CLEVELAND STREET BASIN DRAINAGE STUDY

STORMWATER MANAGEMENT DIVISION
DEPARTMENT OF PUBLIC WORKS
CITY OF TAMPA, FLORIDA

POST, BUCKLEY, SCHUH & JERNIGAN, INC.
CONSULTING ENGINEERS PLANNERS LANDSCAPE ARCHITECTS

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September 8, 1983

Mr. Ron Giovannelli, Chief
Stormwater Management Division
City of Tampa
315 E. Kennedy Blvd.
Tampa, Florida 33602

Dear Mr. Giovannelli:

We are pleased to submit this final report on our study of the Cleveland Street drainage basin.

In this report, we have inventoried the existing drainage system, analyzed the causes of current flooding in the system through computer analysis and have developed and modeled twelve alternative solutions. Through an evaluation process which takes into account estimated costs, water quality impacts and performance, we have recommended a preferred solution to the flooding problems in the Cleveland Street Basin. As part of our recommendations, we have proposed a phasing schedule for implementation of the recommended plan, together with anticipated costs for each phase.

We appreciate this opportunity to carry out this assignment for the City of Tampa as well as the excellent cooperation and support we received from the City's Storm Water Management staff during the preparation of this report.

Very truly yours,

POST, BUCKLEY, SCHUH & JERNIGAN, INC.

Garth W Horne

Garth Horne, P.E.
Project Manager

GH:bt
578-201.00
M/L2

Prepared For:

The City of Tampa

CLEVELAND STREET BASIN

DRAINAGE STUDY

September, 1983

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EXECUTIVE SUMMARY

The City of Tampa, through its Stormwater Management Division, has initiated a series of comprehensive stormwater management studies throughout the City with the major objectives of alleviating severe flooding problems while maintaining acceptable water quality standards in receiving waters. The preparation of the Cleveland Street Basin study was authorized by Work Order No. 1, dated October 19, 1982, issued under Contract 2411-H, dated October 18, 1982 between the City of Tampa and Post, Buckley, Schuh & Jernigan, Inc., Consulting Engineers and Planners.

The objectives of this study were to inventory the existing Cleveland Street Basin drainage facilities, analyze the performance and adequacy of the system through computer model simulation, identify the causes of recurrent flooding wherever such flooding occurs during relatively minor rainfall events and recommend, after evaluating a variety of possible alternatives, a preferred cause of remedial action, based on economic, environmental, feasibility and performance considerations.

City of Tampa records indicate that significant flooding occurs throughout the existing system during rainfall events which are far less severe than the desirable design storm for City drainage facilities. The major cause of this flooding is the overtaxing of the basin's major conveyance system, which was originally designed and constructed to handle runoff from a slightly smaller and much less intensely developed urban area than that which is now served by the system.

An inventory of the existing drainage facilities was accomplished by researching the City of Tampa design and as-built drawings, Florida Department of Transportation as-builts and by field survey. Basin and sub-basin boundaries were determined by review of recent aerial topography, the City of Tampa drainage atlas sheets and, in questionable areas, field observation or survey.

Due to the complexity of the Cleveland Street drainage system, computer modeling techniques were employed to determine the performance and adequacy of the existing system and evaluate alternative improvements to the existing system. A review of existing non-propriatory stormwater models revealed that the hydrologic analysis of the Cleveland Street Basin could best be accomplished using the HEC-1 hydrologic program. Due to the extreme complexity of the Cleveland Street Basin drainage system, the hydraulic program EXTRAN was selected for the hydraulic analysis. To model the basin runoff hydrographs were simulated with the HEC-1 computer program. These hydrographs were then input to the EXTRAN program which simulated, on a real time basis, the hydraulics of the Cleveland Street Basin pipe network.

In order to verify these programs and calibrate the models developed for the Cleveland Street Basin, a gauging station was established at a manhole in the vicinity of the Cleveland Street and Clark Avenue intersection to record stormwater flows in the system. This flow data was then utilized to adjust hydrologic and hydraulic model input parameters until the simulated results of the models agreed, within a reasonable degree of accuracy, with the measured flows. Once calibrated, the models were then ready to analyze the performance and adequacy of the existing facilities for the design event. The design event selected, based on a review of literature and current City policy was the five year, one and one-half hour duration event totaling three and three tenths inches of rainfall. Model simulation results of the design rainfall confirmed that the existing drainage facilities are severely overtaxed. Significant amounts of flooding were indicated by the model to occur along Cleveland Street, west of Grady Street; east of Himes Avenue, both north and south of Kennedy Boulevard; and, in the commercial areas of Westshore.

Based on the results of the existing condition simulation for the design rainfall, twelve improvement alternatives were developed to partially or totally alleviate significant flooding in the basin for the design event. Three of these alternatives eliminates all significant flooding within the basin for the design event. For each of the twelve alternatives, order of magnitude cost estimates were developed and water quality impacts on ultimate receiving waters evaluated. The twelve alternatives were then jointly reviewed by City of Tampa staff and PBS&J and a preferred alternative was selected after weighing implementation cost, environmental impacts, feasibility and performance of each alternative.

The preferred alternative for the Cleveland Street Basin would eliminate all significant flooding within the basin at an estimated cost of $10,270,000 and could be implemented in four phases ranging in cost from $1.5 to $4.3 million. The preferred alternative includes:

- o Construction of a new separate outfall for the Westshore area to accommodate the intense development that has occurred since design and construction of the existing system.
- o Removal of utility conflicts and placement of an epoxy liner in the Cleveland Street box culvert from Himes Avenue to the outfall.
- o Construction of detention facilities and associated minor pipe improvements as shown on Figure 4-11, Page 4-14, Alternative 12, Facility Schematic.

This is the most economical of the three alternatives developed which would entirely eliminate all flooding in the basin during the design event and would result in an estimated eighteen percent reduction in pollutant loading at the outfall. This preferred solution also has the advantage of being suited to phased implementation.

The phasing plan recommended for constructing the proposed improvements in the Cleveland Street Basin includes four major phases:

1. Improve flow conditions in the existing box culvert and construct a new outfall for Westshore. Estimated cost = $1,500,000.

2. Construct detention facilities and associated pipe improvements in the area east of Himes Avenue and south of Kennedy Boulevard. Estimated cost = $2,600,000.

3. Construct detention facilities and piping in the area east of Himes Avenue, north of Kennedy Boulevard and in the vicinity of Grady Avenue and Cleveland Street. Estimated cost = $4,300,000.

4. Construct detention facilities and piping to serve the Westshore area. Estimated cost = $1,910,000.

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Section 1

INTRODUCTION

1.1 BACKGROUND

The City of Tampa, through its Stormwater Management Division, has intitiated a series of comprehensive stormwater management studies throughout the City with two major objectives; to alleviate severe flooding problems wherever they exist (or have the potential to occur) and to maintain acceptable water quality standards in receiving waters. This study of the hydrologic, hydraulic and water quality characteristics of the Cleveland Street Basin is an important first step in the implementation of the City's program. The location of the basin is shown in Figure 1-1.

1.2 AUTHORIZATION

Preparation of the Cleveland Street Basin study was authorized by Work Order No. 1, dated October 19, 1982, issued under Contract 2411-H, dated October 18, 1982 between the City of Tampa and Post, Buckley, Schuh & Jernigan, Inc., Consulting Engineers and Planners (PBS&J).

1.3 PURPOSE AND SCOPE

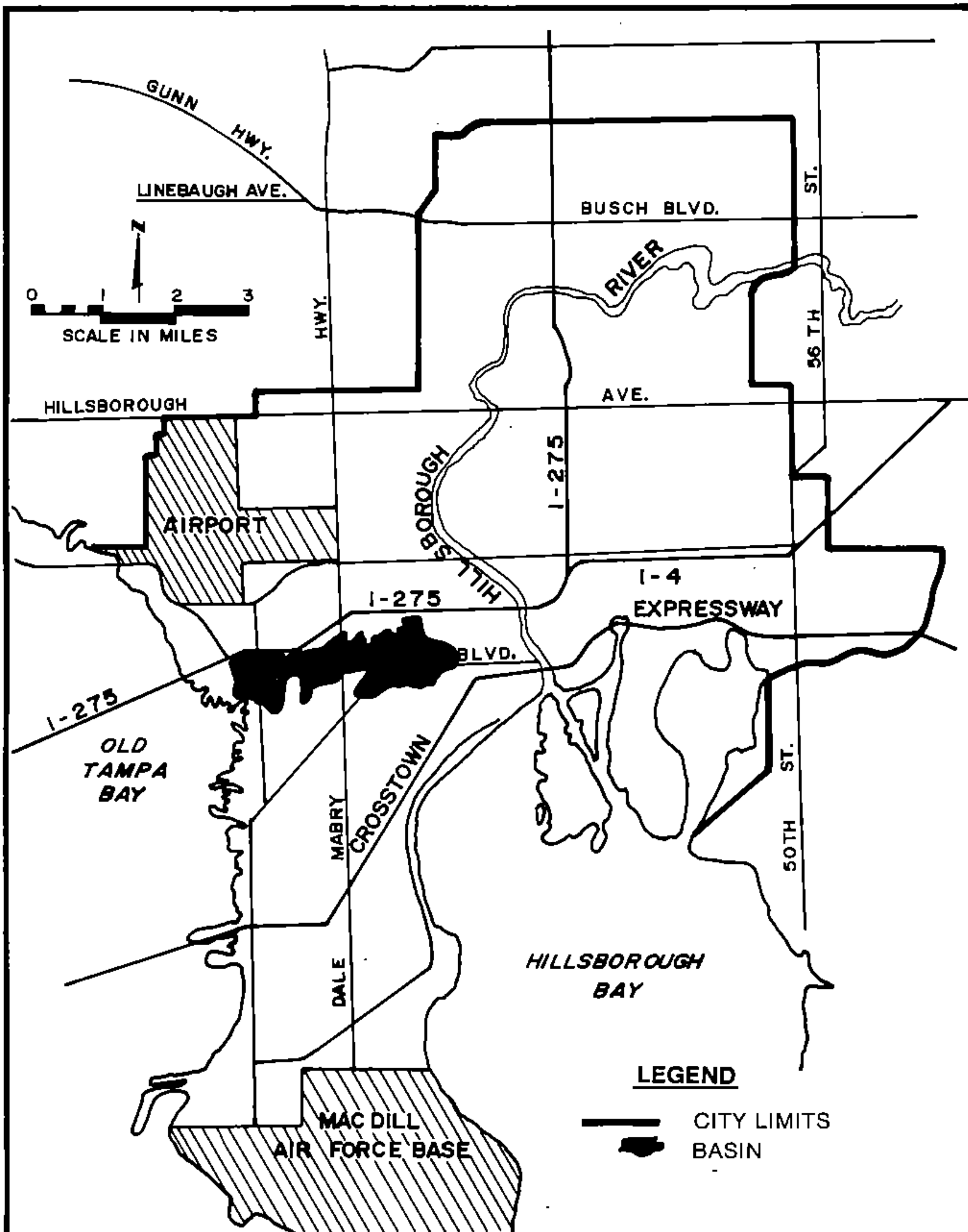
The objectives of this study were to inventory the existing Cleveland Street Basin drainage facilities, analyze the performance and adequacy of the system through computer model simulation, identify the causes of recurrent flooding wherever such flooding occurs during relatively minor rainfall events and recommend, after evaluation of a variety of possible alternatives, a preferred course of remedial action, based on economic, environmental, feasibility and performance considerations.

The initial item of work undertaken by PBS&J was the development of a Plan of Study detailing the scope of work to be performed, the input data required and output for each task in the study. The major tasks performed in order to achieve the stated objectives of the study were:

- Meetings with representatives of the Florida Department of Environmental Regulation (DER) to obtain their input relative to water quality considerations, acquaint them with the scope and purpose of the study and to ascertain their requirements for obtaining environmental approval for proposed alternative system improvements.

- Inventory of the existing Cleveland Street Basin drainage system through location, tabulation, field surveying and verification of design and as-built drawing information available from City of Tampa records.

- Selection of the most appropriate computer model(s) for analysis of the Cleveland Street system.



CLEVELAND STREET BASIN

LOCATION MAP

FIGURE 1-1

- Tabulation of existing Land Use information, determination of stormwater runoff input parameters and establishment of design storm criteria.

- Preparation of model input data, verification of the model and calibration of the Cleveland Street system.

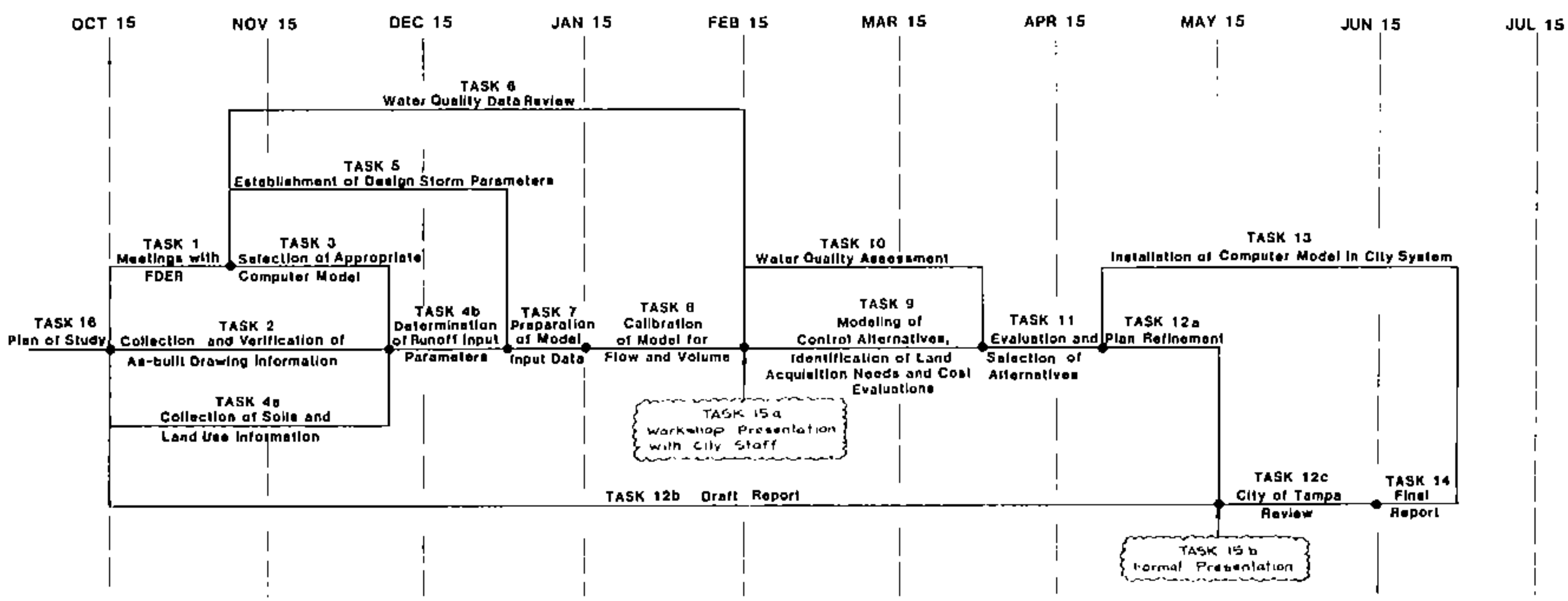
- Modeling of Control alternatives, assessment of the water quality, land acquisition and cost impacts of each alternative and selection of the preferred alternatives.

1.4 AGENCY ROLES

Throughout the progress of this study, PBS&J has worked closely with the staff of the City of Tampa Stormwater Management Division concerning selection of appropriate criteria, methodology to be employed and critical decisions affecting the course of the study. In addition, representatives of the DER were consulted at appropriate points in the study to help assure that the system improvements ultimately selected would provide an environmentally acceptable solution.

1.5 STUDY COORDINATION ACTIVITIES

The schedule of activities for the Cleveland Street Basin Drainage Study, shown in Figure 1-2, indicates the sequential relationship of the various tasks undertaken during the course of the study. In conjunction with these activities, two additional programs authorized by the City of Tampa provided valuable input to the Cleveland Street Study. The first of these was the Nationwide Urban Runoff Program (NURP) Study which established water quality data which was reviewed during the evaluation of the environmental impacts of proposed alternatives. The second was a City-wide data collection effort which provided stage elevation and flow data utilized to calibrate the existing Cleveland Street system.



CLEVELAND STREET BASIN
TASK COMPLETION SCHEDULE

FIGURE I-2

Section 2

INVENTORY OF EXISTING SYSTEM

2.1 HISTORICAL FLOODING: RECORDS AND CAUSES

Although little technical data is available concerning historical flooding in the Cleveland Street Basin, City of Tampa records indicate that significant flooding occurs throughout the existing system during rainfall events which are far less severe than the desirable design storm for City drainage facilities. Figures 2-1W and 2-1E present an overall map of the system indicating those areas in which these flooding occurrences are most prevalent.

The major cause of this flooding appears to be overtaxing of the basin's major conveyance system which was originally designed and constructed to handle runoff from a slightly smaller and much less intensely developed urban area than that which is now served by the system.

The Cleveland Street outfall was originally designed by Florida Department of Transportation, FDOT, in 1956 through a joint agreement between the FDOT and the City of Tampa and was constructed concurrently with the Kennedy Boulevard (S.R. 60) improvements west of Dale Mabry Highway. A number of factors render that design inadequate to handle the runoff from the basin as it exists today:

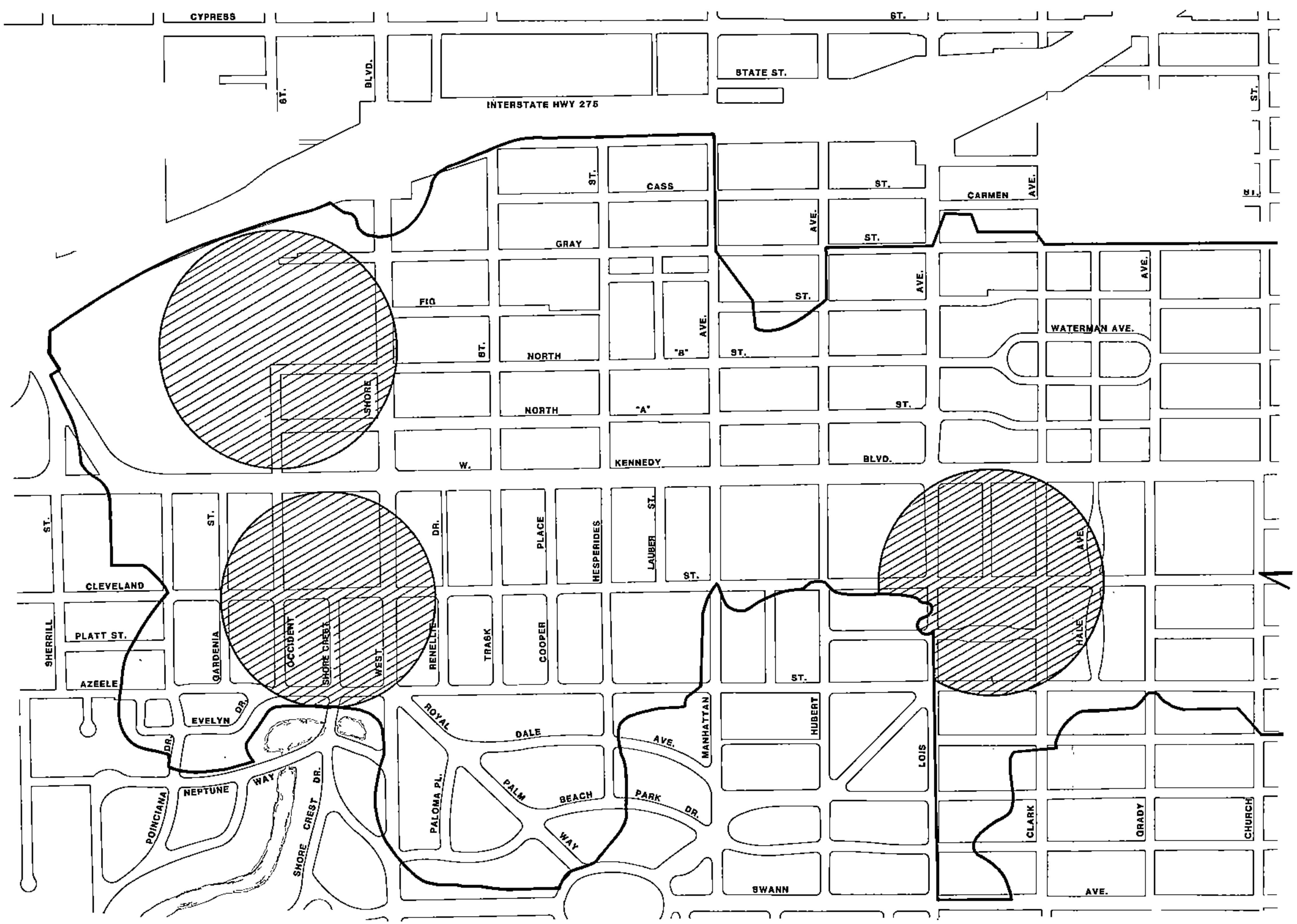
- Runoff coefficients used for design in the basin, which was relatively sparsely developed in 1956, ranged from 0.21 to 0.30 and averaged about 0.25. As a result of the development which has occurred in the area since that time, the average runoff coefficient for the basin has increased to approximately double that figure. In June, 1980 the Tampa Department of Public Works estimated the actual runoff coefficient to be 0.51. This factor alone would double the quantity of stormwater runoff flowing to the Cleveland Street system.

- Whereas the total area considered in the original design of the outfall was 994 acres, the area which now contributes to the Cleveland Street system is estimated at approximately 1,100 acres.

- The design storm used to design the system in 1956 was a three year frequency event. Current Tampa criteria is to design drainage facilities adequate to accommodate a five year event.

- The storm intensities reflected in rainfall curves have increased since the date of the original design. This increase is attributable to a combination of a greater amount of statistical data on which to base the curves, the development since that time of curves based on specific data for the Tampa region and, perhaps, some climatic change in the period since 1956.

Construction of the Cleveland Street outfall was accomplished in the late 1950s. Apparently, the system handled the drainage requirements of the basin adequately until the late 1960s. In 1969, the City began to receive



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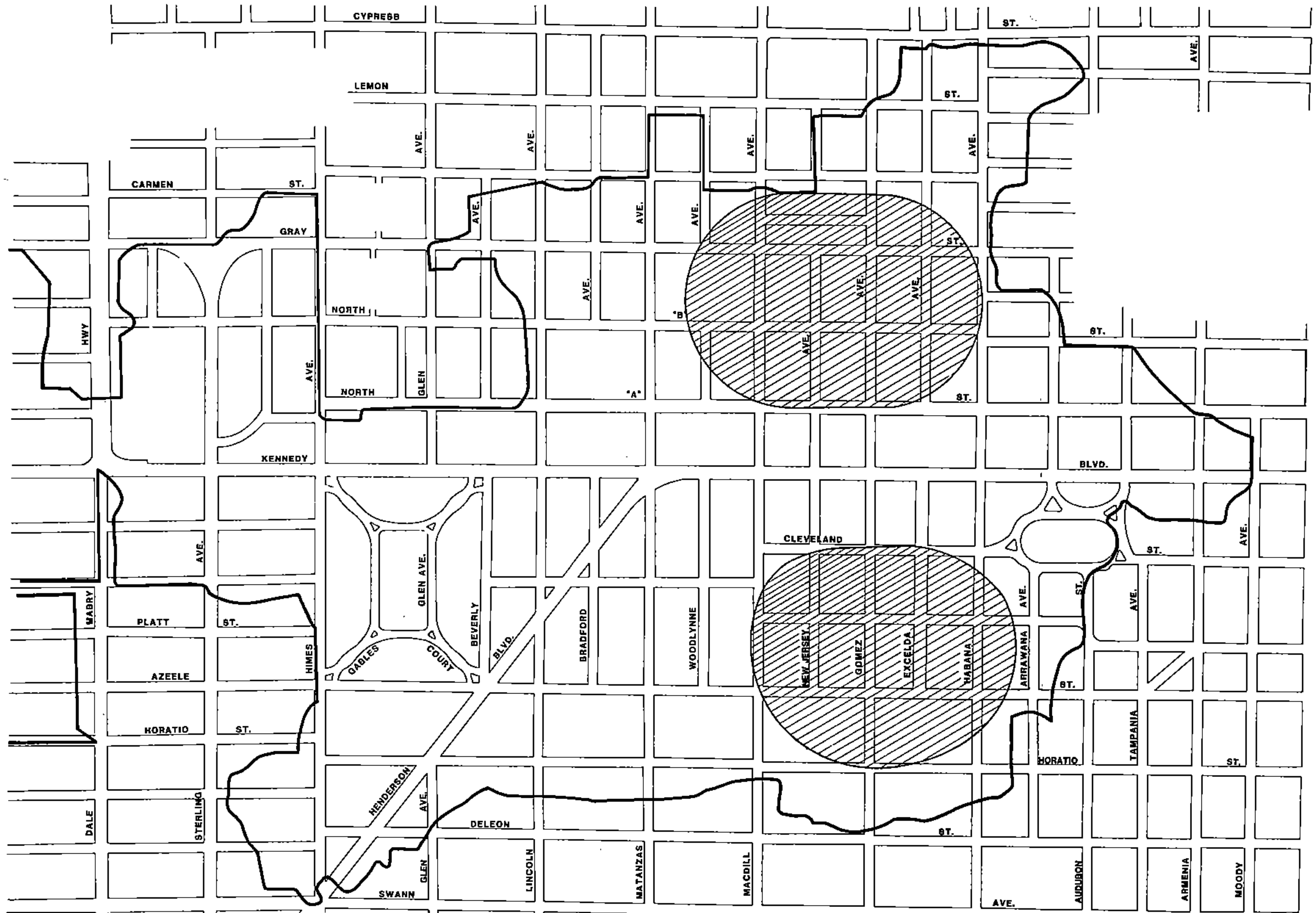
CLEVELAND STREET BASIN (WEST)

FLOOD PRONE AREAS

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION



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CLEVELAND STREET BASIN (EAST)

FLOOD PRONE AREAS

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

SCALE IN FEET

complaints concerning overflow of the system from residents who experienced flooding in the streets, their yards and, in some cases, their homes. Since that time, as development progressed in the basin, flooding has become more frequent and severe.

