.6	743.65	-0.05	ok							
166	749.1	749.08	0.02	ok		295	741.69	741.65	0.04	ok							

The peak water levels from the models and the difference in the levels are listed in Table 5.3. The table also indicates whether the difference was within 1 ft, and if not, which model predicted the higher level. The final column indicates if the node was surcharged.

For the majority of the district the models predict nearly identical peak water levels. It is also notable that surcharging was evident for each location where the SWMM4 levels were over 1 ft higher than the SWMM2 levels. This analysis supports the earlier contention that the SWMM2 model may have over predicted node storage during surcharge conditions.

5.3.2 Discussion of Results

The goal of the modelling exercise was to develop a SWMM4 model that accurately represents the hydraulic response of the system under wet weather flow conditions in the Hart District combined sewer system to assess the possible impacts resulting from accessing inline storage. The original intent was to use the rainfall and sewer level data collected for the initial SWMM2 calibration exercise. Unfortunately the majority of this data could not be retrieved. The SWMM4 model was calibrated to the runoff hydrographs calculated by the original SWMM2 model for the five-year storm event. The SWMM4 model was then verified with the two-year event. As the SWMM2 model was calibrated to monitored data, the comparison of the results indicates that the SWMM4 model is also representative.

There is a strong indication that the original SWMM2 model over predicted system storage at nodes under surcharge conditions. This may have a minor impact on the initial calibration work since the runoff parameters were developed based on storm events with some sewer surcharging, although not to a great extent. It does impact the level of basement flood for more intense events when sewer surcharging is widespread. This is apparent from the five-year event as the SWMM4 Extran analysis indicates significantly higher levels at several locations.

Based on the foregoing, it is reasonable to assume that the SWMM4 model is representative of the hydrology in the Hart District. This model can therefore be used for

ID No.	Location			curb inlet present	cb cover type	Restrictor			Hood present	hook present	pin present	cleaning required	Comments
	Street	House #	Side			present	size	type					
108	UNION	712	S	Y	STO. GRATE	N/A	N/A	N/A	Y	Y	Y	NO	2.16 NEED STAIRS
109	UNION	312	N	Y	STO	N/A	N/A	N/A	Y	Y	Y	NO	2.18 NO STAIRS
110	UNION	288	S	Y	STO	N/A	N/A	N/A	Y	Y	Y	NO	2.18 NO STAIRS
111	UNION	288	N	Y	STO.	N/A	N/A	N/A	Y	Y	Y	NO	2.42 NEED STAIRS
112	UNION	253	S	Y	STO	N/A	N/A	N/A	Y	Y	Y	NO	1.94 (NO STAIRS)
113	UNION	253	N	N	STO.	N	10"	PVC	N	Y	N	NO	1.82 NO STAIRS.
114	MARTIN	317	S	Y	STO.	Y	4"	PVC	Y	Y	Y	NO	1.94
115	MARTIN	317	N	Y	STO.	Y	4"	ASBESTOS	Y	Y	Y	NO	2.11 NO STAIRS
116	MARTIN	268	S	Y	STO.	Y	4"	PVC	Y	Y	Y	NO	NO STAIRS = 23
117	MARTIN	268	N	Y	STO.	Y	4"	PVC	Y	Y	Y	YES	2.20 PULLED IN STAIRS
118	MARTIN	238	S	Y	STO	Y	4"	CWC	N	N	N	YES	2.08 (NO STAIRS) PULLED
119	MARTIN	238	N	Y	STO	Y	4"	CLAY	Y	Y	Y	NO	(NO STAIRS) 2.23
120	HARRISON	250	S	N	STO GRATE	Y	4"	CLAY	Y	Y	Y	NO	(NO STAIRS) 2.24
121	HARRISON	250	N	N	STO GRATE		4"	CLAY	Y	Y	Y	NO	(NO STAIRS) 1.63
122	HARRISON	282	S	N	STO GRATE	Y	4"	CLAY	Y	Y	Y	NO	1.70 (NO STAIRS)
123	HARRISON	282	N	N	STO. GRATE	Y	4"	CLAY	Y	Y	Y	NO	1.70 (NO STAIRS)
124	HARRISON	324	S	N	STO. GRATE	Y	4"	CLAY	Y	Y	Y	YES	1.83 (NO STAIRS) (PULLED)
125	HARRISON	324	N	N	STO. GRATE	Y	4"	CLAY	Y	Y	Y	NO	1.56 (NO STAIRS)
126	BRADLER	200	E	Y	STO. GRATE	N	10"	PVC	N	N	N	NO	1.77 (NO STAIRS)
127	BRADLER	300	W	Y	STO. GRATE	N	10"	CLAY	N	N	N	NO	1.79
128	BRADLER	281	E	Y	STO. GRATE	N/A	N/A	N/A	Y	Y	Y	NO	1.79
129	BRADLER	281	W	N	STO GRATE	N	10"	CLAY	Y	Y	Y	NO	2.12 COULDN'T RAISE HOOD