A review of the City's files concerning the Cleveland Street system reveals that areas where complaints from residents indicate that flooding is most severe include the vicinity of Westshore and Cleveland Street, Clark Avenue and Cleveland and on Platt Street, just south of the intersection of Krental Avenue and Cleveland.

2.2 TOPOGRAPHY

Three basic sources of information were utilized to determine the topographical features of the Cleveland Street Basin. The first source used was Drainage Atlas maps available from the City of Tampa which indicated elevations at street intersections within the basin and general directions of overland flow. The second existing source was topographical maps provided by the Southwest Florida Water Management District (SWFWMD), indicating existing one foot contours. PBS&J supplemented this data in areas where necessary to delineate subbasin boundaries by field inspection and/or field surveys.

One significant topographical feature of the basin was found to be the relatively sudden drop in the profile of Cleveland Street between Church Avenue and Hale Avenue. In this area, the existing surface elevations fall from 22.9± ft. to 11.8± ft. in a distance of approximately 1,000 feet. A profile of the Cleveland Street roadway from Himes Avenue to Occident Street, which displays this sudden drop in ground elevations, is presented in Figure 2-2. East of Church Avenue, the basin is extremely flat, ranging in elevation from 20 to 23 feet over a distance of approximately one mile. West of Hale Avenue, elevations generally slope toward Old Tampa Bay.

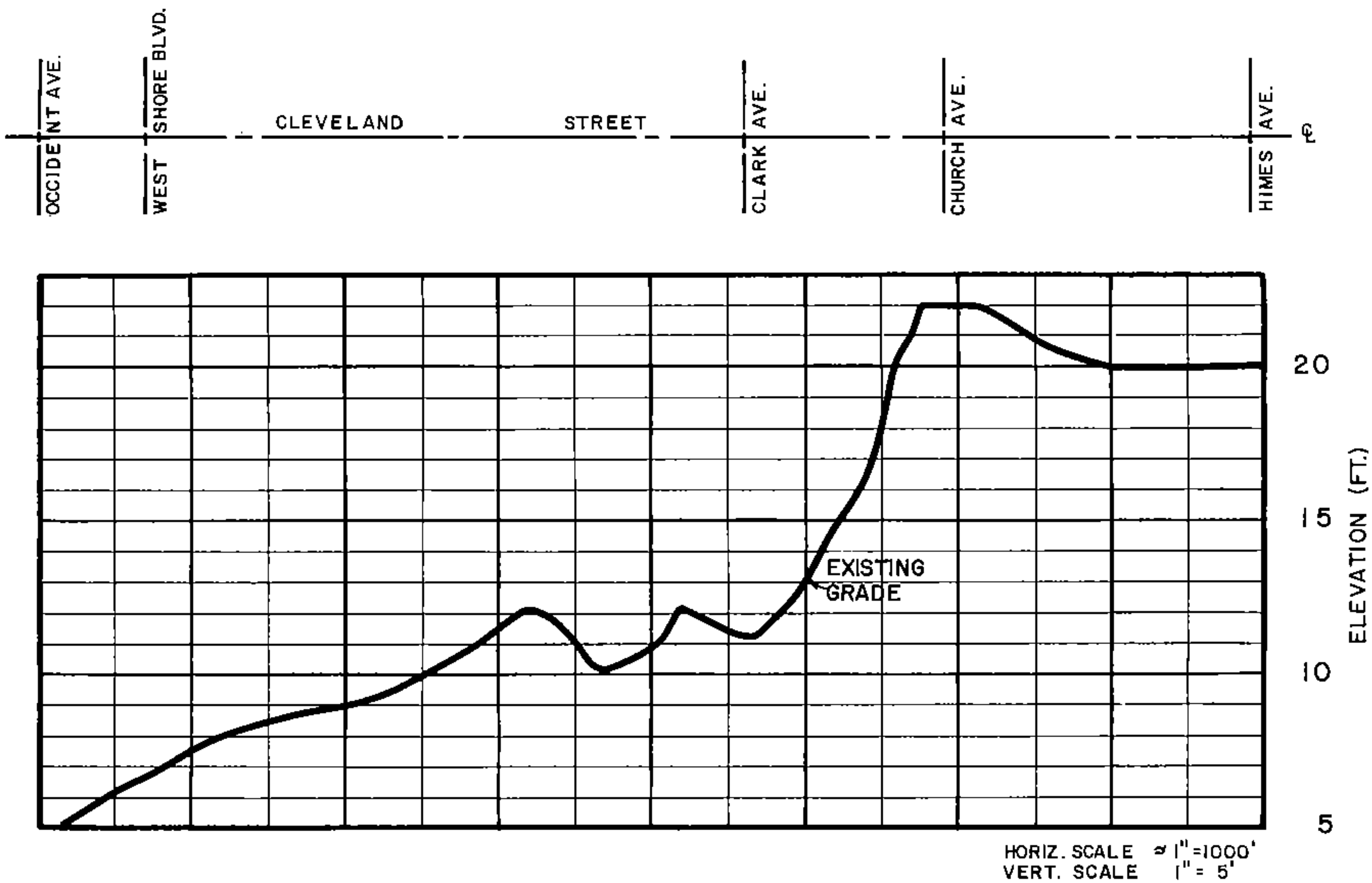
2.3 LAND USE

Since the Cleveland Street Basin is almost totally developed, existing land use is best demonstrated by the aerial zoning map of the study area presented in Figures 2-3W and 2-3E. In general, the land uses in the area are primarily single family residential with heavy concentrations of commercial development along the major arterial streets traversing the site and in the Westshore area.

Existing land uses in the basin are almost universally consistent with those shown in Hillsborough County's Horizon 2000 Land Use Plan. Except for the anticipated development (or redevelopment) of several prime commercial sites within the study area, very little change in land use is expected in the foreseeable future. For that reason, the analyses prepared in the development of this study were based on existing land uses.

2.4 CONVEYANCE SYSTEM GENERAL CHARACTERISTICS

Figure 2-4 indicates the major drainage facilities within the basin. These facilities comprise a closed conduit system connecting smaller feeder systems throughout the basin to a major conveyance made up of increasingly larger



CLEVELAND STREET BASIN
CLEVELAND STREET ROADWAY PROFILE

FIGURE 2-2

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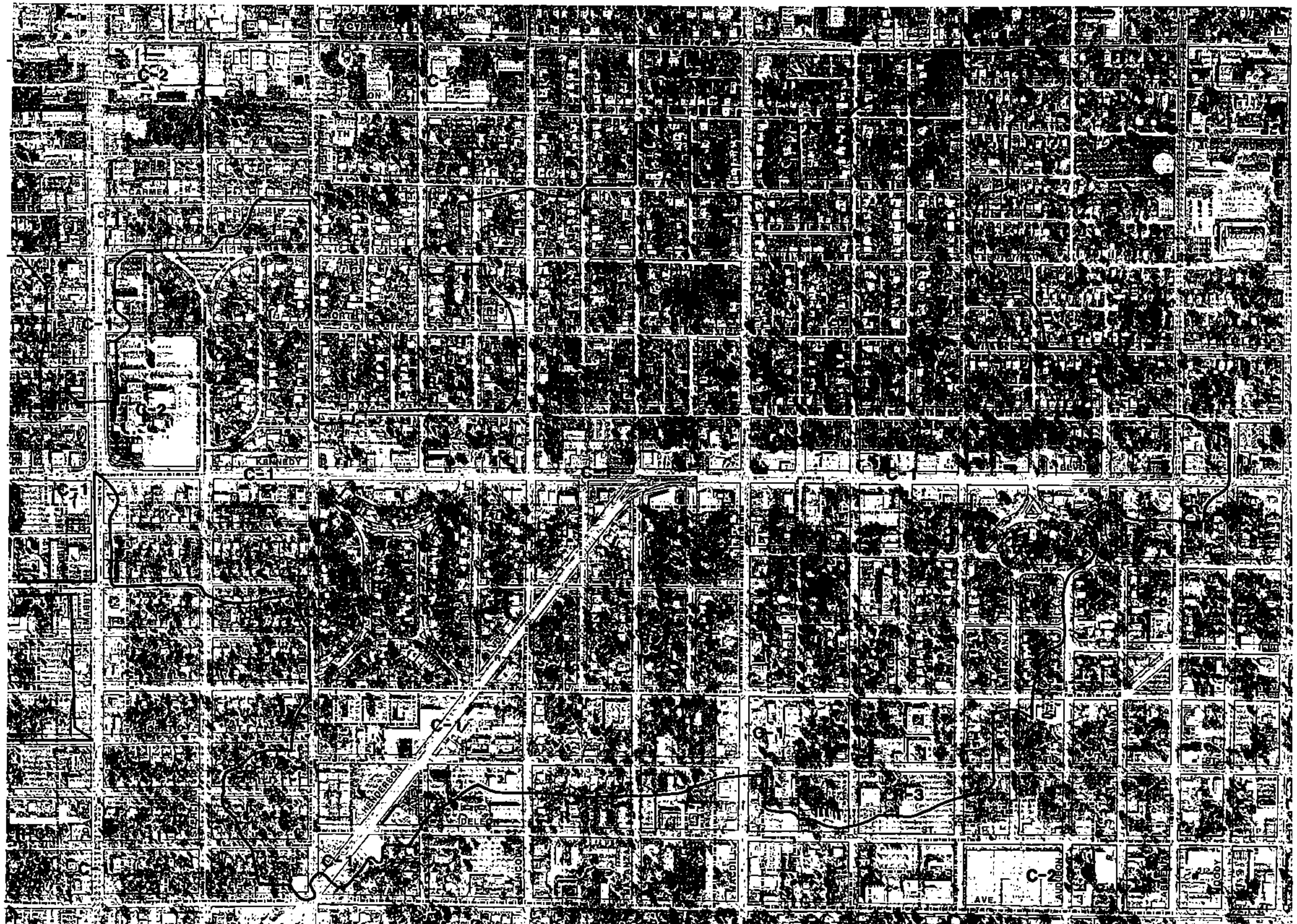
CLEVELAND STREET BASIN (WEST)

ZONING

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

0 200 400 600
SCALE IN FEET



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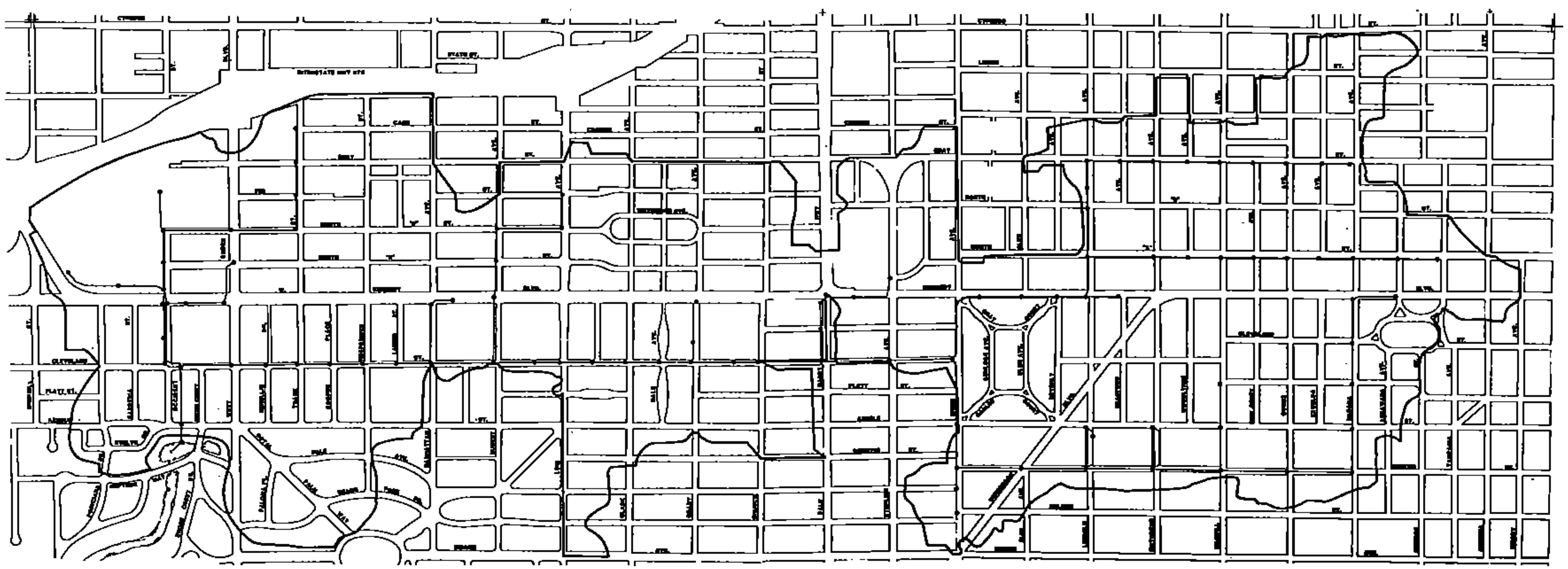
CLEVELAND STREET BASIN (EAST)

ZONING

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

0 200 400 600
SCALE IN FEET



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CLEVELAND STREET BASIN

FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

SCALE IN FEET

concrete box culverts, located in the right-of-way of Cleveland Street and ultimately outfalling in Neptune Lagoon at Neptune Way and Shore Crest.

East of Himes Avenue, the major drainage facilities in the basin consist of two parallel conduit systems; one serving the area north of Kennedy Boulevard and the other serving the area south of Kennedy Boulevard. The confluence of these two sytems occurs in the manhole located at the intersection of Himes Avenue and Cleveland Street.

From Himes Avenue westward to Westshore Boulevard the system consists of the box culvert in Cleveland Street with feeder systems from the north and south connecting directly to it at intermittent locations. In the Westshore area, a large feeder system, constructed during development of the Westshore commercial district, discharges into the Cleveland Street box culvert at the intersection of Cleveland Street and Occident.

2.5 MEASUREMENT OF FLOWS IN EXISTING SYSTEM

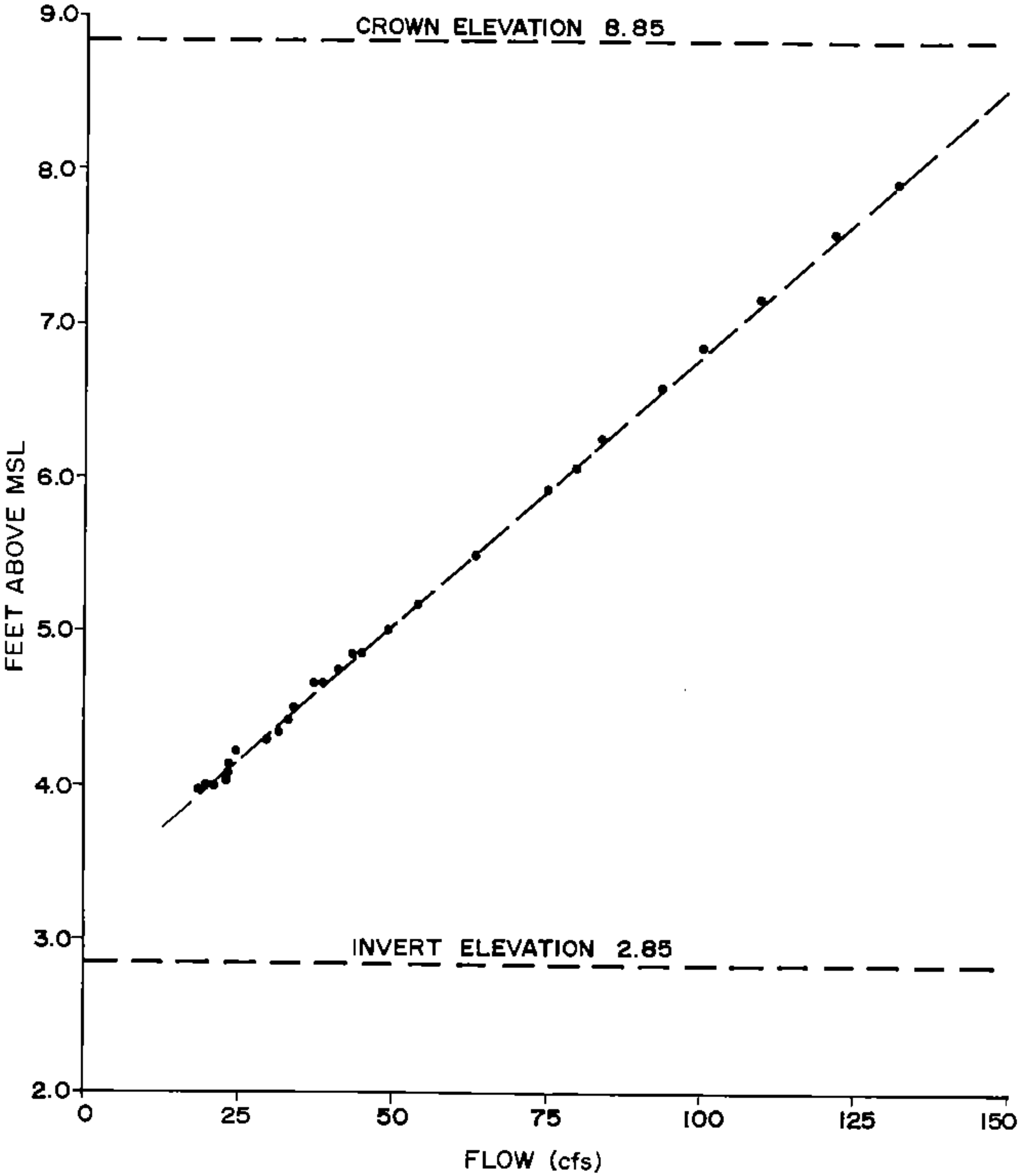
Through a separate work order, authorized by the City, records of flow in the system were obtained by field measurement. The data obtained by these measurements was used to calibrate the model input parameters for generating runoff from the site and for conveyance characteristics of the existing drainage facilities.

To obtain these measurements, a gauging station was established in the system at a manhole located at the intersection of Cleveland Street and Clark Avenue. The equipment used for measuring flows was a Marsh McBirney MMI 265 recording flowmeter which continuously monitored stage elevation and flow velocities at the gauging station and charted the resultant flows in million gallons per day (mgd) on a 24 hour, real time basis. This installation was in place from December 1, 1982 through February 15, 1983. Two significant rainfall events occurred and were recorded during this period; one on January 20, 1983 and one on February 2, 1983.

To supplement these measurements, PBS&J made and recorded field observations during a rainfall event which occurred on March 7, 1983. The results of these observations largely confirmed the data obtained by the previous metered measurements. The rating curve developed for the gauging site based on these measurements is presented in Figure 2-5.

2.6 WATER QUALITY DATA REVIEW

On October 22, 1982, a meeting was held among technical staff of the City of Tampa, the DER and the consultants. The DER staff indicated that since improvements in the Cleveland Street Basin will not likely involve dredging and filling in any surface waters, it would not be necessary to conduct water quality sampling and analysis studies. Instead, the DER suggested a comparison of the peak outflow under existing conditions versus predicted peaks under various alternatives, and making some qualitative conclusions on how the differences in flow conditions may affect water quality. Such an evaluation is provided in subsection 4.5, as part of the evaluation of alternatives.



CLEVELAND STREET BASIN
CLEVELAND ST. AND CLARK AVE.
RATING CURVE

FIGURE 2 - 5

The evaluation in subsection 4.5 is limited to the amount of outflow generated and does not include the receiving lagoon. No water quality data was available for the lagoon. Field observations indicate that it is a body of water with very limited flushing and poor water quality, at least from an aesthetic standpoint.

In the process of becoming familiar with runoff conditions in the Tampa area, the consultant staff reviewed several draft reports produced in the NURP study:

- "Precipitation Quantity and Quality Data" (February, 1983)
- "Runoff Characterization - Water Quality and Flow Data" (March, 1983)
- "Control Testing - Water Quality and Flow Data" (March, 1983)

The runoff characterization study involved sampling at five (5) drainage basins which had a high degree of homogeneity of land use within them. The study basins represented the following land uses:

- High density residential (J. L. Young Apartments)
- Low density residential with fringes of institutional and commercial (Wilder Ditch Basin)
- Institutional and low density residential (North Jesuit High School Basin)
- Low density residential (Charter and Harding)
- High density commercial (Norma Park Ditch Basin)

The above report contains tables and the sampling results for various forms. The report does not summarize the data in a readily usable form (such as pollutant loads per unit areas).

The control testing report involved monitoring at the following types of control devices:

- Detention/retention ponds (water quantity and quality)
- Drainfield/trench systems (quantity only)
- Open bottom inlets (quantity only)
- Ditch systems (quantity only)

As in the characterization report, the data are presented in tabular form but have not been summarized in terms of pollutant loads per unit area, nor in terms of percentage of removal of pollutant input.

Thus, the above draft reports do not readily permit the calculation of annual pollutant loads from the Cleveland Street Basin. It should be noted that a

United States Geological Survey study that would facilitate the use of the NURP data for estimating pollutant loads has not been published, and therefore was not available. Accordingly, estimated pollutant loads for the Cleveland Street Basin have been determined from other sources and are presented as Table 2-1. Since very little change in land use is anticipated to occur in the basin these pollutant loads will be used as a base for order of magnitude pollutant load comparisons of the various alternative solutions developed.

TABLE 2-1

ESTIMATED RUNOFF POLLUTANT LOADS FROM
THE CLEVELAND STREET BASIN

LAND USE	AREA (ACRES)	SUSPENDED SOLIDS	BOD_5	TOTAL NITROGEN	TOTAL PHOSPHORUS
		Unit Pollutant Loadings, in pounds per acre per year			
Single Family		76.9	5.2	1.34	0.18
Multiple Family		258.9	19.8	9.69	0.71
Commercial		786.3	67.5	11.19	0.98
		Pollutant Loadings, in pounds per year			
Single Family	682.33	52,471.18	3,548.12	914.32	122.82
Multiple Family	45.42	11,759.24	899.32	440.12	32.23
Commercial	284.21	223,474.32	19,184.18	3,180.31	278.53
TOTALS	1,011.96	287,704.74	23,631.62	4,534.75	433.58

SOURCE:

Jettmar, R.V. et al. "Dynamic Water Quality Modeling in Southeast Florida, "Journal of the Environmental Engineering Division, American Society of Civil Engineers, February 1980.

Section 3

ANALYSIS OF THE EXISTING SYSTEM

3.1 SELECTION OF APPROPRIATE MODELS

One of the earliest tasks undertaken in the course of the Cleveland Street Basin study was the selection of the most appropriate model, or combination of models to be used for simulation of the hydrology and hydraulics of the study area. Final model selection was accomplished by identifying the characteristics of the basin which establish specific model requirements, reviewing the capability of the range of available models to satisfy those requirements and, through use of an evaluation matrix, determining the most appropriate models.

The specific drainage features of the Cleveland Street Basin which dictated the criteria for the models selected include:

- A complicated, closed conduit drainage network.
- A drainage network sensitive to tailwater conditions.
- The possibility of reverse flow occurring within portions of the system.
- Surcharging within the system.
- Variation in subbasin lag times and time displacement of hydrograph peaks resulting from the linear shape of the basin.
- Looped and parallel pipe systems.

Other factors considered in the model selection process were; availability of the models on a non-proprietory basis, documentation, proven reliability and the compatibility of input requirements with existing, available data.

Computer models considered for the analysis of the Cleveland Street Basin were:

- HEC-1 with Kinematic Routing Option
- Stormwater Management Model (SWMM)
- University of Cincinnati Model
- ILLUDAS (formerly, Road Research Lab Model)
- SPEC Storm Drainage Model
- SCS TR-20
- EXTRAN

- HEC-2
- RUNQUAL

The above list of models was further screened based on a review of user manuals, discussions with prior users and available technical papers concerning past model applications. This review resulted in narrowing the list of candidate models for the Cleveland Street Basin to five; SWMM, EXTRAN, RUNQUAL, TR-20 and HEC-1. The significant features of these five models are tabulated in the Model Summary shown in Table 3-1.

The final assessment of the most appropriate models was made using the evaluation matrix shown in Table 3-2.

As a result of this evaluation it was determined that hydraulic simulation of the Cleveland Street Basin would be accomplished using the EXTRAN model and basin hydrology should be developed with HEC-1 utilizing the SCS Dimensionless Hydrograph depicted in Figure 3-1. As can be seen from Table 3-2, EXTRAN is the only available model considered which satisfies all of the hydraulic criteria established for model selection. HEC-1 was chosen because of the limited amount of hydrologic data available for the study area and the model's flexibility in generating inflow hydrographs.

Documentation is excellent for both models and this will facilitate their use by City staff after the study has been completed.