Figure 5.4

the design of inlet controls and as a basis for the assessment of inline storage and operating protocols. With the implementation of inlet controls, peak runoff rates from the individual subcatchments become of less importance due to the restricted flow rate to the sewer system. However, the runoff volume is important from a street storage perspective, and, for the assessment of long-term performance of the inline storage works. Therefore, it is recommended that the model calibration be reviewed prior to future pilot testing.

5.4 PRELIMINARY FIELD INVESTIGATIONS

Preliminary field investigations were required to determine the size and location of the existing inlet controls in the Hart District. In preliminary meetings with the City it was also determined that the study team would be able to easily pick up additional information during the investigations that would be useful to the City. This included:

- Catchbasin location by street, house number and side of the street.
- If there was an associated curb inlet.
- Type of cover.
- Size and type of inlet control.
- If the hood and/or associated mounting hardware was present.
- Depth from rim to invert of outlet.
- If the catchbasin required cleaning.
- General comments on catchbasin condition and maintenance requirements.

A form was developed for field personnel to complete during the site inspections (Figure 5.4). Subsequently the data was input to a database and forwarded for review and comments.

The City was concerned that the removal of the existing inlet restrictors would increase the amount of effort and thereby the cost of implementing inlet controls. The existing controls were thought to be 0.6 m long clay tile pipes inserted in the outlet and grouted in place. A private contractor was hired to remove the existing pipe inserts at six random catchbasins, and in this manner, determine the level of effort required for removal.