3.2 STORM PARAMETERS

As noted in Section 1.1, the Cleveland Street Basin study is part of a comprehensive City-wide study. In order to maintain consistency in the methodology used and conclusions reached in all of the on-going and future analysis of basins throughout the City, a joint meeting was held with the Stormwater Management Division staff on December 13, 1982 to establish a uniform set of storm parameters to be used during the implementation of this program.

During that meeting, the specific storm parameters discussed and the agreements reached concerning criteria to be utilized in these studies were as follows:

3.2.1 Source of Rainfall Data

Five different sources were available for data concerning the relationship of rainfall intensities, frequencies and durations in the Tampa region. These consisted of the FDOT Rainfall Curves for Zone 6, United States Weather Bureau Technical Paper 40, the Pinellas County Rainfall Report, Curiosity Creek Rainfall Analysis and the Rainfall Quantity Analysis (original report and supplementary data) prepared by the City's consultants for the ongoing NURP study.

Because of the widespread use and the acknowledged reliability of the FDOT curves, this source was chosen as the approved source for rainfall data.

TABLE 3-1 - MODEL SUMMARY

Model	Advantages	Disadvantages	Applicability
SWMM	° Comprehensive ° Good documentation ° Users group ° Frequent updates/improvements ° Quanity and quality addressed ° Has been tested extensively ° Tidal flow can be simulated	° Size ° Complexity ° Large data requirement ° High run time/cost	° Where a simpler model will not work due to size and complexity (i.e., hundreds of pipes and hydraulic controls)
EXTRAN	° Backwater effects are calculated (Unlike runoff and transport blocks) ° Handles flow reversal, surcharge ° Handles looped connections	° Complex ° No quality	° Where complex pipe systems need to be analyzed
RUNQUAL	° Relatively simple ° Can handle quanity and quality ° Allows evaluation of peak flow at moderately designed elements (resize option)	° Necessary to plot inverts for pipes and channels and hydraulic units and determine predominent scope for each unit ° Will only take circular pipes - Equiv. diameters have to be calculated for others	° Preliminary design - pipes, channels, etc., with moderate complexity
FIPC	° Simple ° Quick ° Fairly accurate	° Necessary to re-enter data for each section before re-running. Could be re-written to eliminate this	° Quick & accurate hydraulic profiles on a segment-by-segment basis
TR-20	° Good documentation ° Widely used	° Routing for pipes requires stage-storage input	° Hydrology ° Routing for non-pipe systems
HEC-1	° Good documentation ° Support ° Widely used	° TR-20 type convex routing option should be used for "what-if" simulation only and not detailed designs	° Hydrology ° Routing
HEC-2	° Good documentation ° Support ° Widely used ° Handles irregular cross-sections	° Relatively large amount of data coding required	° Backwater analysis

TABLE 3-2

MODEL EVALUATION MATRIX

CLEVELAND STREET BASIN MODEL CRITERIA

HYDRAULIC CRITERIA:	SWMM TRANSPORT	SWMM EXTRAN	RUNQUAL
° Simulates complicated, closed pipe networks	*	*	
° Backwater conditions		*	
° Reverse Flow		*	
° Surcharging		*	
° Looped Systems	*	*	
° In-line Storage	*	*	*
° Off-line Storage	*	*	*

HYDDROLOGIC CRITERIA:	SWMM RUNOFF	RUNQUAL	TR-20	HEC-1
° Routing Methods				
-Convex Methods (SCS)	*		*	*
-Kinematic Wave Method	*			*
° Interception/Infiltration				
-Initial & Uniform Loss		*		*
-Exponential Loss Rate	*			*
-SCS Curve Numbers			*	*
° Depression Storage	*			*
° Relative Simplicity of Input Data			*	*

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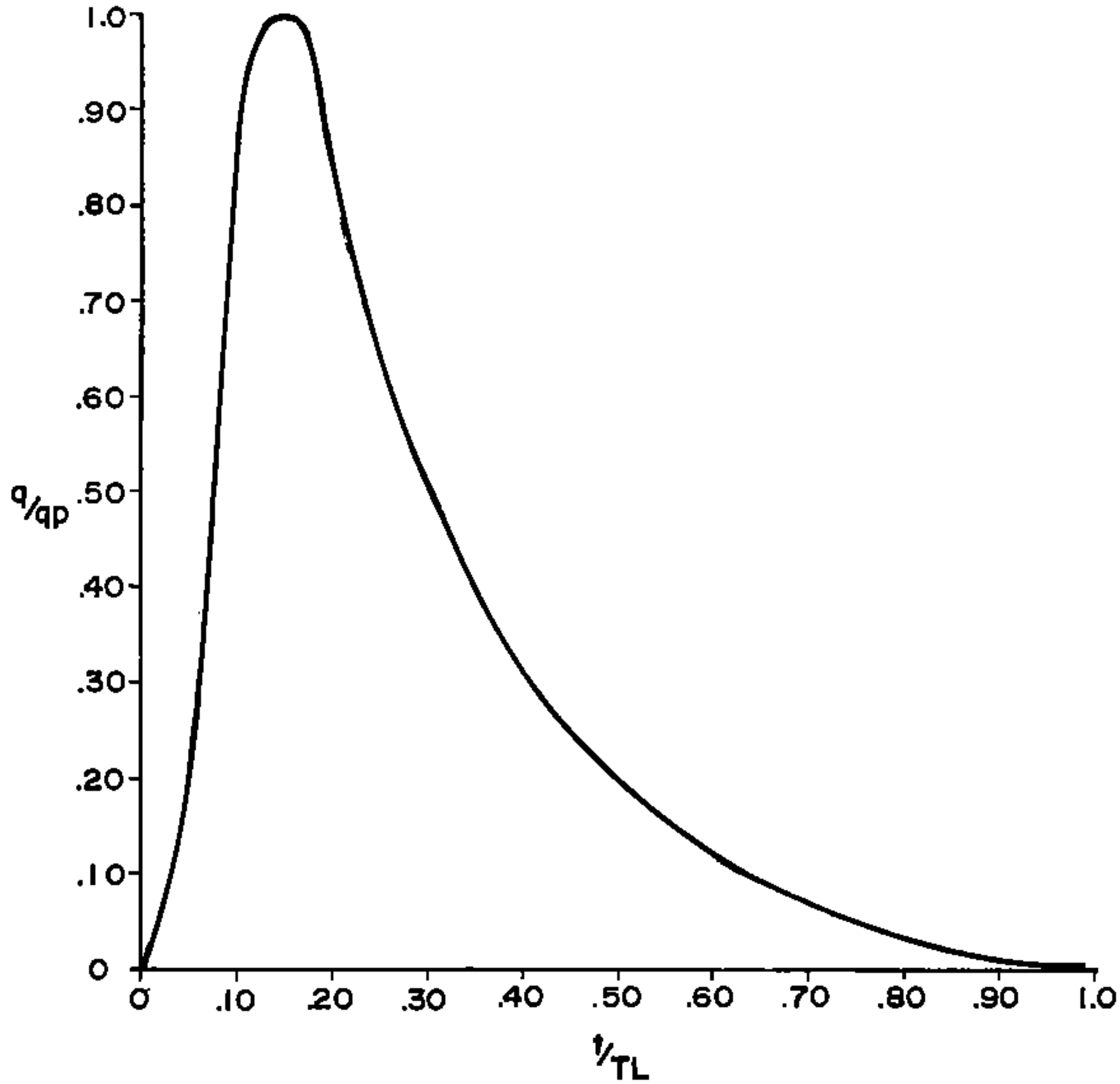


CLEVELAND STREET BASIN
DIMENSIONLESS HYDROGRAPH

FIGURE 3-1

3.2.2 Recurrence Intervals

A major decision in the selection of storm parameters is that of recurrence intervals. This decision determines, to a large extent, the level of protection against flooding that will be provided by the recommended improvements in the drainage system. Recurrence intervals used for drainage design of residential streets are usually two to three years while somewhat more intense (less frequent) storms are generally applied for major arterials. In the City of Tampa a five-year frequency storm is the accepted criteria for roadway drainage design. Therefore, this recurrence interval was agreed upon as the criteria to be used for conduit system design in Cleveland Street and other ongoing studies. In order to provide additional protection, it was further established that storms with a recurrence interval of 25 years would be applied to major detention ponds and 50 years to retention ponds.

For the Cleveland Street Basin only the 5 year recurrence interval storm was considered since the basin is predominantly a closed conduit conveyance system and therefore can be classified as a conveyance dependent basin as opposed to a storage or volume sensitive basin. Any detention facilities proposed for the basin will be viewed as secondary improvements to the existing primary conveyance system.

3.2.3 Storm Durations

An attempt was made to select a standard storm duration to be used in all drainage studies being prepared for the City. Durations from three to seventy-two hours were considered. However, after a lengthy discussion, it was agreed that the critical storm duration should be established for each individual basin based on it's unique hydrologic and hydraulic characteristics.

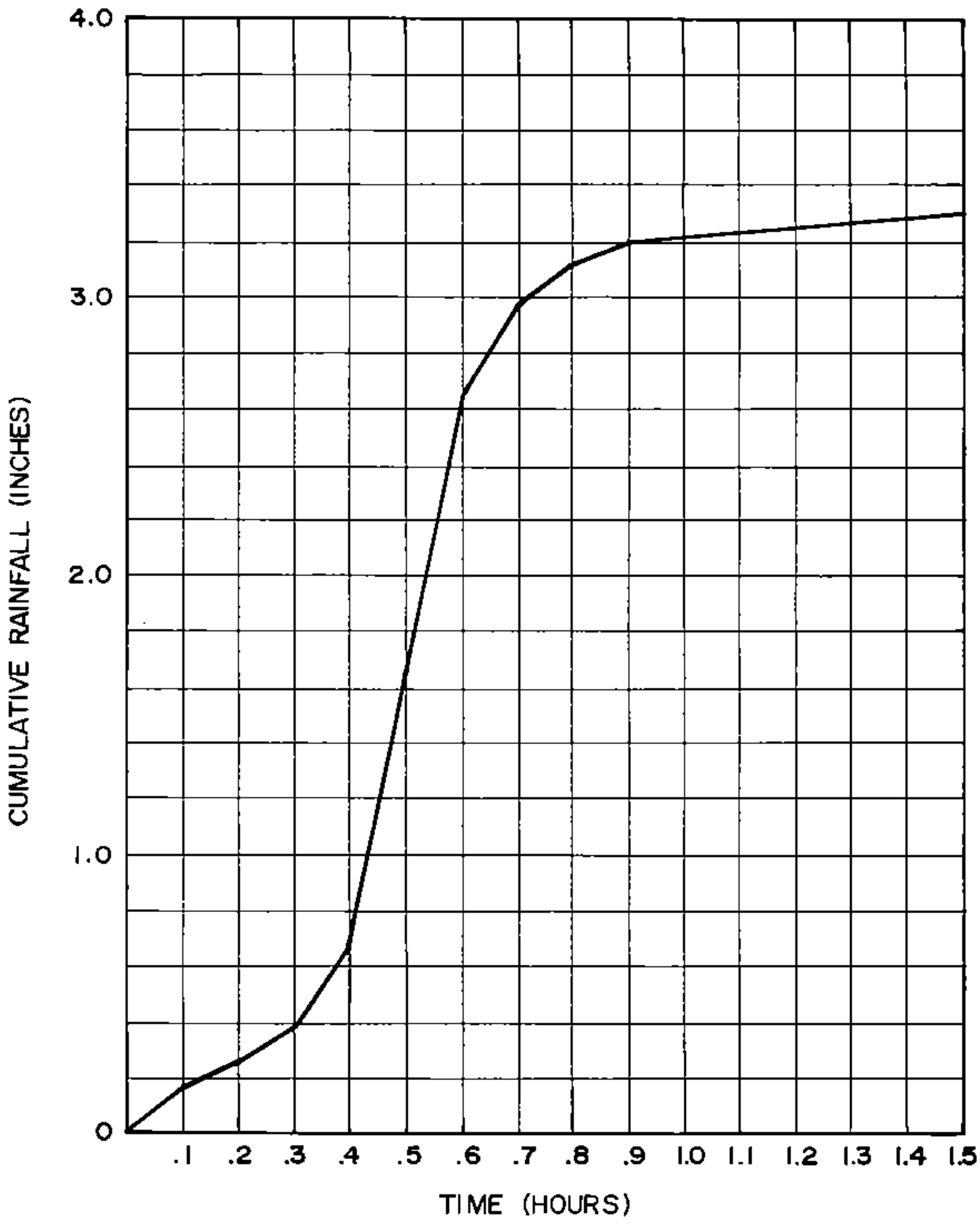
For the Cleveland Street Basin storm durations of one-half hour, one hour, one and one-half hours, two hours, three hours and six hours were simulated during the calibration phase. Based on the simulation results for these duration events the one and one-half hour duration was determined to be the appropriate critical duration event.

3.2.4 Rainfall Distribution

Several rainfall time distribution patterns were considered for use in the City drainage studies. These included SCS Rainfall Distributions, Huff Rainfall Distributions, distributions found in the Curiosity Creek Report and the pattern developed by the City's consultants for the Tampa NURP Study. After some discussion, it was agreed that, since the pattern used in the NURP Study was based on a statistical evaluation of every significant recorded event in the Tampa area over a 22-year period, this source provided the most reliable rainfall distribution data currently available. This pattern is presented in Figure 3-2 for the 5 year recurrence interval.

3.3 TABULATION OF CONVEYANCE SYSTEM DATA

The initial source of data used by PBS&J to inventory the existing drainage facilities in the Cleveland Street Basin was the Drainage Atlas maps maintained by the City Stormwater Management Division. These maps indicate



CLEVELAND STREET BASIN
CUMULATIVE RAINFALL HYETOGRAPH
5 YEAR RECURRENCE INTERVAL

FIGURE 3-2

the general location and size of conduits within the system as well as the direction of overland flow.

In order to obtain more detailed information about these facilities, PBS&J researched the design and as-built records available in the files of the City of Tampa and, in some cases, the FDOT. Utilizing these records, an Existing Facilities Map was developed at a scale of 1" = 200' indicating the location, size, length and invert elevation of all drainage facilities 36 inches in diameter or larger throughout the system. Wherever conduits less than 36 inches in size were felt to be significant in the analysis of the overall system, they were also shown to this level of detail. The Existing Facilities Map is presented in Figures 3-3W and 3-3E. In cases where detailed as-built information was not available, appeared questionable or was not consistent with cursory field observations, detailed field observations were made and the map of existing facilities adjusted accordingly. In addition, field surveys were conducted to verify the location and elevation of major system components and to provide answers to questions which could not be adequately resolved by field observation. During this stage of investigation, no attempt was made to identify or locate utility conflicts or other obstructions in the system.

3.4 PREPARATION OF MODEL INPUT

3.4.1 Hydrologic Input Data Required for HEC-1

The HEC-1 model was used to generate runoff hydrographs for each subbasin in the Cleveland Street Basin system. In order to develop the input data required by HEC-1, the following series of evaluations was required:

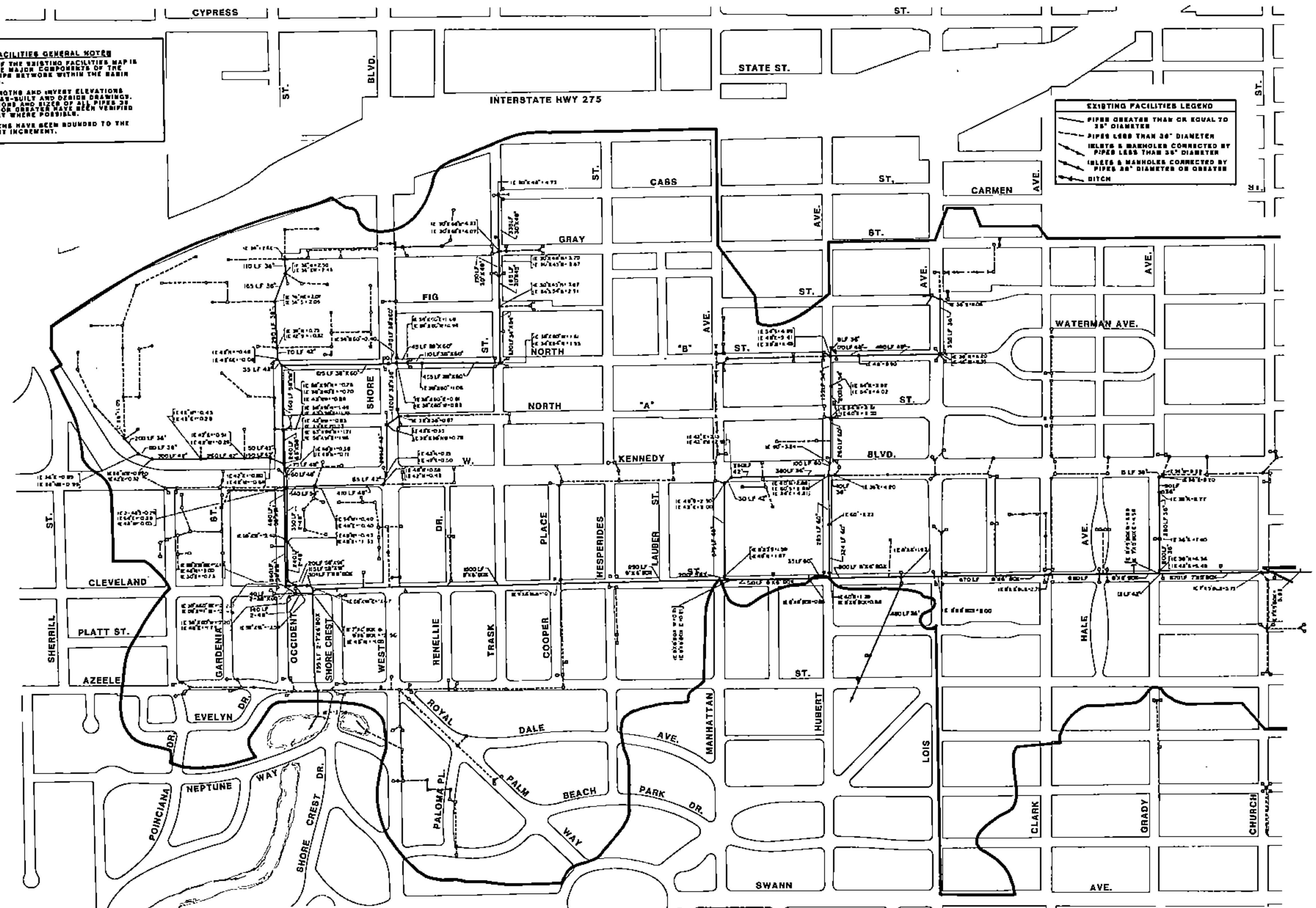
3.4.1.1 Delineation of Subbasins

The boundaries of subbasins within the study area were determined from available topographical data, supplemented by field observation and, in some instances, field survey. The delineation of the subbasins, overlayed on the Existing Facilities Map, is presented in Figures 3-4W and 3-4E. Utilizing the subbasin map, plotted at a scale of 1" = 200', the area of each subbasin was determined by planimeter measurement.

3.4.1.2 Determination of Impervious Areas

A vital input parameter in the determination of runoff from any subbasin is the percentage of the total area which is impervious. In order to make this determination, a rigorous analysis was made of each subbasin within the Cleveland Street Basin to measure, on recent aerial photographs, roof, pavement and other areas considered to be impervious to water. This procedure was applied to residential areas of varying density as well as to commercial and public or institutional sites within the basin.

As the study progressed, it became important to distinguish directly connected impervious areas; i.e., areas which contribute stormwater runoff directly to the drainage system, from non-directly connected areas. Non-directly connected areas are defined as impervious areas from which runoff must flow across adjacent pervious areas such as lawns or open fields prior to



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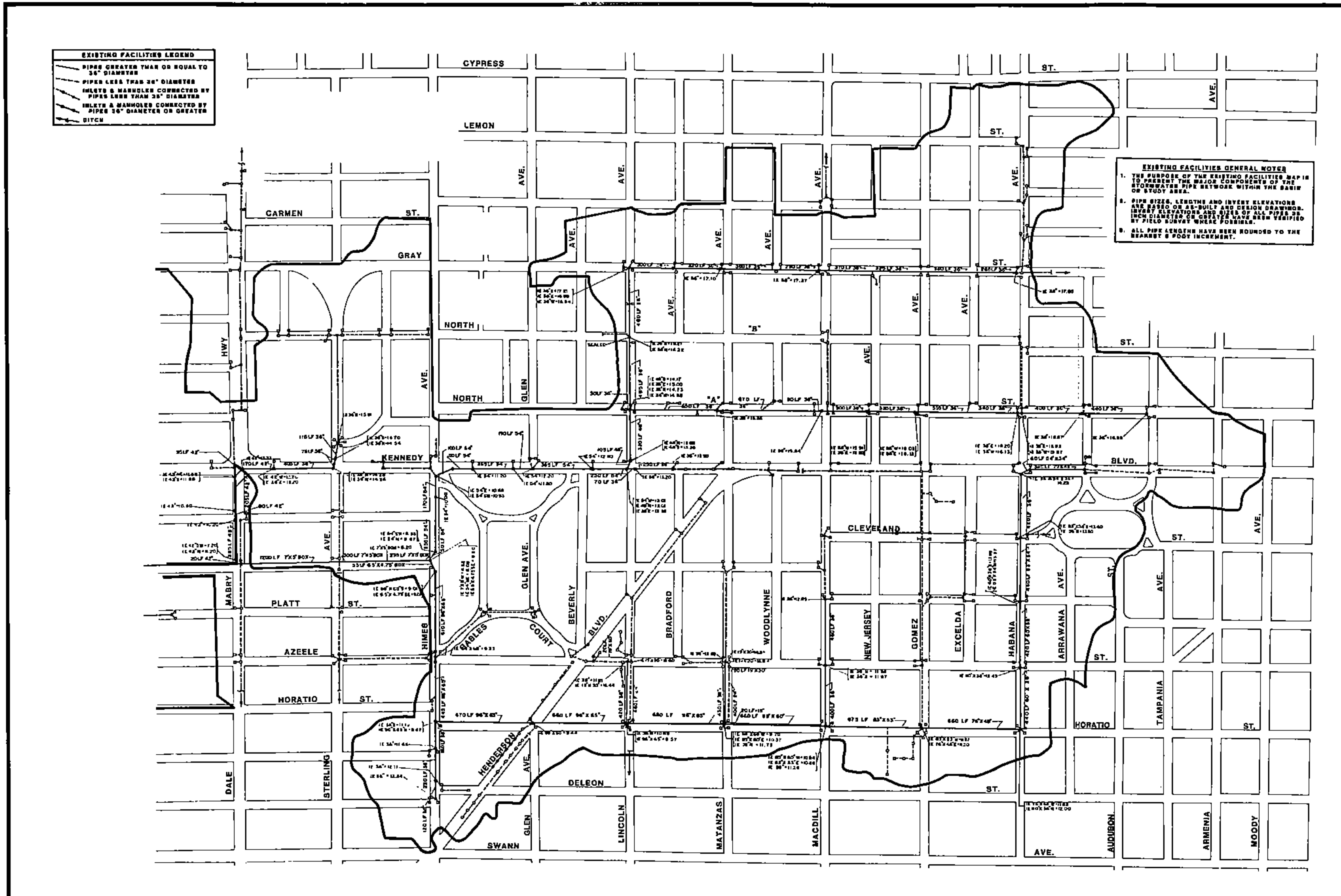
CLEVELAND STREET BASIN (WEST)

EXISTING FACILITIES

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

SCALE IN FEET



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CLEVELAND STREET BASIN (EAST)

EXISTING FACILITIES

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

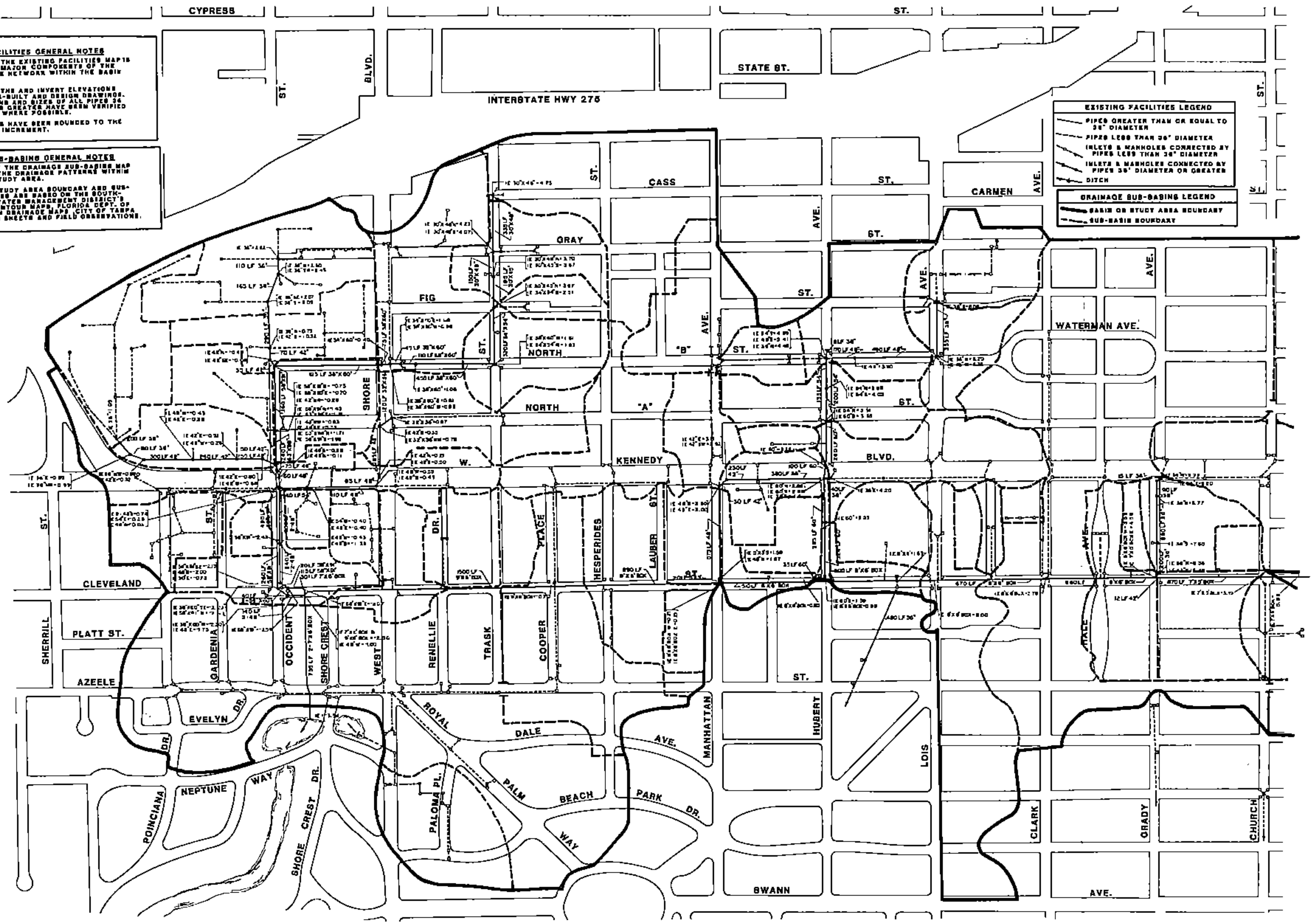
0 200 400 600
SCALE IN FEET

EXISTING FACILITIES GENERAL NOTES

1. THE PURPOSE OF THE EXISTING FACILITIES MAP IS TO PRESENT THE MAJOR COMPONENTS OF THE STORMWATER PIPE NETWORK WITHIN THE BASIN OR STUDY AREA.
2. PIPE SIZES, LENGTHS AND INVERT ELEVATIONS ARE BASED ON AS-BUILT AND DESIGN DRAWINGS. INVERT ELEVATIONS AND SIZES OF ALL PIPES 36 INCH DIAMETER OR GREATER HAVE BEEN VERIFIED BY FIELD SURVEY WHERE POSSIBLE.
3. ALL PIPE LENGTHS HAVE BEEN ROUNDED TO THE NEAREST 5 FOOT INCREMENT.

DRAINAGE SUB-BASINS GENERAL NOTES

1. THE PURPOSE OF THE DRAINAGE SUB-BASINS MAP IS TO PRESENT THE DRAINAGE PATTERNS WITHIN THE BASIN OR STUDY AREA.
2. THE BASIN OR STUDY AREA BOUNDARY AND SUB-BASIN BOUNDARIES ARE BASED ON THE SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT'S 1981 AERIAL CONTOUR MAPS, FLORIDA DEPT. OF TRANSPORTATION DRAINAGE MAPS, CITY OF TAMPA DRAINAGE ATLAS SHEETS AND FIELD OBSERVATIONS.



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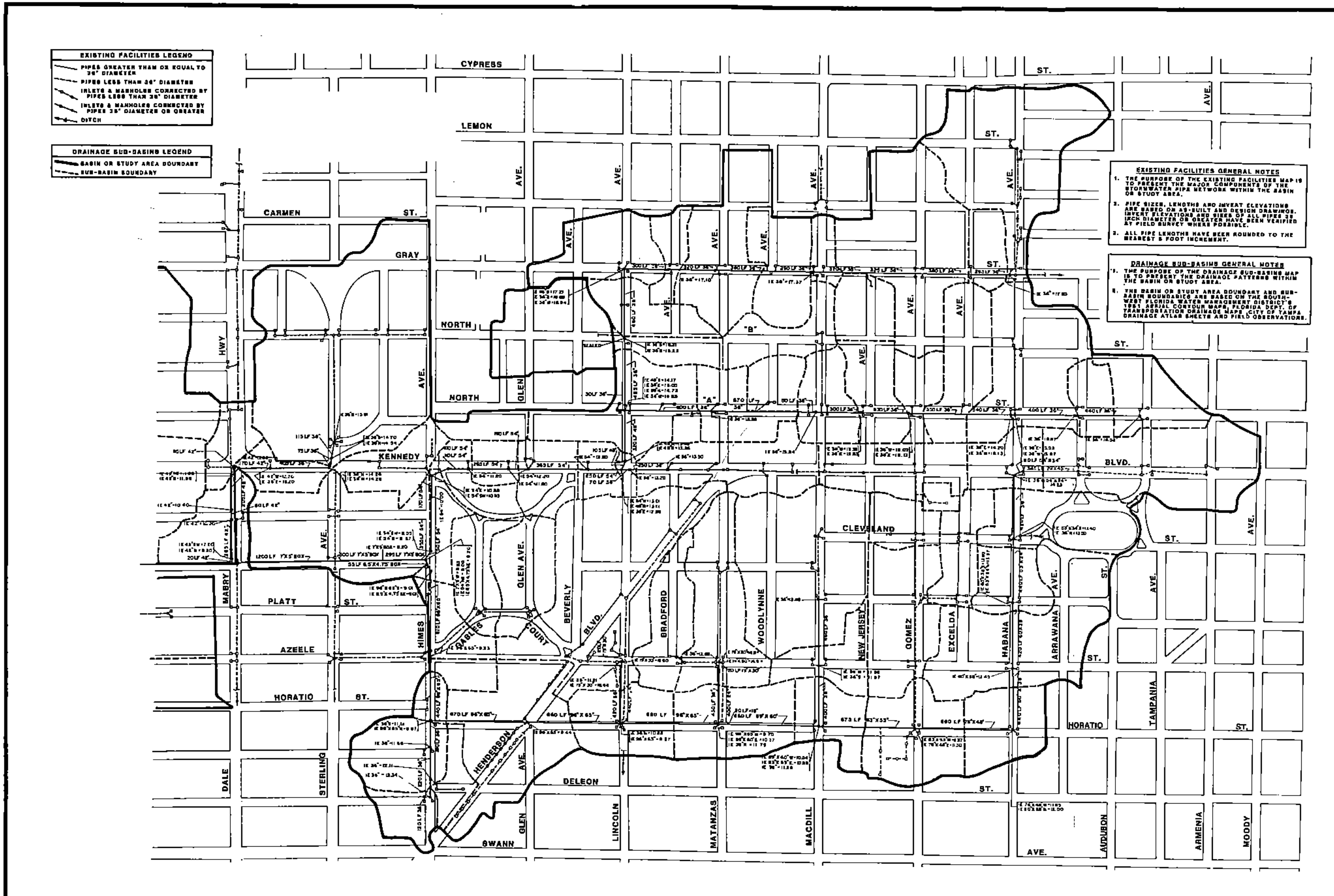
CLEVELAND STREET BASIN (WEST)
DRAINAGE SUB-BASINS
EXISTING FACILITIES

CITY OF TAMPA FLORIDA

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

0 200 400 600
SCALE IN FEET



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CLEVELAND STREET BASIN (EAST)
DRAINAGE SUB-BASINS
EXISTING FACILITIES

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

0 200 400 600
SCALE IN FEET

contributing to the drainage system. Once these areas were measured from aerial maps of the study area, the effective percent imperviousness was estimated for each subbasin.

3.4.1.3 Time of Concentration

Times of concentration were developed for each subbasin based on surface conditions, length of overland flow, internal conduit systems and subbasin topography. Overland flow times were computed using the FDOT velocity runoff curves and in some instances, where gutter flow occurs and profile grades were known, by Manning's Equation. Times of travel in smaller pipe systems not modeled were computed based on an average velocity of two and one-half feet per second.

3.4.1.4 Initial Abstraction and Uniform Loss Rate

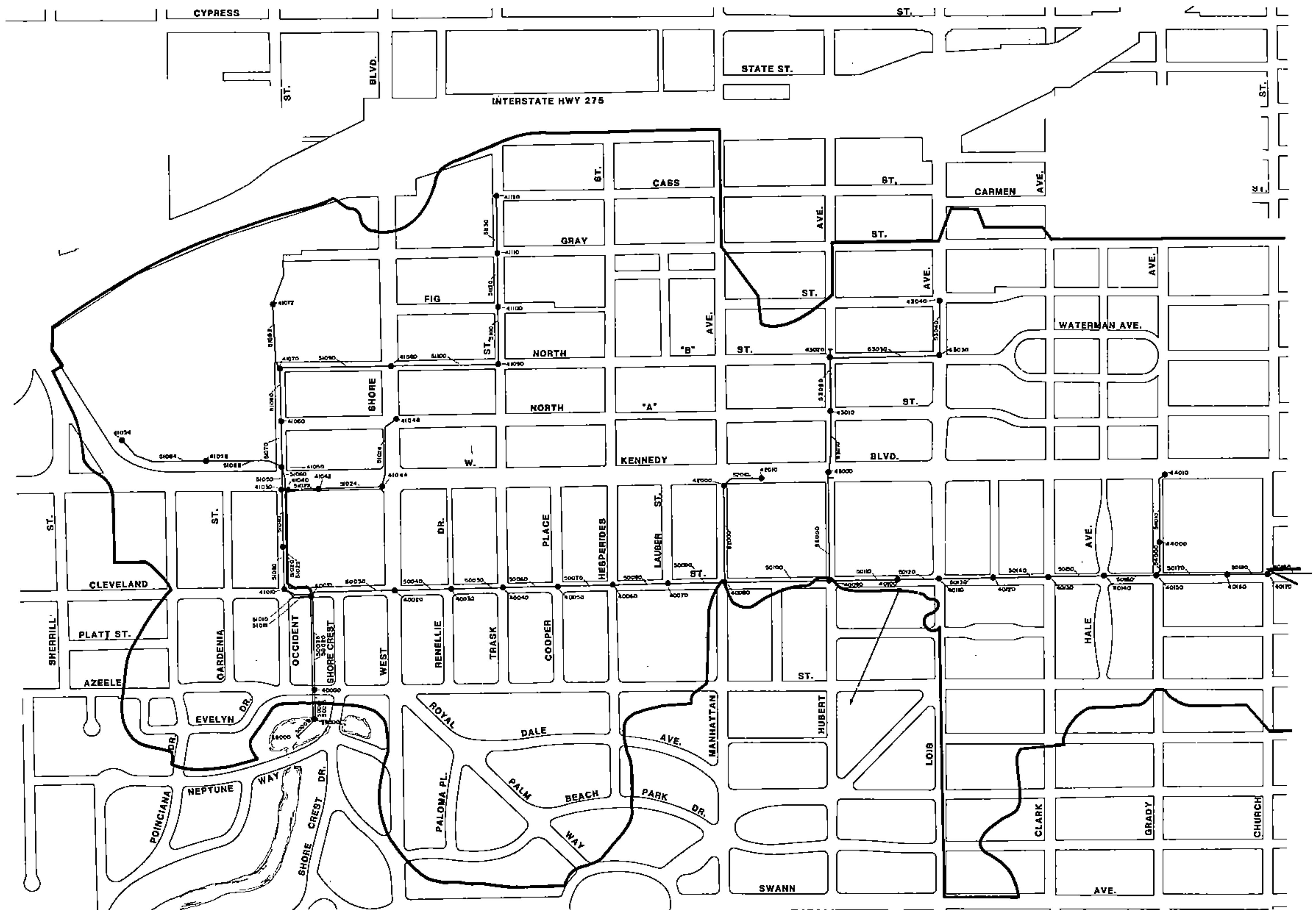
The HEC-1 model permits several different methods for evaluating precipitation losses in pervious areas. For the Cleveland Street Basin, the method chosen was the application of an initial abstraction (the volume of water, measured in inches, that is absorbed into the ground before any runoff occurs) and a uniform loss rate (the amount of water which is absorbed during each one hour period after the initial abstraction). The other available methods for estimating precipitation losses were generally based on an evaluation of various properties of existing soil types. Since no reliable soils data, such as Soil Conservation Service (SCS) soils maps, is available for the Cleveland Street Basin and much of the developed areas has been filled with imported materials, these methods could not be utilized. The initial values input for these two parameters were one half inch initial abstraction and one-half inch uniform loss rate. These values were adjusted during the calibration stage in order to match actual measured runoff volumes and flows.

3.4.2 Hydraulic Data Required for EXTRAN

As noted in Section 3.1, the EXTRAN model was used to simulate the hydraulics of the Cleveland Street Basin system. The input data required by EXTRAN consists of the physical characteristics of the components of the conveyance system, stormwater flows to the system from contributing subbasins and a definition of flow conditions at the beginning of the simlulated event.

3.4.2.1 Link/Node Map

A Link/Node Map representing the existing primary drainage facilities in the basin is shown in Figures 3-5W and 3-5E. This map was used as the basis for all data input preparation, and indicates the links (conduits) which make up the conveyance system and the nodes which occur at all stormwater inflow points and at all junctions. In the development of the Link/Node Map, some nodes were designated to receive the runoff contribution from more than one subbasin and some minor feeder systems were represented with a single node.



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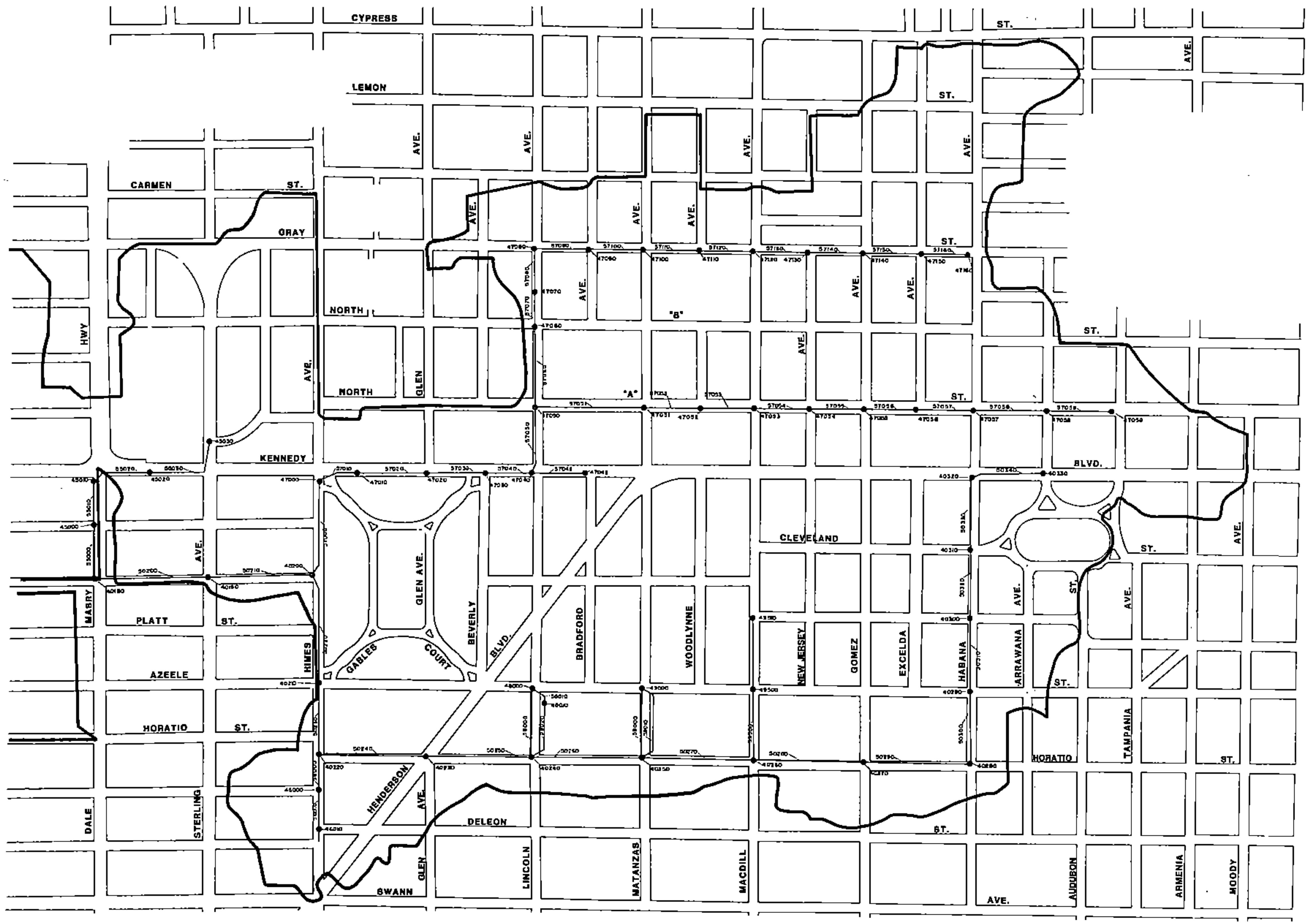
CLEVELAND STREET BASIN (WEST)

LINK/NODE MAP

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

0 200 400 600
SCALE IN FEET



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CLEVELAND STREET BASIN (EAST)

LINK/NODE MAP

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

SCALE IN FEET

3.4.2.2 Physical Characteristics of System Components

For each link and node in the system, data was input defining the length, size and shape of all conduits, invert elevations at each node and the elevations of conduits at each node relative to the node invert. Existing ground elevations at all nodes were also defined. These elevations were taken from the City's Draingae Atlas maps wherever such data was indicated. In those instances where elevations at junctions were not shown on the atlas sheets, they were determined from the SWFWMD contour maps.

3.4.2.3 Ambient Conditions

The hydraulic computations performed by EXTRAN take into account existing tailwater conditions, the volume of water stored in the system and any ambient flows at the beginning of the modeled event. For the model runs made for the Cleveland Street Basin, elevation +2.0 was input as the assumed water elevation at the outfall and initial water depths were input at the downstream nodes consistent with that assumed elevation. Ambient flows were all defined as zero.

3.4.2.4 Inflow Hydrographs

In order to input the runoff hydrographs generated by HEC-1 into the EXTRAN model, a special computer program was developed to convert the HEC-1 output into a format which could be accepted by EXTRAN. HEC-1 flow hydrographs at each input node were printed at three minute intervals for the simulation period. HEC-1 output consisted of the input node number and the corresponding tabulated flow hydrograph. To interface HEC-1 with EXTRAN it was necessary to develop a program that would rearrange this output to reflect the tabulated flows of all input nodes at each time step.

3.4.2.5 System Losses

Four types of energy losses were taken into account in the preparation of input data for EXTRAN; friction losses, connection losses, entrance losses at manholes and junctions and losses due to bends. Field observations of the age and condition of existing facilities were made as a basis for evaluating the appropriate "n" values to be used for friction losses. These values were then adjusted to account for losses anticipated due to connections, bends in the system, manhole entrance conditions and obstructions, which typically occur in urbanized storm sewer systems.

Losses associated with connections, bends and entrance conditions were estimated using the procedures presented in "Water Supply and Pollution Control", <u>Clark Viessman and Hammer</u>, 1971. For each link in the system an "n" value representing friction, bend, connection and entrance losses was computed usings Manning's Equation. These "n" values were then compared to the range of "n" values presented in "Open Channel Hydraulics", <u>Chow</u>, 1959, for concrete lined conduit sewer systems. In most instances the estimated "n" values did fall within the normal and maximum "n" value range presented by Chow for straight systems. Consequently, the "n" value range presented by Chow were deemed to be applicable to the Cleveland Street conduit system and were generally used except when it was felt previous estimates justified slightly greater values. Reference material for "n" value estimates are included in Appendix A.

During the calibration phase adjustments to the originally estimated "n" values for the Cleveland Street box culvert were required to gain calibration. These greater "n" values were felt to be attributed to numerous utility obstructions found to exist and a slightly rougher box floor.

3.4.2.6 Adjustments

Some adjustments to the input data were required to accommodate certain limitations of the EXTRAN model. In instances where pipe size increased within a link, the pipe was input as one size. This was accomplished by adjusting the length of the shorter pipe, keeping the "n" value constant, to a length that would yield the same hydraulic losses as those that would occur in the actual system.

In some cases, the actual length between nodes could not be input because it was too short. EXTRAN, which computes conditions in the system at specified time steps, requires that each conduit in the system be of sufficient length to satsify the equation "Maximum Time Step (in seconds) = $L/\sqrt{gD}$" where L = the length of the pipe in feet, D the depth of the pipe in feet and g is the gravitational acceleration. Wherever it was necessary to increase pipe lengths to satisfy this condition, an equivalent "n" value (the coefficient for friction losses in the pipe) was computed and input in order to assure that friction losses would not be distorted.

3.5 MODEL CALIBRATION

In order to verify the input data for the Cleveland Street Basin, a series of steps was taken to confirm that the computer model was operating correctly and that the results obtained from calibration runs of the model agreed with field measured data.

3.5.1 Verification of EXTRAN Model

The initial step in the calibration process was to verify the EXTRAN model as installed in the PBS&J system. This was accomplished by inputing a sample problem with hydraulic conditions similar to those known to exist in the Cleveland Street system and obtaining known, correct results. The sample problem used was a small system consisting of five conduits and a reservoir having free outfall. The rainfall event used was sufficiently small so that no surcharging occurred in the sample run. The time step used for this simulation was 20 seconds. The results obtained from this sample run were satisfactory.

The second step taken to verify the model was to simulate a minor event in the Cleveland Street system. The event chosen was one-half inch rainfall over a one hour period. No tailwater conditions were input. When this event was modeled using a ten second time step, it did not produce surcharging in the system and no problems were encountered.

Next, the same event was simulated, but in this run tailwater conditions were introduced by setting the existing water elevation at the outfall at elevation +2.0 and specifying an ambient water depth for all nodes with invert elevations lower than +2.0. As a result of this trial, it was found that the EXTRAN program does not automatically compute initial water volumes in the pipe system based on ambient depths. Consequently, the volume of water in the system at the beginning of the event is not considered when the program calculates the overall mass continuity of the system (the volume of stormwater entering, stored and leaving the system). However, a review of the simulation results revealed that this minor inadequacy in the program's accounting procedure did not affect the answers obtained for the Cleveland Street system. The results of this run also revealed that, when modeling a system with large conduits and appreciable ambient depths, minor instabilities can occur during low flow conditions.

Finally, the model was tested using a one and one-half inch rainfall over a one hour period. This event resulted in widespread surcharging throughout the system and serious model instability. Based on review of the EXTRAN program and discussions with Mr. Larry Rosener, one of the authors of EXTRAN, it was determined that this instability was caused by the large number of nodes surcharging simultaneously in the Cleveland Street system, the large conduit sizes within the system and the iterative methodology used by the program during surcharge.

One of the basic equations used by EXTRAN to compute heads and flows at each time step during simulation of an event is the relationship of head differential (Δh) to flow (Q) and the area of the water's surface (A_s) at each node; i.e., $\Delta h \propto Q/A_s$. When conduits are flowing partially full, EXTRAN computes the water surface area in each conduit and assigns half of this area to the node at either end. However, during surcharge, when the hydraulic profile is above the crown of the conduit, this surface area becomes zero and the equation can no longer be applied. When this condition occurs, EXTRAN switches to a different set of equations which approximate the solution through an iterative trial and error process in order to find a combination of heads and flows at each node that will provide a continuity balance throughout the entire system. This method is successful when applied to small systems, but due to the large number of variables in the Cleveland Street Basin, EXTRAN was unable to converge and yield accurate results.

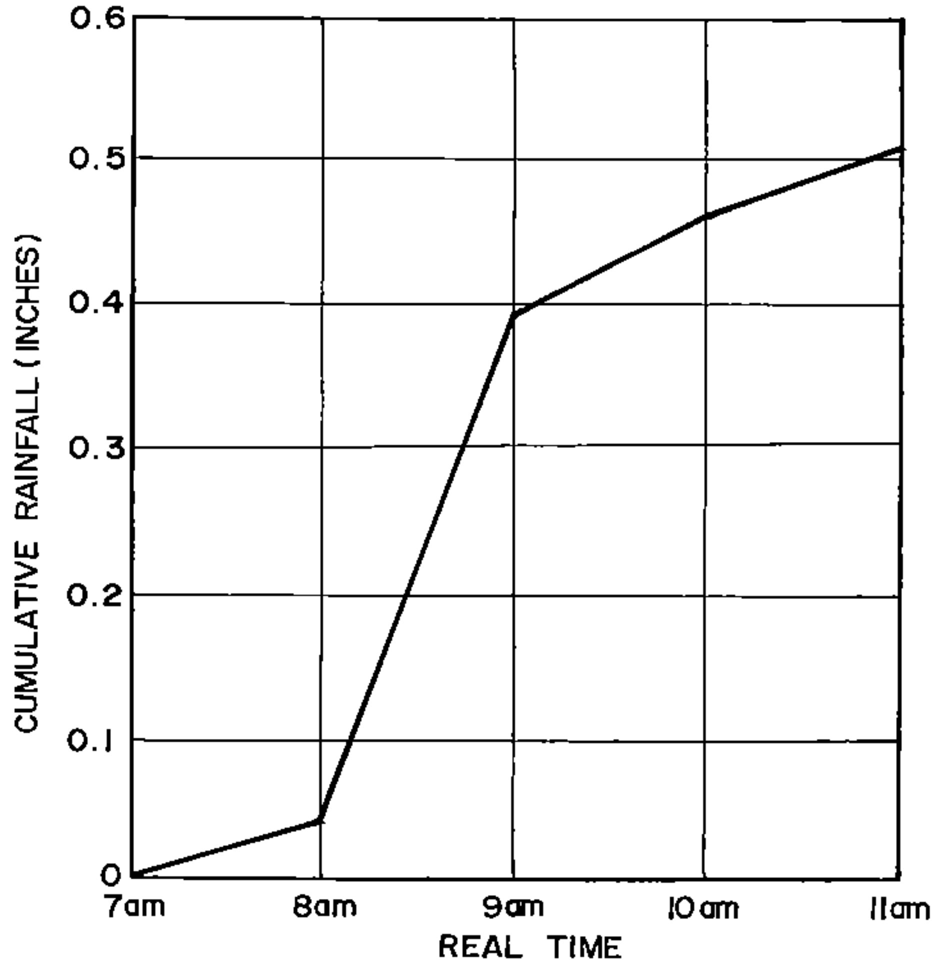
In order to alleviate this problem and eliminate the need for the model to solve by iteration, constant surface areas that function as small surge tanks were introduced at each node. This allowed the program to continue to use the basic equations. Initially the surface area input was 500 square feet at each node. Through a trial process, these areas were ultimately reduced to a minimum (as small as 50 square feet). A comparison of the results of the program at low flows, with and without the storage areas, indicated that the impact of introducing these areas was minimal in terms of simulated elevations and flows.