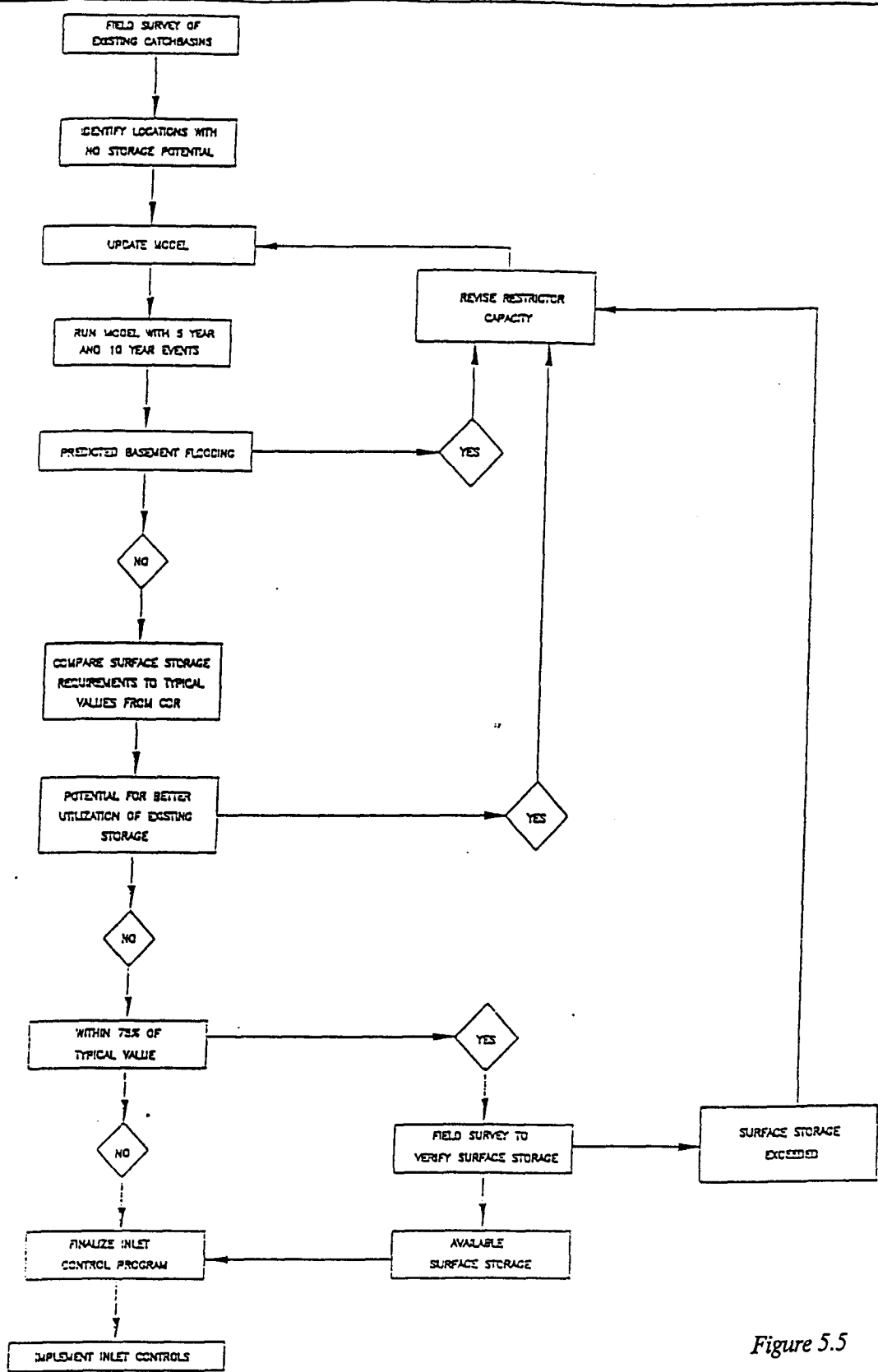


Figure 5.5

WARDROP ENGINEERING INC.
ENGINEERS ARCHITECTS TORONTO TELEPHONE 347

HART DISTRICT
 FLOW CHART FOR INLET CONTROL DESIGN

DESIGNED BY: G.S.	DRAWN BY: M.M.	DWG NO.
CHECKED BY:	DATE: 97.07.23	940012-02-48

This work indicated that the pipe inserts were grouted in place at the catchbasin wall. Although an air compressor was on hand, five of the six pipe inserts were removed with a sharp rap from a small sledgehammer. Any excess grout was easily removed from the interior of the outlet and the new disc-type orifice controls were installed with ease. This indicates that the removal of the existing inlet controls will have little, or no, impact of the cost of implementing inlet controls in the Hart District.

5.5 STATUS

The status of the Hart District modelling with respect to the three proposed workstreams is as follows:

- Calibration – The calibration works have been completed. The SWMM4 model was based on hydrographs created by the original SWMM2 model of the Hart District. There is a concern that the SWMM2 model over predicts node storage during surcharge conditions but this is a basement flood protection issue. The SWMM4 model is sufficiently representative of the Hart District hydrology for use in the implementation on inlet controls and the analysis if inline storage. It is recommended that the model's accuracy, particularly with respect to runoff volume, be reviewed, and adjusted as required, during future inline storage pilot testing.
- Inlet Controls – The preliminary field investigations have been completed and the information added to a database. This information has been forwarded to the City for review. It is anticipated that the removal of the existing pipe insert controls will have little, or no, impact on the cost of implementing inlet controls in the district. Detailed design of inlet controls was not started; however, a flow chart for the design was developed and is shown on Figure 5.5.
- Inline Storage and Gate Control Protocol – This work was not carried out. It will require importing the SWMM4 model to the XPSWMM environment to utilize the regulator link facility to model the proposed control gate operations.

6. REFERENCES

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