3.5.2 Calibration of Hydrologic Input Data

Field measurements taken during two actual storm events were utilized to calibrate the hydrologic and hydraulic input data used for the Cleveland Street Basin. The first of these events occurred on January 20, 1983, during which 0.51 inches of rain was recorded. The second event, on February 2, 1983, measured 1.7 inches. The hyetographs developed to represent rainfall distribution during these two events are shown in Figures 3-6 and 3-7.

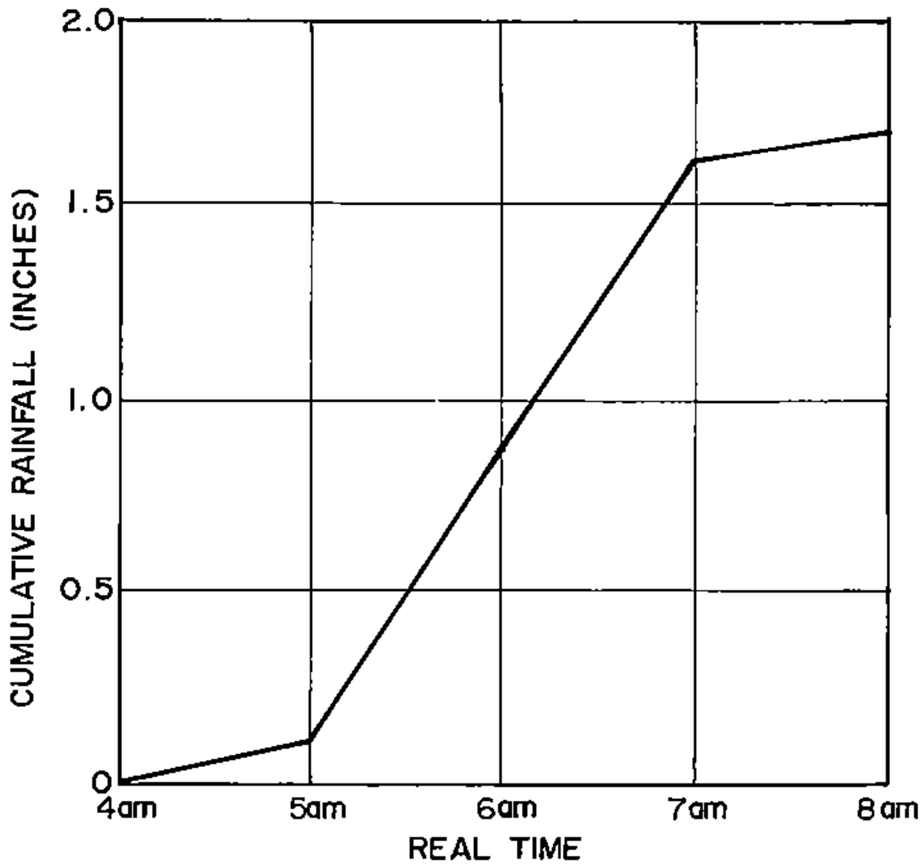
3.5.2.1 Precipitation Losses in Pervious Areas

Initially, the rainfall events used for calibration were simulated with the HEC-1 model using a value of one-half inch for both the Initial Abstraction



CLEVELAND STREET BASIN
JAN. 20, 1983 CALIBRATION EVENT
RAINFALL DISTRIBUTION

FIGURE 3-6



CLEVELAND STREET BASIN
FEB. 2, 1983 CALIBRATION EVENT
RAINFALL DISTRIBUTION

FIGURE 3-7

3.5.2.2 Impervious Areas

As noted in the discussion of input parameters in Section 3.4.1.2, it became apparent during calibration that the runoff contribution from indirectly connected impervious areas was significantly different than that from directly connected impervious areas. For the calibration events, using initial abstraction and uniform loss rate values presented in the previous section, numerous simulation runs were made to determine the quantity of runoff contribution from impervious areas. Results of these simulations revealed that no runoff from the non-directly connected impervious areas appears to have occurred.

Initially, directly connected impervious areas were input as contributing 100 percent of the rainfall which fell on them to the conveyance system. However, even with no contribution from pervious and indirectly connected impervious areas, the volume of runoff computed using all of the rainfall on directly connected areas exceeded measured volumes. Calibration was achieved by reducing the contribution of directly connected impervious areas to 85 percent for roadways, driveways, parking lots, etc. and 80 percent for roof tops. This slightly lower percentage of contribution from roof tops takes into account the effect of roof ponding.

3.5.3 Calibration of Hydraulic Input Data

Based on a cursory inspection of the age and condition of the conveyance facilities in the Cleveland Street Basin, together with an analysis of the anticipated effect of losses due to connections, bends and entrance conditions, "n" values in the range from .015 to .017 were initially estimated for the Cleveland Street box. However, a comparison of the simulated flows and water surface elevations computed using these values to measured data indicated that the actual effective "n" values were much higher. In order to obtain calibration, values in the range from .020 to .025 were required.

Because these "n" values appeared extremely high, they caused some concern about the accuracy of the field measurements against which the simulated results were being calibrated. In order to ascertain whether such values were realistic, the box culvert was field inspected from Clark Avenue to Occident Street, a distance of approximately one mile. In this reach, 18 utility conflicts ranging from 6 inches to 18 inches in diameter were observed. While the side walls and top of the box were found to be relatively smooth and in good condition, the floor of the box was observed to be quite rough in texture. Some minor factors which increased friction within the culvert, such as roughness at construction joints, tie rods extending into the box and barnacles on the sidewalls in tidal areas were also observed. In addition, silt up to one foot in depth was found in the area between Westshore and Occident. An analysis made of the probable effect of the obstructions in the box culvert and the roughness of the box floor indicated that, given the conditions observed, an effective "n" value of .022 was reasonable.

3.5.4 Calibration Results

Comparisons of the simulated and measured flows and elevations at the gauging site (located at the intersection of Cleveland Street and Clark Avenue) for the February 2nd event are presented in Figures 3-8 and 3-9, respectively. Figures 3-10 and 3-11 present similar comparisons at the same location for the event which occurred on January 20.

As can be observed, the simulated runoff volumes, peak flows and times of concentration (the point in time when peak flows occurred) are reasonably close to the measured data for these events. The apparent differences between simulated and measured results were felt to occur primarily because only the major facilities (generally, 36 inches or larger) in the system were modeled, the uniformity of rainfall throughout the basin assumed by the model is questionable for the actual event and because of the limitations of the model itself. Considering these factors, the input parameters derived through the calibration process were deemed to represent, within an acceptable degree of accuracy, the hydrologic and hydraulic conditions existing in the Cleveland Street Basin and to be suitable for use in the evaluation of proposed alternative solutions.

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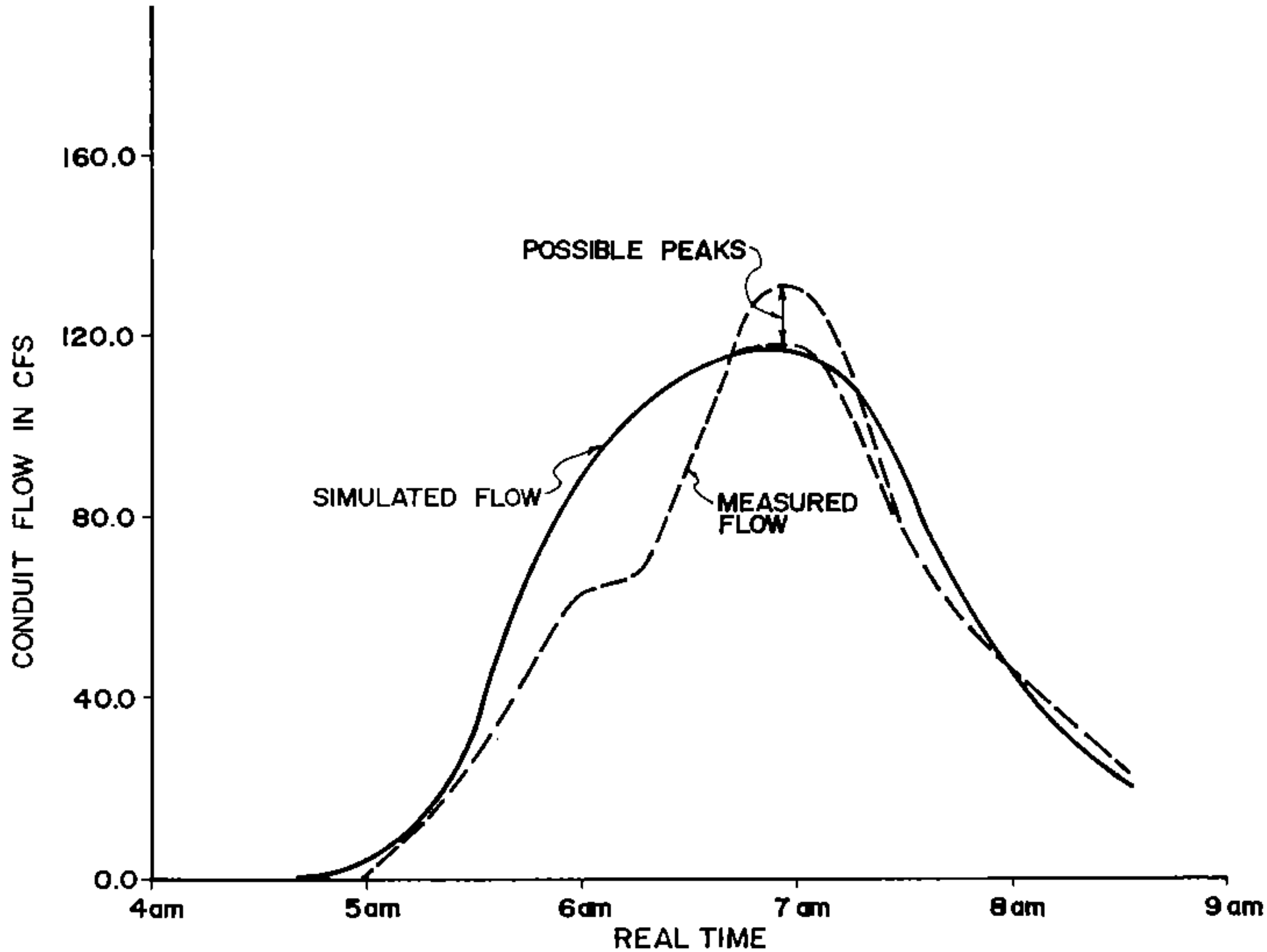
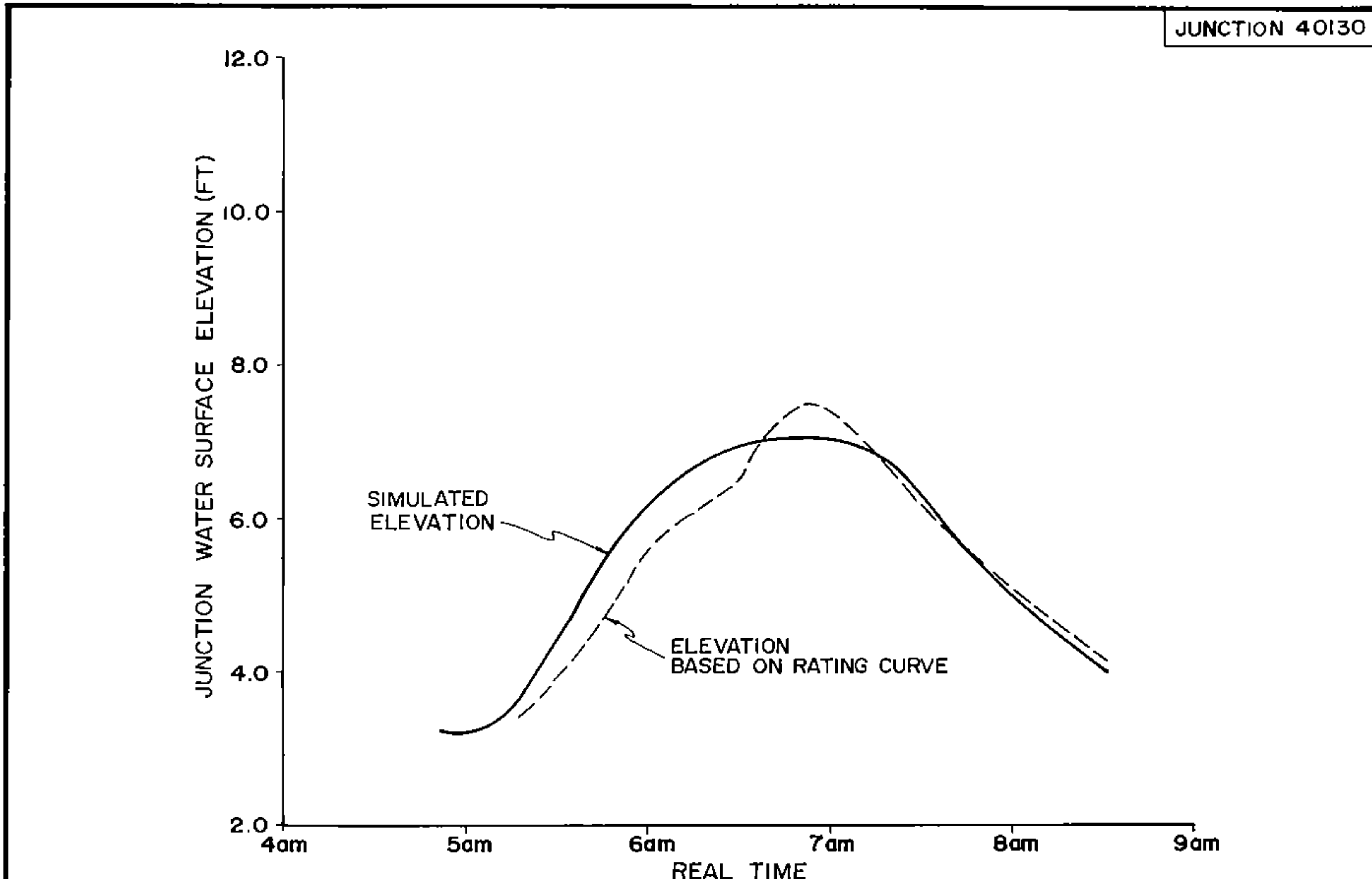


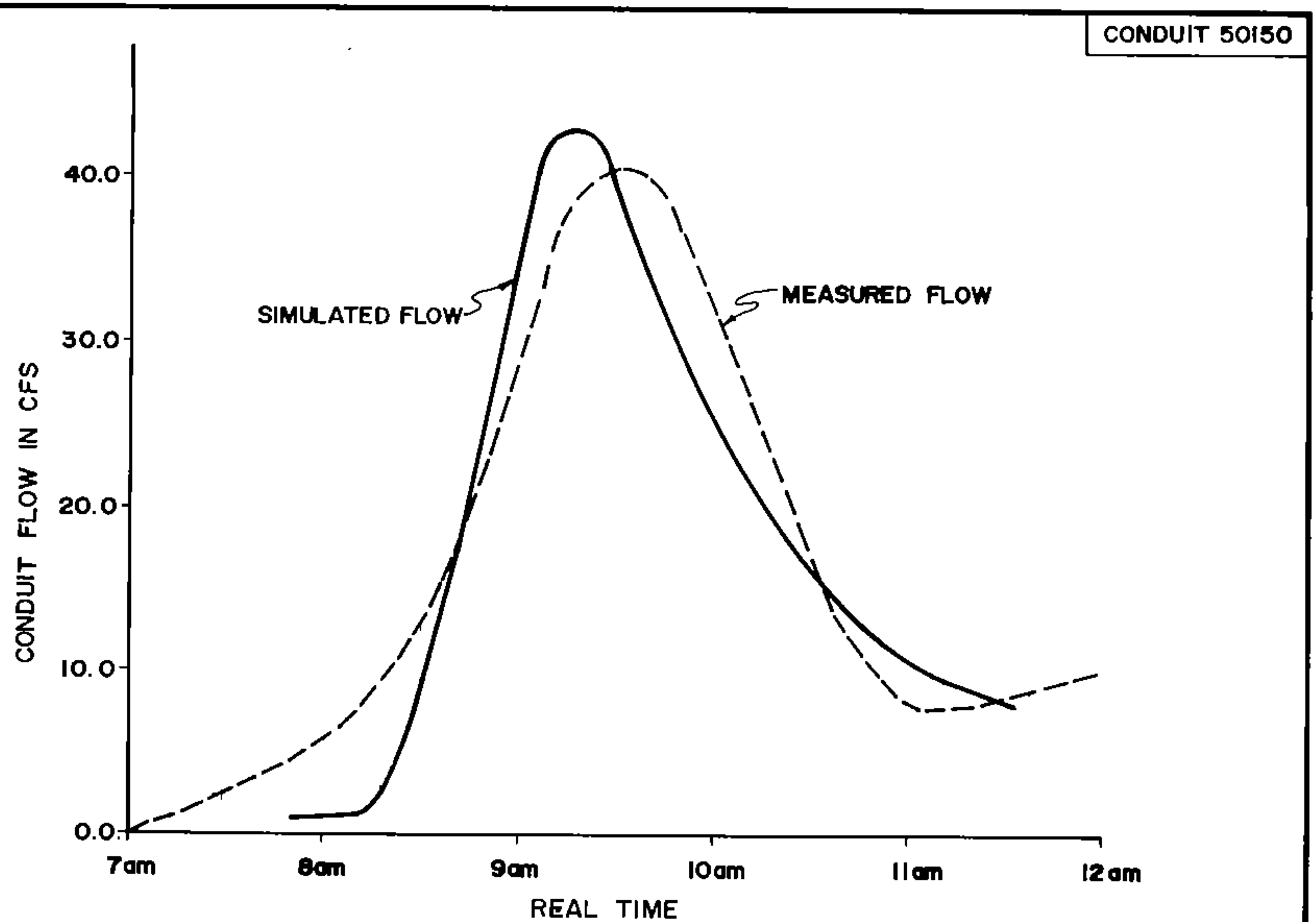
CLEVELAND STREET BASIN
FEB. 2, 1983 CALIBRATION EVENT
FLOW HYDROGRAPH

FIGURE 3-8



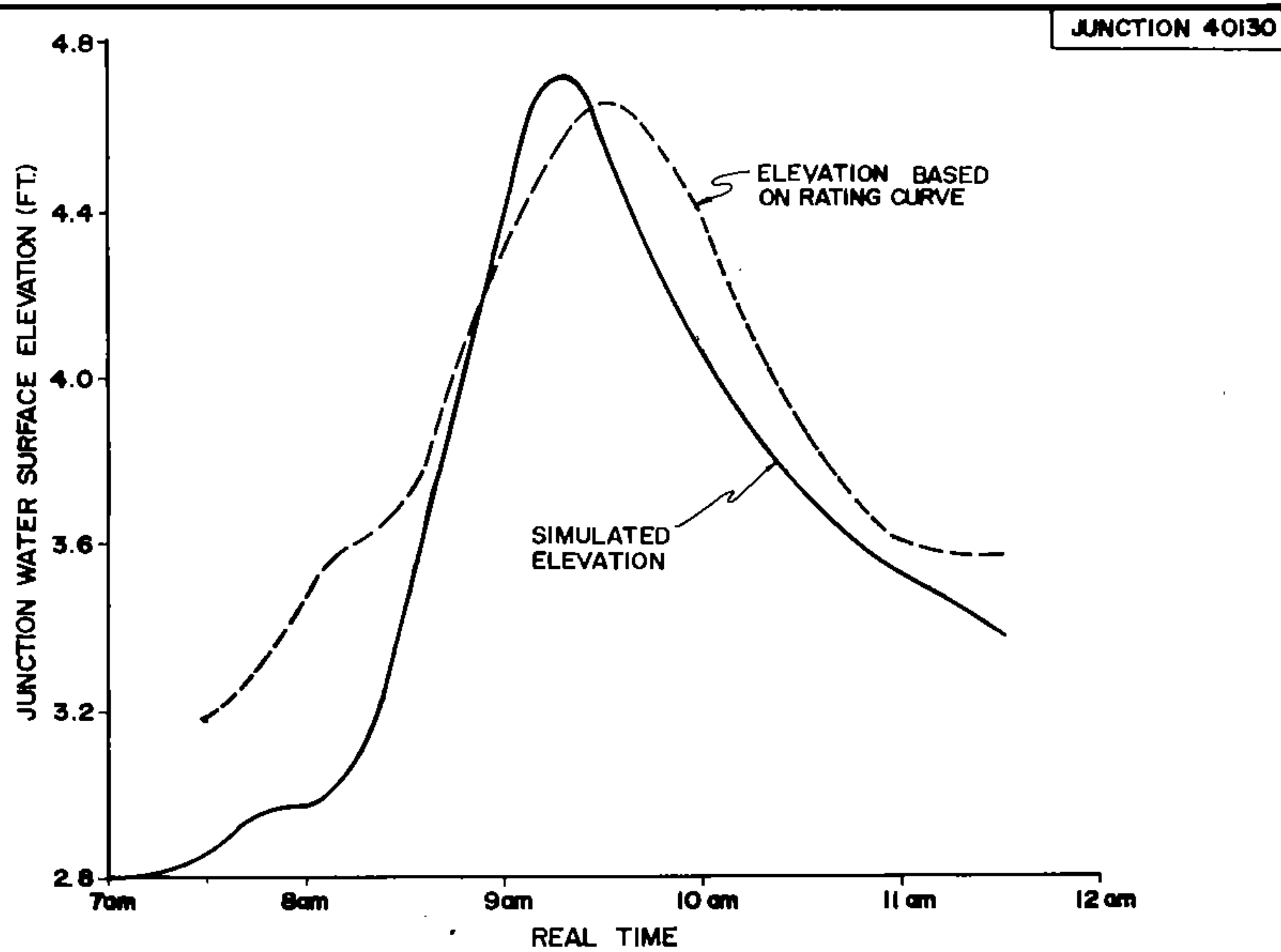
CLEVELAND STREET BASIN
FEB. 2, 1983 CALIBRATION EVENT
ELEVATION HYDROGRAPH

FIGURE 3-9



CLEVELAND STREET BASIN
JAN. 20, 1983 CALIBRATION EVENT
FLOW HYDROGRAPH

FIGURE 3-10



CLEVELAND STREET BASIN
JAN. 20, 1983 CALIBRATION EVENT
ELEVATION HYDROGRAPH

FIGURE 3-11

Section 4

ANALYSIS OF SYSTEM ALTERNATIVES

4.1 PROBLEM AREAS

Predicated on the historical records of flooding in the Cleveland Street Basin and the areas of most severe flooding identified by the computer model, five problem areas were established as the basis for developing proposed alternative system improvements. These five areas were:

- °Problem Area No. 1: The area north and south of Cleveland Street from Azeele to Kennedy Boulevard between the western boundary of the basin and Manhattan Avenue.
- °Problem Area No. 2: The area within the Cleveland Street Basin lying south of Kennedy Boulevard and east of Himes Avenue.
- °Problem Area No. 3: The areas just north and south of Cleveland Street between Manhattan and Church Avenues.
- °Problem Area No. 4: The area within the Cleveland Street Basin lying north of Kennedy Boulevard and east of Himes Avenue.
- °Problem Area No. 5: The area north of problem area No. 1 between Kennedy Boulevard and I-275 from the northward extension of Occident to Manhattan Avenue.

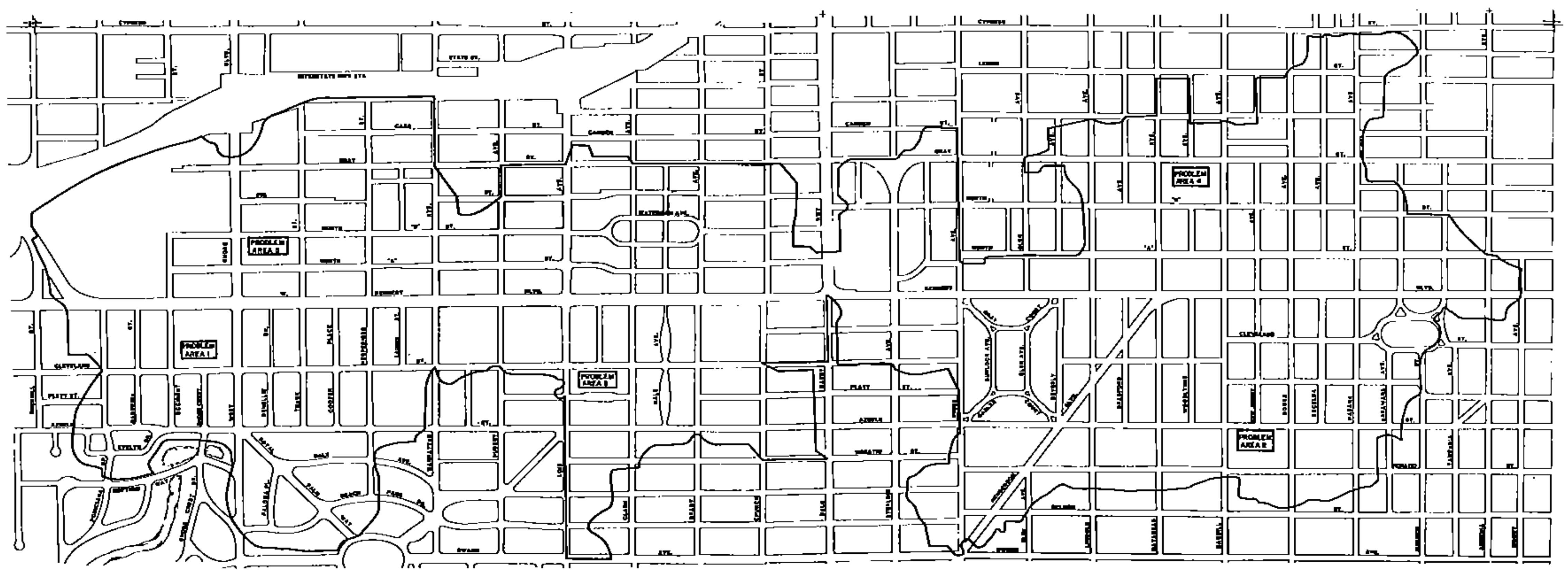
Figure 4-1 displays the approximate locations of these areas.

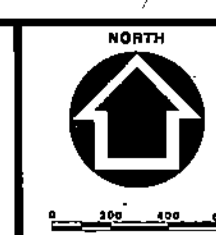
4.2 SYSTEM IMPROVEMENT TYPES AVAILABLE

Four types of system improvements appeared to be available to alleviate the severe flooding occurring within the basin. Those considered included:

- New conveyance facilities, paralleling or replacing the existing system.
- Detention facilities.
- A separate outfall for the system which serves the Westshore shopping center and surrounding area.
- Improvement of flow conditions in the existing system by reducing friction losses and eliminating obstructions.

Of these, new conveyance facilities and detention facilities are the only improvement types which have the potential to effectively eliminate flooding throughout the basin during a design storm event. However, the separate outfall for Westshore and improvement of the flow conditions in the existing system both provide significant relief with relatively minor cost. For that





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CLEVELAND STREET BASIN

PROBLEM AREAS

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

reason, a series of alternative solutions utilizing combinations of the four improvement types available were identified, analyzed with the computer model and assessed in terms of water quality impact and cost.

4.3 ALTERNATIVES CONSIDERED

Altogether, twelve system improvement alternatives were analyzed. These alternatives were developed by applying the four improvement types discussed above to the five problem areas which had been identified. They represent a spectrum of possible solutions which range from minor improvements that will reduce, but not eliminate, flooding in any problem area to major improvements needed to eliminate all flooding within the Cleveland Street Basin during a design storm event.

Alternative No. 1, shown in Figure 4-2, consists of a complete new closed conduit system paralleling the existing system. This alternative would eliminate flooding throughout the basin, but was the most costly alternative considered.

Alternative No. 2, presented in Figure 4-3, would also eliminate flooding throughout the basin. This solution is comprised of a system of detention basins with secondary, localized pipe improvements.

Alternative No. 3 was analyzed to determine the benefit that could be derived from improving flow conditions in the existing box culvert in Cleveland Street by lining the floor of the box and eliminating obstructions. In this alternative, no other improvements were considered. The computer model run of Alternative No. 3 indicated that only minor reduction in flooding would result from implementation of this improvement alone.

Alternative No. 4, shown in Figure 4-4, combined improvement of the Cleveland Street box flow conditions with the construction of a new, separate outfall for the Westshore area. This alternative would eliminate flooding in Problem Area 1, but would have little impact elsewhere in the basin.

Alternative No. 5, presented in Figure 4-5, includes the new Westshore outfall, improvement of the Cleveland Street box flow conditions and detention facilities with minor pipe improvements east of Himes Avenue. This alternative would eliminate flooding in Problem Areas 1, 2 and 4.

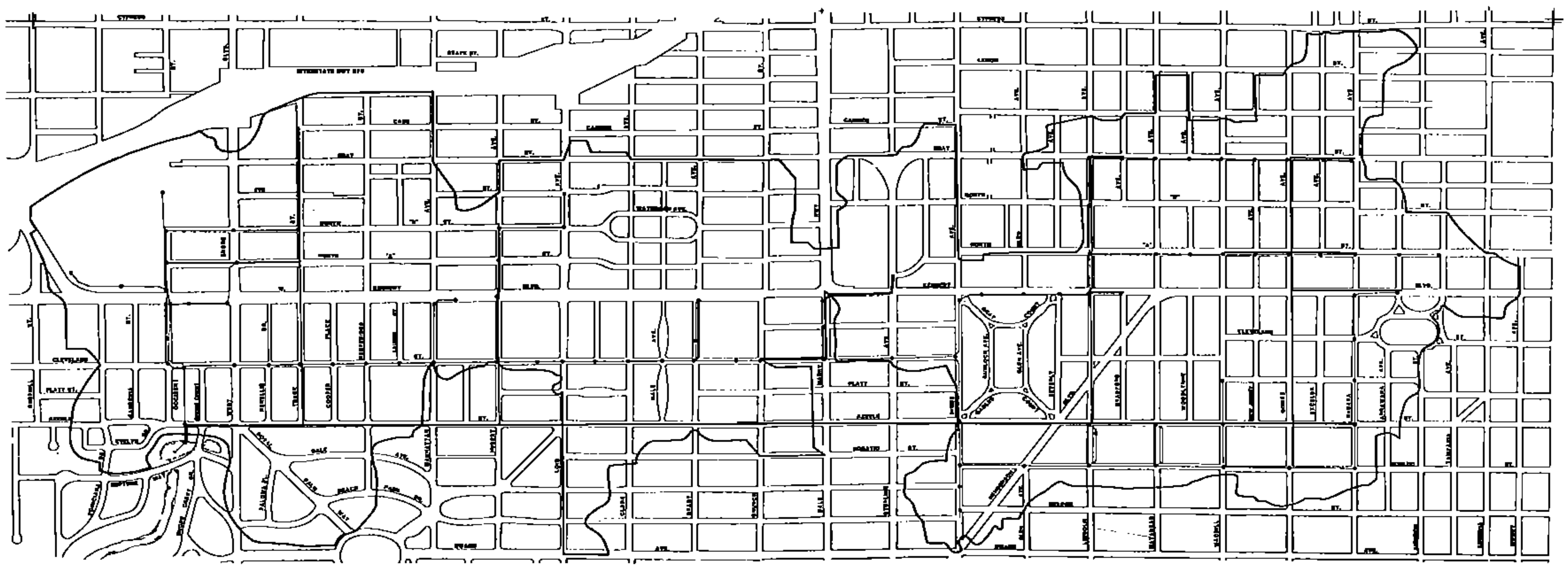
Alternative No. 6 is depicted in Figure 4-6. It is similar to Alternative No. 5 except that no improvements are provided in the area north of Kennedy Boulevard, east of Himes Avenue. These improvements would eliminate flooding in Problem Areas 1 and 2.

Alternative No. 7, shown in Figure 4-7, included a new Westshore outfall, improvement of flow conditions in the existing box culvert and detention faclities in the area south of Kennedy Boulevard, east of Himes Avenue and in the vicinity of Cleveland Street and Grady Avenue. This alternative would eliminate flooding in Problem Areas 1, 2 and 3.

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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CLEVELAND STREET BASIN

ALT 1
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

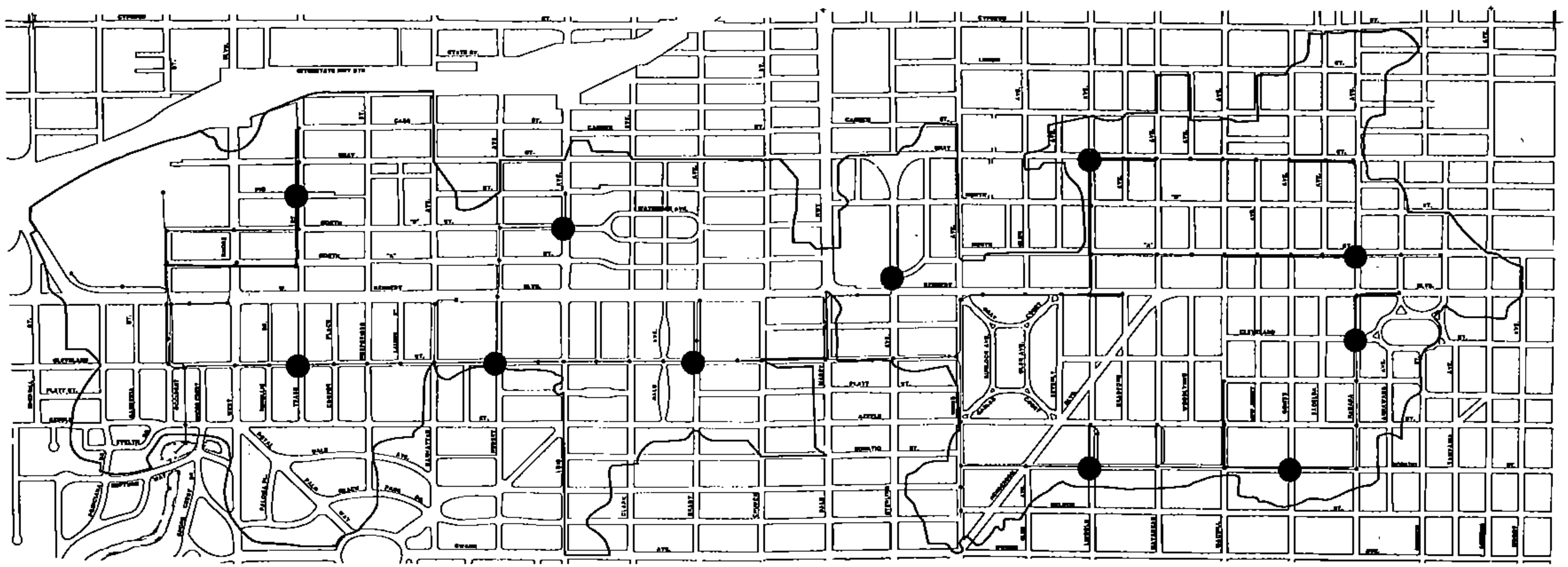
NORTH

0 200 400 600
SCALE IN FEET

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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CLEVELAND STREET BASIN

ALT 2
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

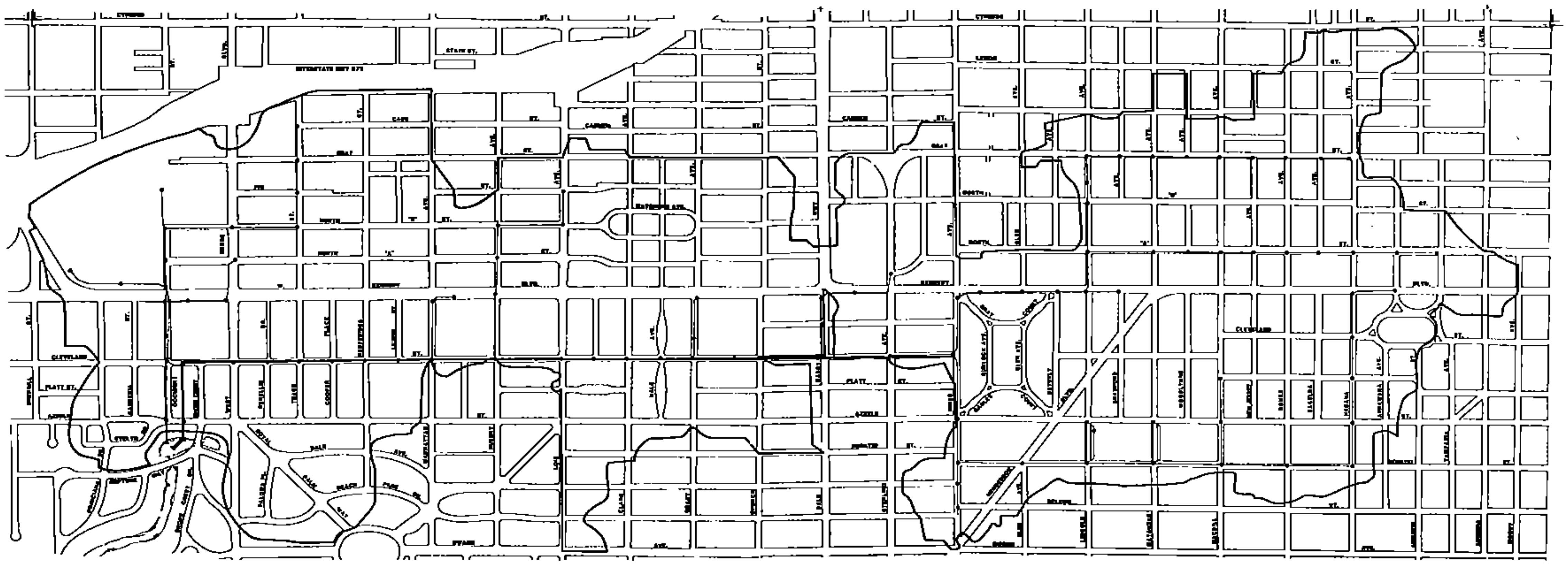
NORTH

0 200 400 800
SCALE IN FEET

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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CLEVELAND STREET BASIN

ALT 4
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

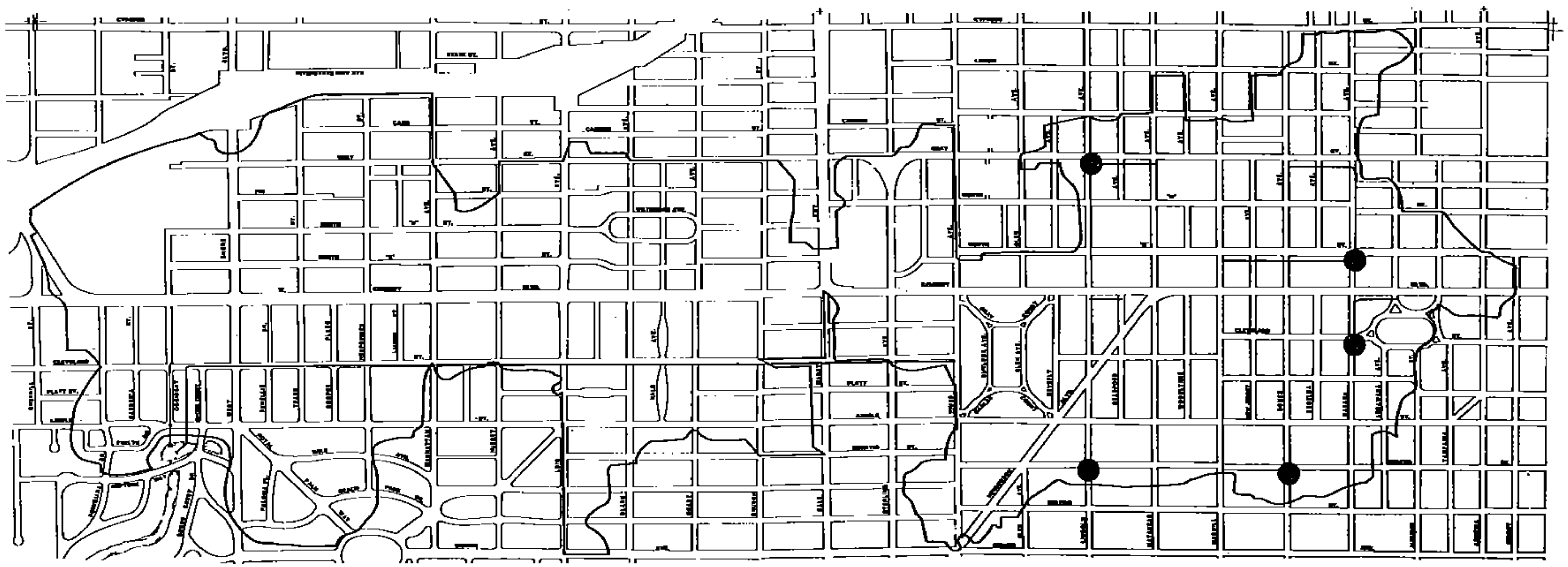
NORTH

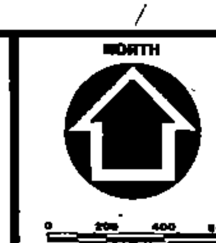
SCALE IN FEET

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.





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CLEVELAND STREET BASIN

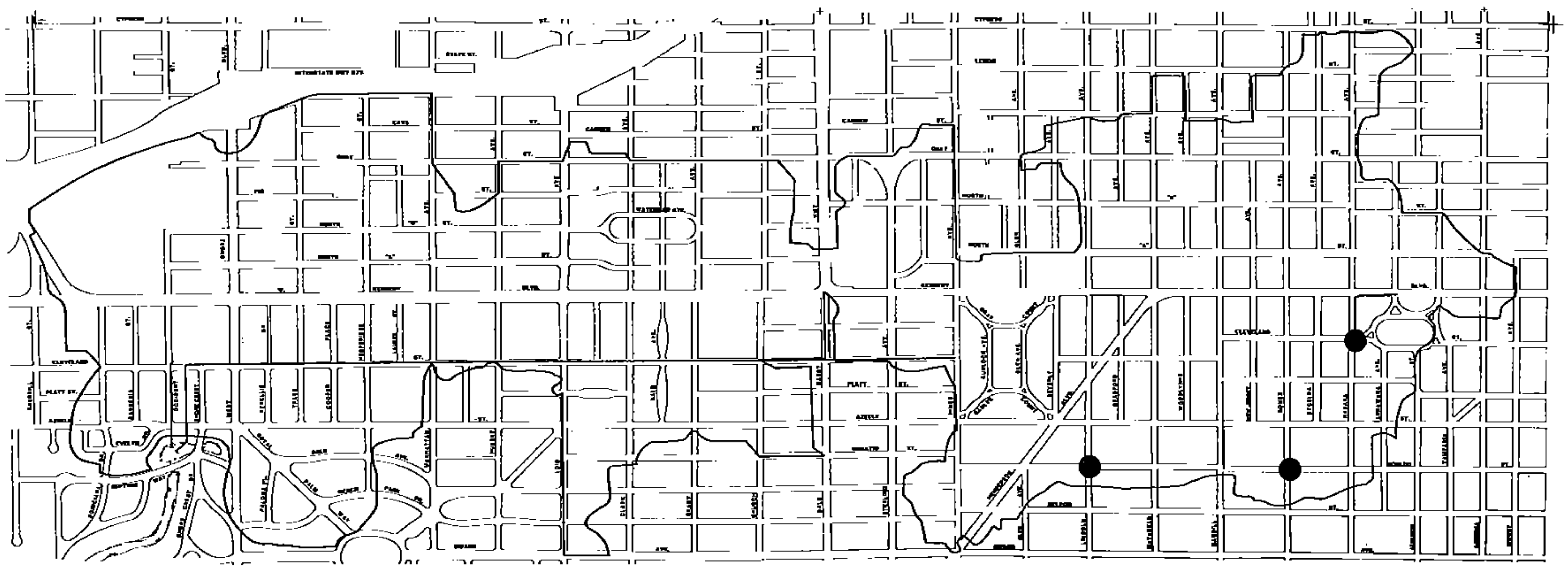
ALT 5
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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TAMPA, FLORIDA

DATE: 8/83 JOB No. 578-201.00

CLEVELAND STREET BASIN

ALT 6
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

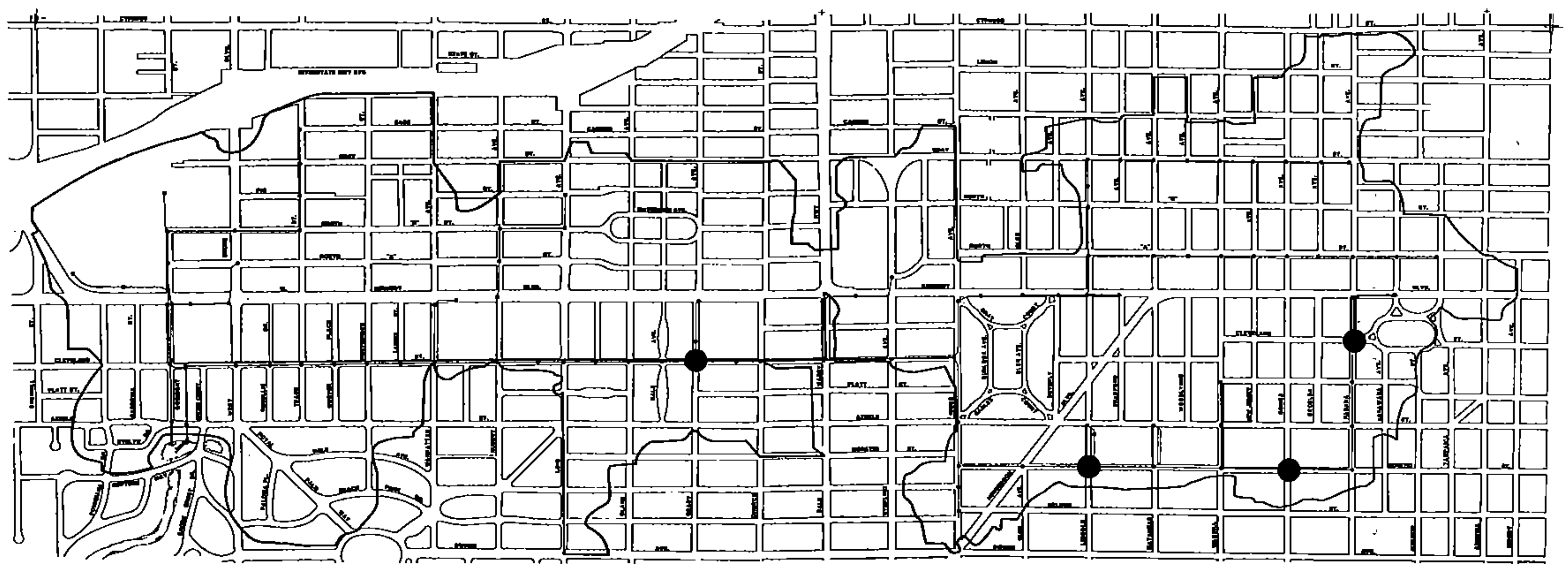
NORTH

SCALE IN FEET

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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DATE: 6/88 JOB No. 678-201.00

CLEVELAND STREET BASIN

ALT 7
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

SCALE IN FEET

Alternative No. 8, shown in Figure 4-8, also would eliminate flooding in Problem Areas 1, 2 and 3. This alternative consists of a new Westshore outfall, improvement of flow conditions in the Cleveland Street box, detention facilities south of Kennedy Boulevard, east of Himes Avenue and a new box culvert, paralleling the Cleveland Street box along Azeele Street from Clark Avenue west to the existing outfall.

Alternative No. 9 is presented in Figure 4-9. The improvements considered in this alternative include the new Westshore outfall, improved conditions in the Cleveland Street box and detention facilities north and south of Kennedy Boulevard, east of Himes Avenue, and in the vicinity of the intersection of Cleveland Street and Grady Avenue. Implementation of this alternative would eliminate flooding in all Problem Areas except Problem Area 5.

Alternative No. 10, shown in Figure 4-10, includes all the improvements in Alternative No. 9 except the new Westshore outfall. The results of the computer model for this alternative indicate that, without the Westshore outfall, flooding will occur in Problem Area 1 as well as minor flooding in Problem Area 3.

Alternative No. 11 is identical to Alternative No. 10 with the exception that it does not include improving flow conditions in the Cleveland Street box culvert. With this alternative, significant flooding would occur in Problem Areas 1 and 3. This alternative demonstrates that the flooding in Problem Area 1 can only be solved either by providing storage in the immediate vicinity or by conveyance improvements to the Cleveland Street box and the separate Westshore outfall.

Alternative No. 12, shown in Figure 4-11, includes the new Westshore outfall, improved flow conditions in the existing Cleveland Street box culvert and detention facilities north and south of Kennedy Boulevard, east of Himes Avenue, in the vicinity of the intersection of Cleveland Street and Grady Avenue and in the Westshore area. This alternative would eliminate flooding in all five problem areas.

4.4 MODELING OF CONTROL ALTERNATIVES

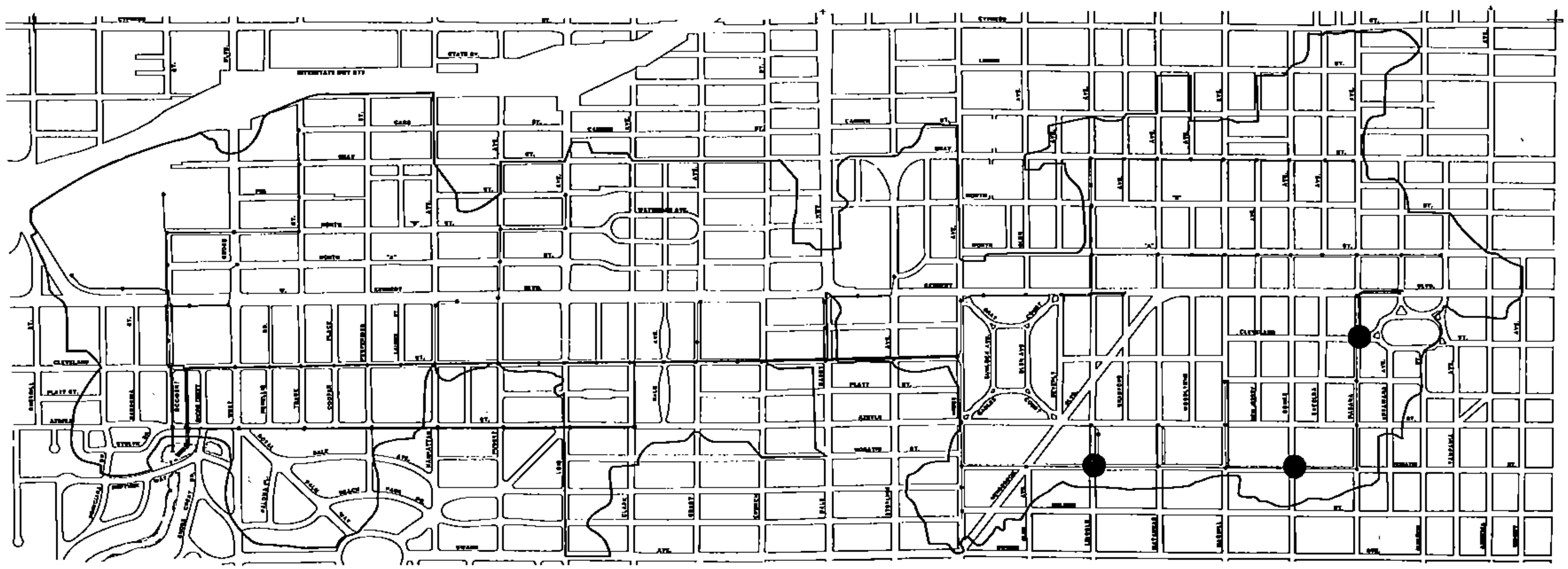
All twelve alternatives cited in Section 4.3 were analyzed with the computer model. Input parameters used during these model runs were those established during calibration. Each alternative was modeled using the design storm (five year recurrence frequency) event.

The criteria adopted for evaluating the effectiveness of each alternative in eliminating flooding was the volume of overflow from nodes within the system during surcharge and the duration of surcharge. The maximum acceptable volume of overflow at any point in the system was one acre-foot of stormwater. In nearly all areas the volume of overflow was maintained at less than 5,000 cubic feet and overflow durations were generally less than ten minutes.

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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CLEVELAND STREET BASIN

ALT 8
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

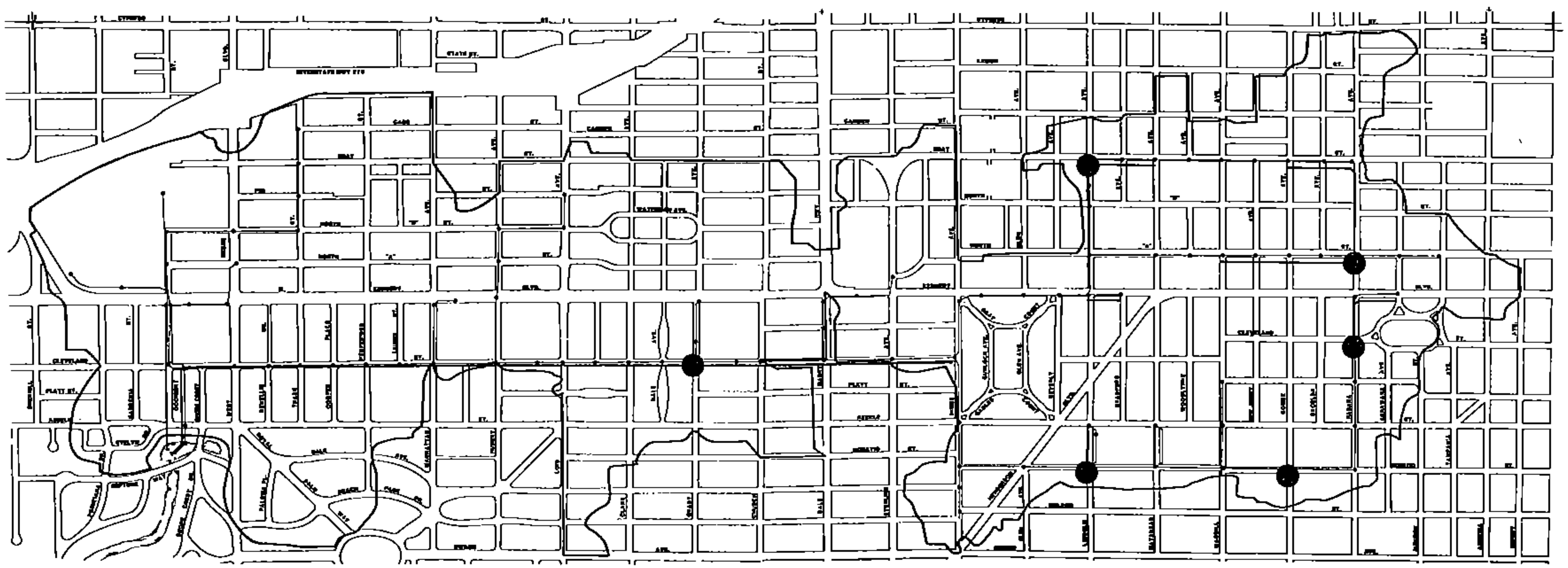
NORTH

0 200 400 800
SCALE IN FEET

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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CLEVELAND STREET BASIN

ALT 9
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

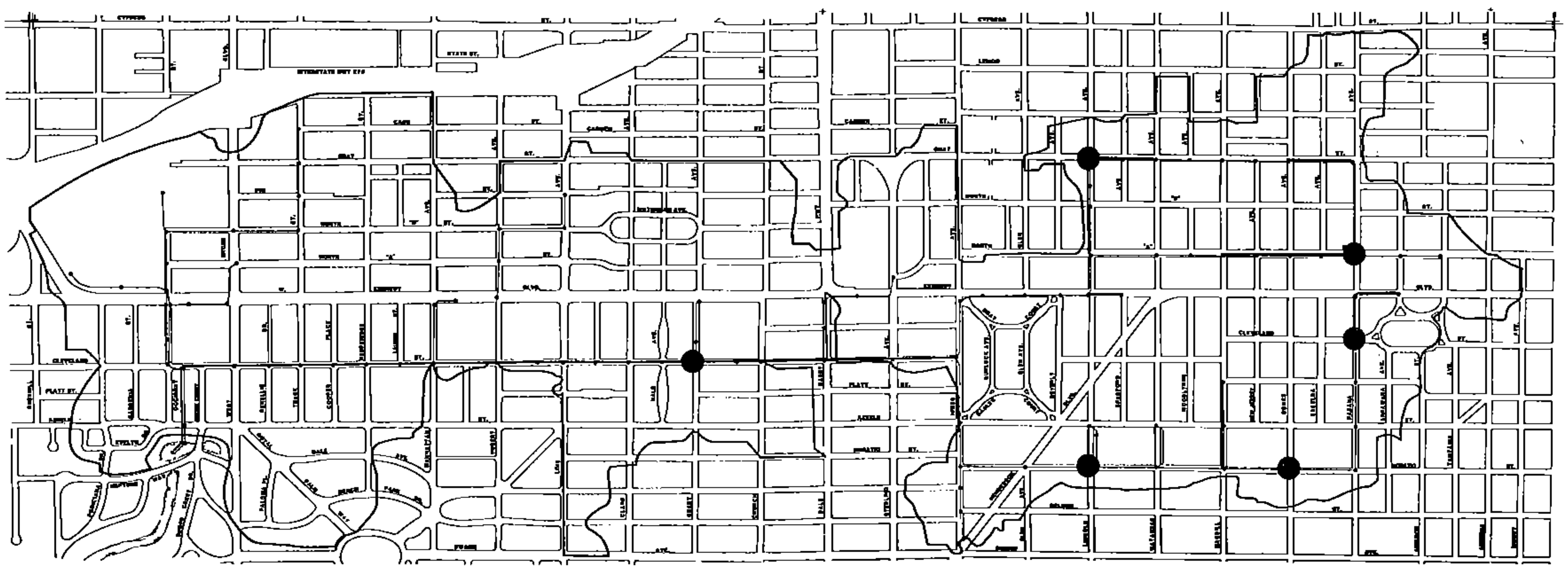
NORTH

0 200 400 800
SCALE IN FEET

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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CLEVELAND STREET BASIN

ALT 10
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

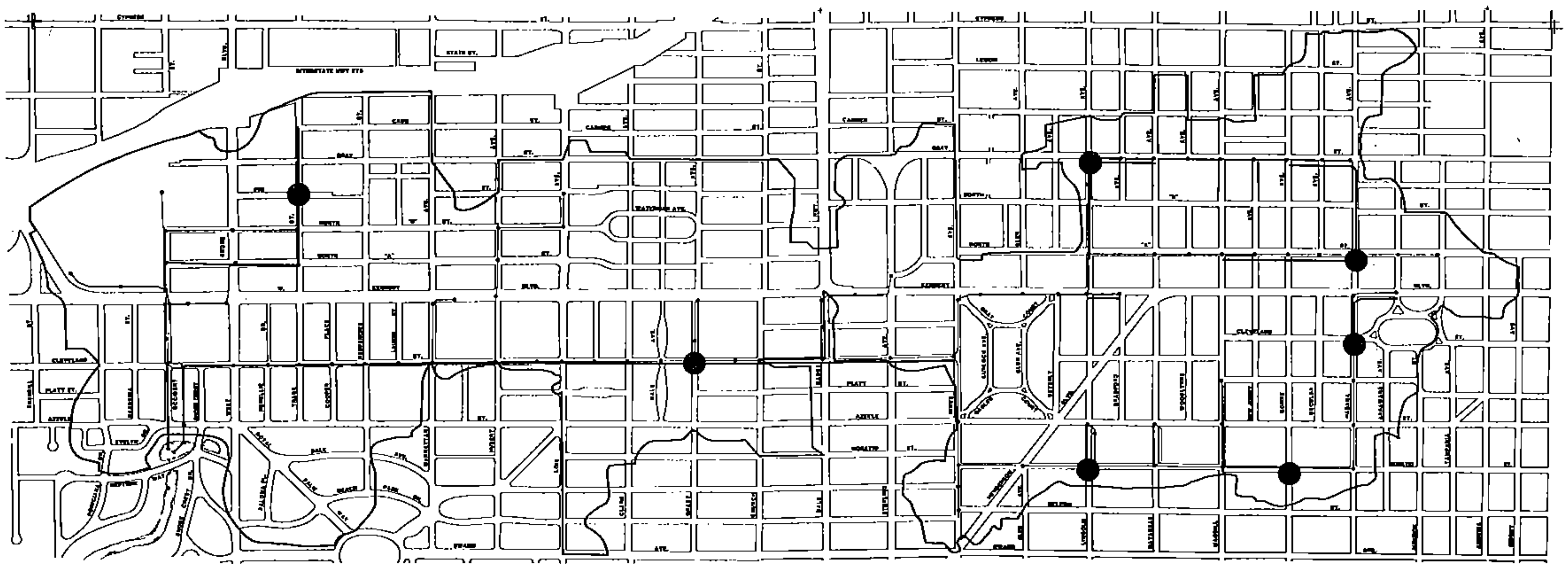
NORTH

0 200 400 800
SCALE IN FEET

LEGEND

- EXISTING SYSTEM
- PROPOSED CONVEYANCE IMPROVEMENTS
- PROPOSED DETENTION FACILITY

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE PROPOSED ALTERNATIVE IMPROVEMENTS IN CONCEPT FORM.



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CLEVELAND STREET BASIN

ALT 12
FACILITY SCHEMATIC

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

0 200 400 800
SCALE IN FEET

4.5 WATER QUALITY ASSESSMENT

4.5.1 Peak Flows

Table 4-1 lists and summarizes the various alternatives, starting with Alternative 0, which is the "no action" plan. The table shows the the computed peak outflow rates and the respective percentages of the "base" or "no action" figures. Also, the table indicates which priority areas (1 through 5) are predicted to undergo significant flood control improvement under each alternative.

Since the rainfall and land use characteristics are the same in all cases, the total volume of water leaving the system for the various alternatives should be very similar. This volume is approximately 5.9 million cubic feet during the design storm event.

4.5.2 Detention Times and Pollutant Load Reductions

The pollutant load resulting from each alternative is affected (to some degree) by the extent of detention storage provided. The alternatives differ in the size, location and number of detention basins. In a typical subarea, the peak flow is in the range of 100 to 200 cubic feet per second (cfs). Most detention basins have volumes ranging from half a million to a million cubic feet. Thus, the detention times are estimated to be between 1 and 4 hours.

The stormwater study, entitled "Study and Assessment of the Capabilities and Cost of Technology of Control of Pollutant Discharges from Urban Runoff", conducted by the National Commission on Water Quality in 1975 reports pollutant load reductions for various detention times. For the range of 1 - 4 hours, a 30 percent reduction in BOD suspended solids, nitrogen and phosporus appears reasonable.

Since Alternative 2 provides detention ponds throughout the Cleveland Street Basin, a 30 percent pollutant load reduction is estimated relative to the "no action" condition. Alternatives 5, 9, 10 and 11 each provide 5 or 6 detention basins, thus providing detention for approximately 50 percent of the flow. For these, a reduction of 15 percent (i.e., half of the 30 percent above) is estimated. Alternatives 6, 7 and 8 each provide 3 or 4 detention basins, or detention of approximately one-third of the flow. An overall removal rate of 10 percent is estimated for these alternatives. Alternative 12 provides detention for approximately 60 percent of the flow and would therefore result in an estimated removal rate of 18 percent. Table 4-2 displays the estimated runoff pollutant loads for the various alternatives considered.

4.5.3 Shock Loading

The alternatives not only differ in their total pollutant loads depending on the use of detention, but also in the distribution of the pollutant load over the duration of the storm. Generally, the higher the peak flow and peak loading rate, the more adverse would be the impact on the receiving lagoon.

The degree of usage of in-system detention storage has a great impact on peak flow, as can be seen by comparing the peak flows of two of the most effective alternatives in terms of reduced flooding:

TABLE 4-1

COMPARISON OF COMPUTED PEAK FLOWS AND FLOODING IMPROVEMENTS FOR VARIOUS ALTERNATIVES DURING THE DESIGN STORM[a]

ALTERNATIVE		COMPUTED PEAK RATE		SIGNIFICANT IMPROVEMENTS IN PROBLEM AREAS				
No.	Description	Flow (cfs)	% of Base	1	2	3	4	5
0	Base condition (No action)	567	100					
1	Closed conduit system	1,615	285	X	X	X	X	X
2	Primary storage improvements with some pipe improvements	526	93	X	X	X	X	X
3	Improved flow conditions for Cleveland Street box culvert	652	115					
4	Improved flow conditions for Cleveland Street box culvert and separate outfall for Westshore area	906 (combined)	160	X				
5	Same as Alternative 4, plus improvements from Alternative 2 located east of Himes	907 (combined)	160	X	X		X	
6	Same as Alternative 5, plus improvements south of Kennedy Boulevard	906	160	X	X			
7	Same as Alternative 6 plus storage at Grady and Cleveland from Alternative 2	906 (combined)	160	X	X	X		
8	Same as Alternative 6 plus parallel outfall along Azeele Street (from Clark Street to existing outfall)	1,067 (combined)	188	X	X	X		
9	Same as Alternative 5 plus storage at Grady and Cleveland from Alternative 2	849	150	X	X	X	X	

TABLE 4-1 (Continued)

ALTERNATIVE		COMPUTED PEAK RATE		SIGNIFICANT IMPROVEMENTS IN PROBLEM AREAS				
No.	Description	Flow (cfs)	% of Base	1	2	3	4	5
10	Same as Alternative 9, but without Westshore outfall	652	115	(X)[b]	(X)	(X)	(X)	
11	Same as Alternative 10, but without improvements to Cleveland Street box flow conditions	570	101	(X)	(X)	(X)	(X)	
12	Same as Alternative 9 plus detention storage in Westshore area.	830	146	X	X	X	X	X

[a] Design storm is a once in 5-year frequency, 1.5 hour duration event, with total rainfall of 3.3 inches.

[b] Parentheses indicate some improvements but not as significant as those from previous alternatives.

ESTIMATED RUNOFF POLLUTANT LOADS FROM
THE CLEVELAND STREET BASIN FOR THE VARIOUS
ALTERNATIVES CONSIDERED

ALTERNATIVE NUMBER	YEARLY RUNOFF POLLUTANT LOADS, LBS.			
	SUSPENDED SOLIDS	BOD_5	TOTAL NITROGEN	TOTAL PHOSPHORUS
1, 3, 4	287,705	23,632	4,535	434
6, 7, 8	258,935	21,269	4,081	391
5, 9, 10, 11	244,549	20,087	3,854	369
12	235,918	19,378	3,719	356
2	201,393	16,542	3,174	304

REP20:K

- Alternative 1: closed conduit systems (peak flow 285 percent of "base" peak)

- Alternative 2: primary storage improvements with some pipe improvements (peak flow 93 percent of "base" peak)

If other factors (e.g., cost) were equal, Alternative 2 would be preferable to 1 from a water quality "shock loading" standpoint.

Alternatives 5 through 9 may be grouped in a "moderately effective" group, since they all alleviate flooding in some problem areas, but not in all. In this group, the range of percentage of peak flow to the base is fairly narrow, 150 to 188. Thus, the impact of these alternatives lies between that of alternatives 1 and 2, discussed above.

Alternative 12 effectively alleviates flooding in the problem areas. The percentage of peak flow to the base in 146. Thus, the shock load impact of this alternative is slightly better than the "moderately effective" group.

Alternatives 3, 10 and 11 have a very moderate impact in terms of shock loading, just slightly greater than that for Alternative 2. However, these three alternatives offer less improvement in flooding.

For the alternatives which include detention ponds, a reduction in total pollutant loads can be expected and should partially offset any increase in the peak pollutant loading rate to Neptune Lagoon. Also, because ambient water quality in the existing lagoon appears to be poor, any short term reductions in salinity due to increased peak flow rates should not degrade existing water quality.

4.5.4 Summary of Water Quality Evaluation

From a water quality standpoint, Alternative 2 would be preferred. In addition to alleviating flooding in all priority areas, it would reduce the peak outflow rate from that in the "no action" plan. It would also reduce pollutant loading by approximately 30 percent from the "no action" plan. Alternative 2 would be favored if the necessary land for all the detention basins could be obtained at a reasonable cost.

Alternatives 5 through 12 involve detention basins. Of these, 10 and 11 result in very moderate peak flow increases over the "no action" alternative, but they do not result in as much improvement in flood reduction as others in the group. Of the other solutions considered, alternative 12 has the smallest increase in peak flow and flood control benefits to the largest number of priority areas. Its reduction in pollutant load relative to the "no action" plan is estimated at 18 percent. Alternative 12 would be a good second choice after Alternative 2 in terms of water quality impact.

4.6 EVALUATION OF ALTERNATIVES

The evaluation of the twelve alternatives considered and selection of the preferred alternative were based on three major factors; cost, water quality impact and performance.

Order of magnitude cost estimates were prepared for each alternative. These estimates included anticipated right-of-way costs based on current land value guidelines provided by the City of Tampa Right-of-Way Division, projected costs for the installation of new facilities and, where appropriate, the cost of restoring roadway facilities which would be affected by the construction of proposed drainage improvements. These order of magnitude estimates are summarized in Table 4-3.

The comparative water quality impacts of the twelve alternatives considered were as discussed in Section 4.5.

The basis used for evaluating the performance of each alternative was its impact on flooding in each of the five problem areas identified. These impacts were discussed in Section 4.3 and are summarized, together with costs and water quality impacts in the matrix presented in Table 4-4.

All twelve alternatives analyzed were considered to be feasible from the standpoint of constructability. With the exception of Alternative No. 3 (improvement of flow conditions in the Cleveland Street box culvert), the implementation of each alternative necessarily involves some negative impacts. These range from disruption of traffic during the installation of new conveyance facilities to the possibility of condemnation and relocation in order to obtain and construct stormwater detention sites. Since the evaluation of these factors is not simply a matter of engineering judgement, it is recommended that the weight given to these factors be determined by officials of the City of Tampa with whatever technical input they may require from PBS&J.

TABLE 4-3

SUMMARY OF ALTERNATE IMPROVEMENT ESTIMATES

ALT. NO.	PIPING COST	LAND ACQUISITION COST	RETENTION POND CONSTRUCTION	BOX IMPROVEMENT	TOTAL COST
1	$21,538,000	--	--	--	$21,538,000
2	$ 2,948,000	$7,555,000	$628,000	--	$11,131,000
3	--	$ 121,000	--	$667,000	$ 788,000
4	$ 595,000	$ 231,000	--	$667,000	$ 1,493,000
5	$ 1,959,000	$4,121,000	$303,000	$667,000	$ 7,050,000
6	$ 1,033,000	$2,208,000	$163,000	$667,000	$ 4,071,000
7	$ 1,033,000	$3,391,000	$290,000	$667,000	$ 5,381,000
8	$ 4,543,000	$2,208,000	$164,000	$667,000	$ 7,582,000
9	$ 1,959,000	$5,304,000	$430,000	$667,000	$ 8,360,000
10	$ 1,364,000	$5,194,000	$430,000	$667,000	$ 7,655,000
11	$ 1,364,000	$5,073,000	$430,000	--	$ 6,867,000
12	$ 3,195,000	$5,930,000	$478,000	$667,000	$10,270,000

NOTE: All estimates are based on May, 1983 prices.

TABLE 4-4

SUMMARY OF EVALUATION CRITERIA

Alternative Number	Estimated Costs	Water Quality Impacts (at outfall)	Performance (Problem Areas Eliminated)
1	$21,538,000	Increases short term loading	All
2	$11,131,000	30% reduction in pollutant loading	All
3	$ 788,000	No impact	None. Minor reduction in 1 and 3
4	$ 1,493,000	Increases short term loading	1
5	$ 7,050,000	15% reduction in pollutant loading	1, 2 & 4
6	$ 4,071,000	10% reduction in pollutant loading	1 & 2
7	$ 5,381,000	10% reduction in pollutant loading	1, 2 & 3
8	$ 7,582,000	10% reduction in pollutant loading	1, 2 & 3
9	$ 8,360,000	15% reduction in pollutant loading	1, 2, 3 & 4
10	$ 7,655,000	15% reduction in pollutant loading	2 & 4 minor flooding in 3
11	$ 6,867,000	15% reduction in pollutant loading	2 & 4
12	$10,270,000	18% reduction in pollutant loading	All

SECTION 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 SELECTION OF THE PREFERRED ALTERNATIVE

As can be seen from a review of Table 4-4, only three of the schemes considered, Alternatives 1, 2 and 12, provide relief from flooding in all five problem areas. Of these, Alternative No. 1 was by far the most expensive with an estimated cost of $21,500,000, almost twice as costly as the next highest solution. In addition, Alternative No. 1 results in the most adverse water quality impacts of any of the candidate solutions. Alternative No. 2 is equally effective in eliminating existing flooding problems, and has the most favorable impact on water quality. It does not, however, utilize two improvement types (improvement of flow conditions in the Cleveland Street box and a separate outfall for Westshore) which provide significant benefits at relatively low cost.

Alternative No. 12, through inclusion of these improvement types, provides relief from flooding in all five problem areas at the least cost. Because this solution includes the new Westshore outfall, the water quality impact resulting from its implementation would not be quite as positive as that for Alternative No. 2, but would compare favorably with the existing system. This alternative also lends itself well to a phased program for implementation. For these reasons, Alternative No. 12 is recommended as the preferred alternative for solution of the existing flooding problems in the Cleveland Street Basin. The physical improvements associated with the preferred alternative are presented in Figures 5-1W and 5-1E.

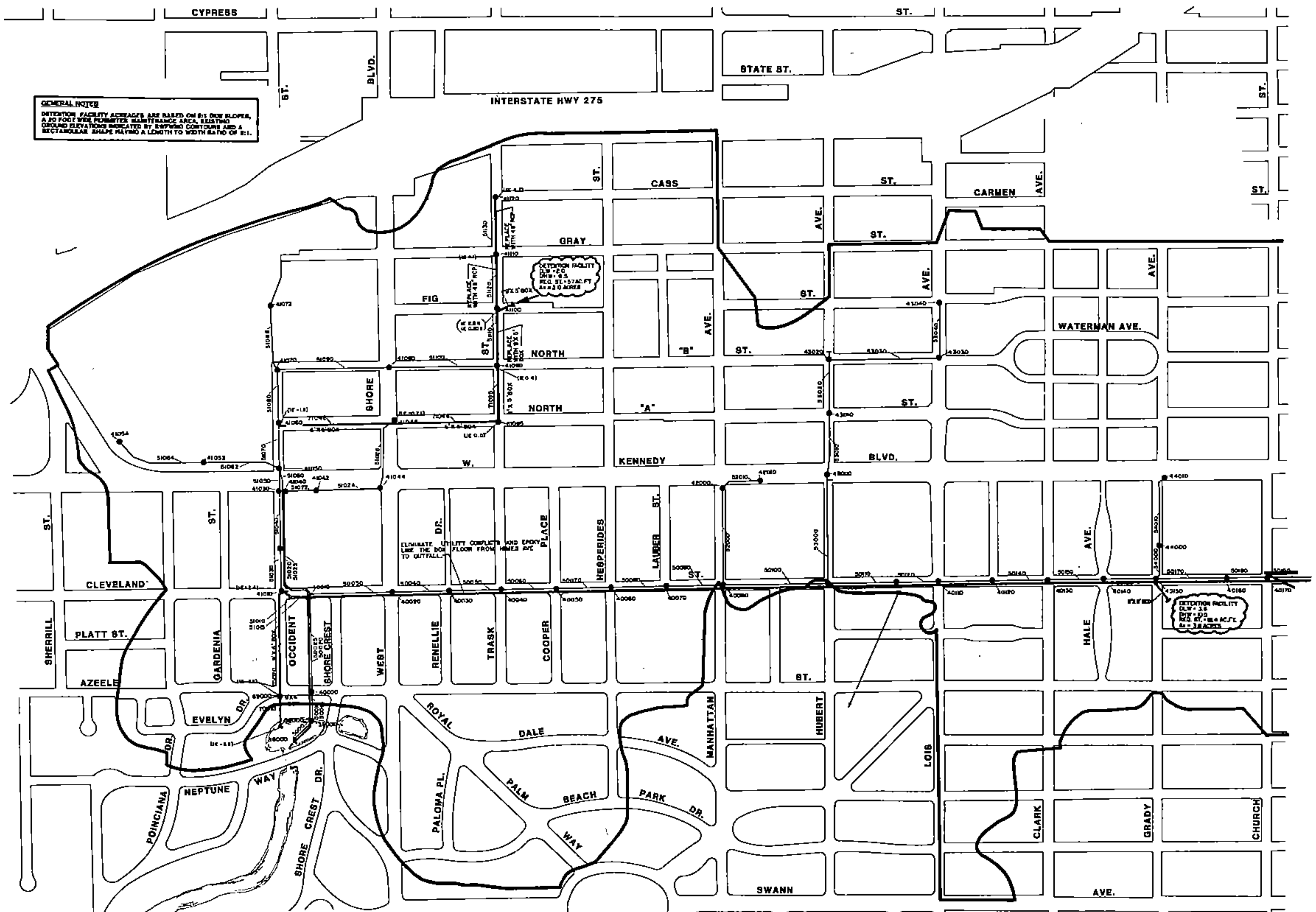
5.2 DER RESPONSE

A subsequent meeting with the DER was held to discuss permitting requirements, possible constraints and overall acceptability of the preferred solution. Pertinent points made during this meeting are indicated below.

- Regardless which alternative is finally selected the total volume of runoff draining to Old Tampa Bay will be the same for all alternatives since no change in land use is expected nor total retention proposed within the basin.

- Alternatives that include detention facilities would be preferred over alternatives that do not include detention facilities.

- The DER does not have jurisdiction over the conveyance systems within the basin. Their jurisdiction and primary concern is at the outfall, Neptune Lagoon.



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TAMPA, FLORIDA

DATE: 8/83 JOB No. 678-201.00

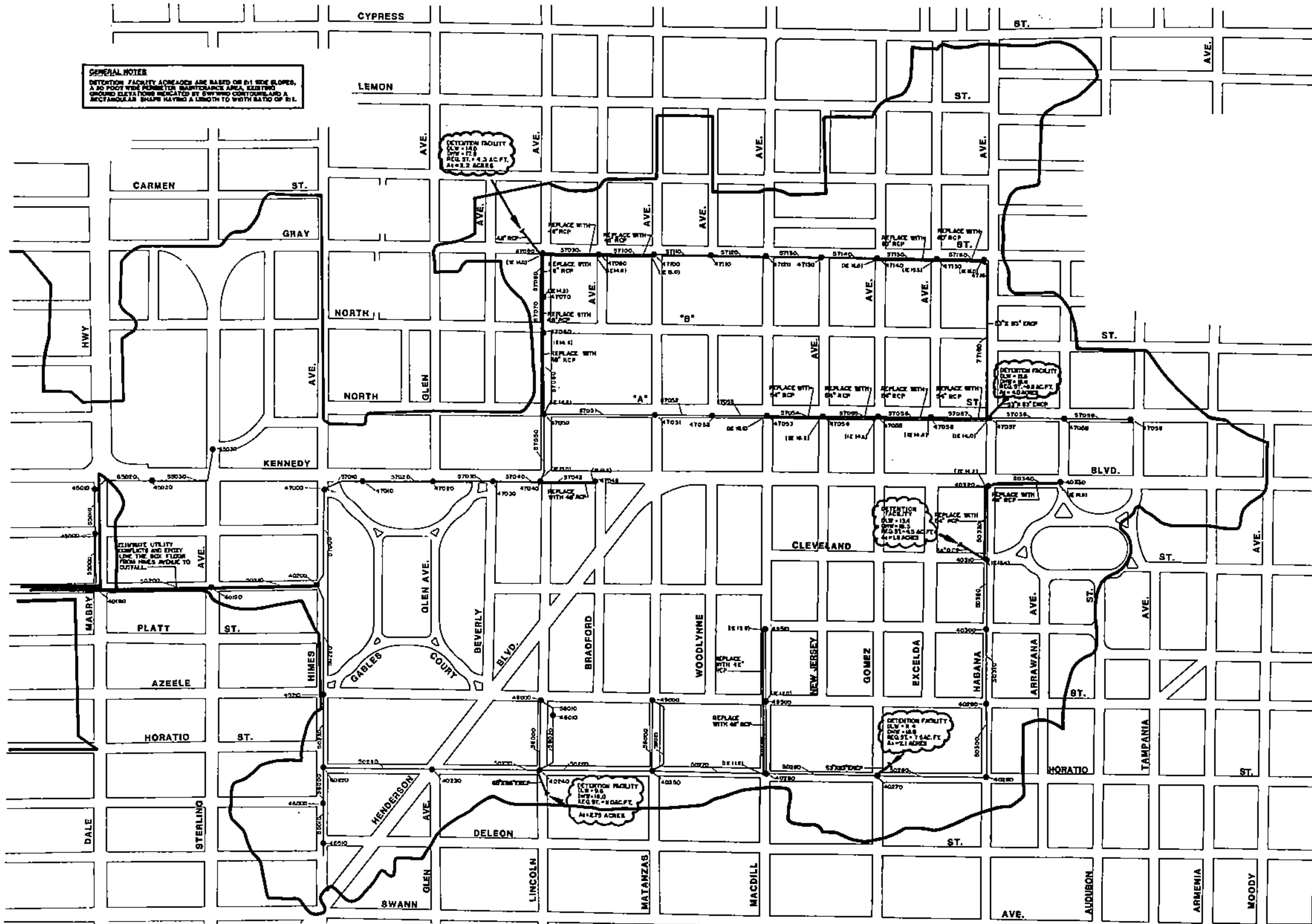
CLEVELAND STREET BASIN (WEST)

PREFERRED ALTERNATIVE IMPROVEMENTS

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

0 200 400 800
SCALE IN FEET



POST, BUCKLEY, SCHUH & JERNIGAN, INC.
CONSULTING ENGINEERS & PLANNERS
TAMPA, FLORIDA

DATE: 5/89 JOB No. 578-201.00

CLEVELAND STREET BASIN (EAST)

PREFERRED ALTERNATIVE IMPROVEMENTS

DEPARTMENT OF PUBLIC WORKS
STORMWATER MANAGEMENT DIVISION

NORTH

SCALE IN FEET

- It was indicated to DER that where economically practical the preferred alternative does utilize detention facilities to alleviate flooding and reduce total pollutant loads to Neptune Lagoon.

- The structural improvements at the existing outfall (Westshore relief line) would require a joint Dredge and Fill and Notice of Stormwater Discharge permit. The DER did not seem overly concerned with this improvement. It was indicated to them that although the "shock load" would be increased the total pollutant load would be decreased because the preferred alternative does include numerous detention facilities. A suggestion that the DER made was to create a shallow natural wetland at the new outfall. This could be accomplished by terminating the new outfall just south of Azeele Street and excavating a shallow area that could be planted with wetland vegetation to connect the new outfall to Neptune Lagoon.

- The preferred alternative improvements does not exempt development or redevelopment projects from meeting DER Notice of Stormwater Discharge requirements.

- The water quality evaluation was sufficient to "weigh" the potential effects of the various improvements.

In summary, the DER was pleased to find that the preferred alternative did include numerous detention facilities throughout the basin and preliminary indications were that the improvements proposed in the preferred alternative would be permitable activities.

5.3 FINANICAL REQUIREMENTS AND PHASING

A four step phasing plan for implementation of the preferred alternative, indicating the proposed improvements and associated costs for each phase, is presented in Table 5-1.

Land acquisition represents a substantial portion of the overall cost to implement the plan and will be a critical element in the schedule for completing the improvement program. It is therefore recommended that design of the system, in sufficient detail to clearly delineate the areas required for detention, and actual land acquisition be budgeted and initiated as the first step in the process. No inflation factor has been included in the cost estimates presented in this report and it is reasonable to assume that early acquisition will also result in lower land costs.

TABLE 5-1

PROPOSED PHASING PLAN

PHASE NO.	IMPROVEMENTS	RIGHT-OF-WAY	CONSTRUCTION	TOTALS	REMARKS
1	Improve flow conditions in the existing Cleveland Street box and construct a new outfall for the Westshore area.	$ 231,000	$1,269,000	$1,500,000	Relieves flooding in problem area 1.
2	Construct detention facilities and associated pipe improvements in the area east of Himes Ave., south of Kennedy Blvd.	$1,977,000	$ 523,000	$2,600,000	Relieves flooding in problem area 2. Configuration of the system will be the same as Alternative No. 6 (See Figure 4-6)
3	Install the proposed detention facilities and local pipe improvements in the area east of Himes Ave., north of Kennedy Blvd.	$3,100,000	$1,200,000	$4,300,000	Relieves flooding in problem area 4. System improvements completed through this phase will be the same as those shown for Alternative No. 9. (See Figure 4-9)
4	Install the proposed detention facilities and piping in the Westshore area between Kennedy Blvd. and I-275.	$ 626,000	$1,284,000	$1,910,000	Completes improvements

APPENDIX A

Minor Losses

OPEN CHANNEL HYDRAULICS

for n between 0.011 and 0.040. For practical purposes, the following approximate forms of Eq. (5-9) are generally suggested for use:

$$y = 1.5\sqrt{n} \qquad \text{for } R < 1.0 \text{ m} \tag{5-10}$$
$$y = 1.3\sqrt{n} \qquad \text{for } R > 1.0 \text{ m} \tag{5-11}$$

5-7. Determination of Manning's Roughness Coefficient. In applying the Manning formula or the G. K. formula, the greatest difficulty lies in the determination of the roughness coefficient n; for there is no exact method of selecting the n value. At the present stage of knowledge, to select a value of n actually means to estimate the resistance to flow in a given channel, which is really a matter of intangibles. To veteran engineers, this means the exercise of sound engineering judgment and experience; for beginners, it can be no more than a guess, and different individuals will obtain different results.

In order to give guidance in the proper determination of the roughness coefficient, four general approaches will be discussed; namely, (1) to understand the factors that affect the value of n and thus to acquire a basic knowledge of the problem and narrow the wide range of guesswork, (2) to consult a table of typical n values for channels of various types, (3) to examine and become acquainted with the appearance of some typical channels whose roughness coefficients are known, and (4) to determine the value of n by an analytical procedure based on the theoretical velocity distribution in the channel cross section and on the data of either velocity or roughness measurement. The first three approaches will be given in the next three articles, and the fourth approach will be taken up in Art. 8-7.

5-8. Factors Affecting Manning's Roughness Coefficient. It is not uncommon for engineers to think of a channel as having a single value of n for all occasions. In reality, the value of n is highly variable and depends on a number of factors. In selecting a proper value of n for various design conditions, a basic knowledge of these factors should be found very useful. The factors that exert the greatest influence upon the coefficient of roughness in both artificial and natural channels are therefore described below. It should be noted that these factors are to a certain extent interdependent; hence discussion about one factor may be repeated in connection with another.

A. Surface Roughness. The surface roughness is represented by the size and shape of the grains of the material forming the wetted perimeter and producing a retarding effect on the flow. This is often considered the only factor in selecting a roughness coefficient, but it is actually just one of several major factors. Generally speaking, fine grains result in a relatively low value of n and coarse grains, in a high value of n.

In alluvial streams where the material is fine in grain, such as sand, clay, loam, or silt, the retarding effect is much less than where the material is coarse, such as gravels or boulders. When the material is fine, the value of n is low and relatively unaffected by change in flow stage. When the material consists of gravels and boulders, the value of n is generally high, particularly at low or high stage. Larger boulders usually collect at the bottom of the stream, making the channel bottom rougher than the banks and increasing the value of n at low stages. At high stages, a portion of the energy of flow is used in rolling the boulders downstream, thus increasing the value of n. A theoretical discussion of surface roughness will be given in Art. 8-2.

B. Vegetation. Vegetation may be regarded as a kind of surface roughness, but it also markedly reduces the capacity of the channel and retards the flow. This effect depends mainly on height, density, distribution, and type of vegetation, and it is very important in designing small drainage channels.

At the University of Illinois an investigation has been made to determine the effect of vegetation on the coefficient of roughness [22]. On one of the drainage ditches in central Illinois under investigation, an average n value of 0.033 was measured in March, 1925, when the channel was in good condition. In April, 1926, there were bushy willows and dry weeds on the side slopes, and n was found to be 0.055. This increase in n represents the result of one year's growth of vegetation. During the summers of 1925 and 1926 there was a thick growth of cattails on the bottom of the channel. The n value at medium summer stages was about 0.115, and at a nearly bankfull stage it was 0.099. The cattails in the channel were washed out by the high water in September, 1926; the average value of n found after this occurrence was 0.072. The conclusions drawn from this investigation were, in part, as follows:

1. The minimum value of n that should be used for designing drainage ditches in central Illinois is 0.040. This value is obtainable at high stages during the summer months in the most carefully maintained channels, where the bottom of the channel is clear of vegetation and the side slopes are covered with grass or low weeds, but no bushes. This low value of n should not be used unless the channel is to be cleared annually of all weeds and bushes.

2. A value of $n = 0.050$ should be used if the channel is to be cleared in alternate years only. Large weeds and bushy willows from 3 to 4 ft high on the side slopes will produce this value of n.

3. In channels that are not cleared for a number of years, the growth may become so abundant that values of $n > 0.100$ may be found.

4. Trees from 6 to 8 in. in diameter growing on the side slopes do not impede the flow so much as do small bushy growths, provided overhanging branches are cut off.

The U.S. Soil Conservation Service has made studies on flow of water in small shallow channels protected by vegetative linings (Chap. 7, Sec. C). It was found that n values for these channels varied with the shape and cross section of the channel, the slope of the channel bed, and the depth of flow. Comparing two channels, all other factors being equal, the lesser average depth gives the higher n value, owing to a larger proportion of affected vegetation. Thus, a triangular channel has a higher n value than a trapezoidal channel, and a wide channel has a lower n value than a narrow channel. A flow of sufficient depth tends to bend over and submerge the vegetation and to produce low n values. A steep slope causes greater velocity, greater flattening of the vegetation, and low n values.

The effect of vegetation on flood plains will be discussed later in item *H*.

C. Channel Irregularity. Channel irregularity comprises irregularities in wetted perimeter and variations in cross section, size, and shape along the channel length. In natural channels, such irregularities are usually introduced by the presence of sand bars, sand waves, ridges and depressions, and holes and humps on the channel bed. These irregularities definitely introduce roughness in addition to that caused by surface roughness and other factors. Generally speaking, a gradual and uniform change in cross section, size, and shape will not appreciably affect the value of n, but abrupt changes or alternation of small and large sections necessitates the use of a large value of n. In this case, the increase in n may be 0.005 or more. Changes that cause sinuous flow from side to side of the channel will produce the same effect.

D. Channel Alignment. Smooth curvature with large radius will give a relatively low value of n, whereas sharp curvature with severe meandering will increase n. On the basis of flume tests, Scobey [23] suggested that the value of n be increased 0.001 for each 20 degrees of curvature in 100 ft of channel. Although it is doubtful whether curvature ever increases n more than 0.002 or 0.003, its effect should not be ignored, for curvature may induce the accumulation of drift and thus indirectly increase the value of n. Generally speaking, the increase of roughness in unlined channels carrying water at low velocities is negligible. An increase of 0.002 in n value would constitute an adequate allowance for curve losses in most flumes containing pronounced curvatures, whether built of concrete or other materials. The meandering of natural streams, however, may increase the n value as high as 30%.

E. Silting and Scouring. Generally speaking, silting may change a very irregular channel into a comparatively uniform one and decrease n, whereas scouring may do the reverse and increase n. However, the dominant effect of silting will depend on the nature of the material deposited. Uneven deposits such as sand bars and sand waves are channel irregularities and will increase the roughness. The amount and uniformity of scouring will depend on the material forming the wetted perimeter. Thus, a sandy or gravelly bed will be eroded more uniformly than a clay bed. The deposition of silt eroded from the uplands will tend to even out the irregularities in a channel dredged through clay. The energy used in eroding and carrying the material in suspension or rolling it along the bed will also increase the n value. The effect of scouring is not significant as long as the erosion on channel bed caused by high velocities is progressing evenly and uniformly.

F. Obstruction. The presence of log jams, bridge piers, and the like tends to increase n. The amount of increase depends on the nature of the obstructions, their size, shape, number, and distribution.

G. Size and Shape of Channel. There is no definite evidence about the size and shape of a channel as an important factor affecting the value of n. An increase in hydraulic radius may either increase or decrease n, depending on the condition of the channel (Fig. 5-4).

H. Stage and Discharge. The n value in most streams decreases with increase in stage and in discharge. When the water is shallow, the irregularities of the channel bottom are exposed and their effects become pronounced. However, the n value may be large at high stages if the banks are rough and grassy.

When the discharge is too high, the stream may overflow its banks and a portion of the flow will be along the flood plain. The n value of the flood plains is generally larger than that of the channel proper, and its magnitude depends on the surface condition or vegetation. If the bed and banks of a channel are equally smooth and regular and the bottom slope is uniform, the value of n may remain almost the same at all stages; so a constant n is usually assumed in the flow computation. This happens mostly in artificial channels. On flood plains the value of n usually varies with the stage of submergence of the vegetation at low stages. This can be seen, for example, from Table 5-4, which shows the n values for various flood stages according to the type of cover and depth

TABLE 5-4. VALUES OF n FOR VARIOUS STAGES IN THE NISHNABOTNA RIVER, IOWA, FOR THE AVERAGE GROWING SEASON

Depth of water, ft	Channel section	Flood-plain cover				
		Corn	Pasture	Meadow	Small grains	Brush and waste
Under 1	0.03	0.06	0.05	0.10	0.10	0.12
1 to 2	0.03	0.06	0.05	0.08	0.09	0.11
2 to 3	0.03	0.07	0.04	0.07	0.08	0.10
3 to 4	0.03	0.07	0.04	0.06	0.07	0.09
Over 4	0.03	0.06	0.04	0.05	0.06	0.08

of inundation, as observed in the Nishnabotna River, Iowa, for the average growing season [24]. It should be noted, however, that vegetation has a marked effect only up to a certain stage and that the roughness coefficient can be considered to remain constant for practical purposes in determining overbank flood discharges.

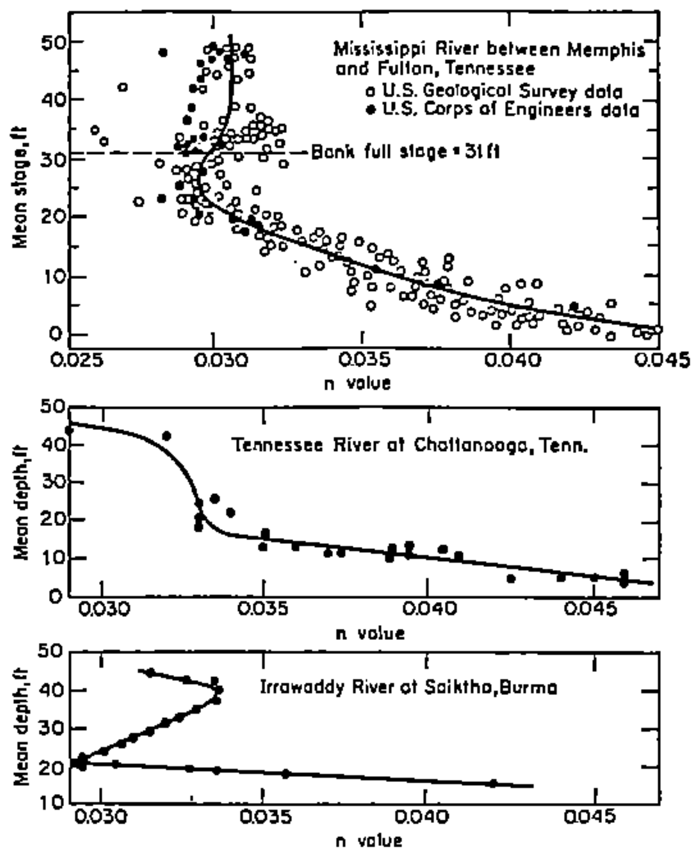


FIG. 5-4. Variations of the n value with the mean stage or depth.

Curves of n value versus stage (Fig. 5-4) in streams have been given by Lane [25], showing how value of n varies with stage in three large river channels. For the roughness of large canals, a study in connection with the design of the Panama Canal was made by Meyers and Schultz [26].[1] The two most important conclusions reached from this study were (1) that the n value for a river channel is least when the stage is at or somewhat above normal bankfull stage, and tends to increase for both

[1] A table of n values for eleven large channels at the most efficient depths and the curves showing the variations of n value with hydraulic radius in eight river channels are also given in this reference.

higher and lower stages; and (2) that the bankfull n values do not vary greatly for rivers and canals in different kinds of material and in widely separated locations.

For circular conduits, Camp [27,28] was able to show that the n value for a conduit flowing partially full is greater than that for a full conduit. Using measurements on clean sewer pipe and drain tile, both clay and concrete, from 4 to 12 in. in size, he found an increase of about 24% in the n value at the half-depth (Fig. 6-5).[1] The n value for the pipe flowing full was found to vary from 0.0095 to 0.011. Taking an average value of 0.0103, the n value at half-depth should be about 0.013. This is identical with the usual design value, which is based largely on measured values in sewers flowing partially full.

I. Seasonal Change. Owing to the seasonal growth of aquatic plants, grass, weeds, willow, and trees in the channel or on the banks, the value of n may increase in the growing season and diminish in the dormant season. This seasonal change may cause changes in other factors.

J. Suspended Material and Bed Load. The suspended material and the bed load, whether moving or not moving, would consume energy and cause head loss or increase the apparent channel roughness.

All the above factors should be studied and evaluated with respect to conditions regarding type of channel, state of flow, degree of maintenance, and other related considerations. They provide a basis for determining the proper value of n for a given problem. As a general guide to judgment, it may be accepted that conditions tending to induce turbulence and cause retardance will increase n value and that those tending to reduce turbulence and retardance will decrease n value.

Recognizing several primary factors affecting the roughness coefficient, Cowan [32] developed a procedure for estimating the value of n. By this procedure, the value of n may be computed by

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \qquad (5\text{-}12)$$

where n_0 is a basic n value for a straight, uniform, smooth channel in the natural materials involved, n_1 is a value added to n_0 to correct for the effect of surface irregularities, n_2 is a value for variations in shape and size of the channel cross section, n_3 is a value for obstructions, n_4 is a value for vegetation and flow conditions, and m_5 is a correction factor for meandering of channel. Proper values of n_0 to n_4 and m_5 may be selected from Table 5-5 according to the given conditions.

[1] The n/n_0 curve was based on measurements by Wilcox [29] on 8-in. clay and concrete sewer pipes and by Yarnell and Woodward [30] on open-butt-joint concrete and clay drain tiles 4 to 12 in. in size. For depths less than about $0.15d_0$, the curve was verified by the data of Johnson [31] for large sewers.

In selecting the value of n_1, the degree of irregularity is considered *smooth* for surfaces comparable to the best attainable for the materials involved; *minor* for good dredged channels, slightly eroded or scoured side slopes of canals or drainage channels; *moderate* for fair to poor dredged channels, moderately sloughed or eroded side slopes of canals or drainage channels; and *severe* for badly sloughed banks of natural streams, badly eroded or sloughed sides of canals or drainage channels, and unshaped, jagged, and irregular surfaces of channels excavated in rock.

In selecting the value of n_2, the character of variations in size and shape of cross section is considered *gradual* when the change in size or shape occurs gradually, *alternating occasionally* when large and small sections alternate occasionally or when shape changes cause occasional shifting of main flow from side to side, and *alternating frequently* when large and small sections alternate frequently or when shape changes cause frequent shifting of main flow from side to side.

The selection of the value of n_3 is based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders, and fallen and lodged logs. One should recall that conditions considered in other steps must not be reevaluated or double-counted in this selection. In judging the relative effect of obstructions, consider the following: the extent to which the obstructions occupy or reduce the average water area, the character of obstructions (sharp-edged or angular objects induce greater turbulence than curved, smooth-surfaced objects), and the position and spacing of obstructions transversely and longitudinally in the reach under consideration.

In selecting the value of n_4, the degree of effect of vegetation is considered

(1) *Low* for conditions comparable to the following: (*a*) dense growths of flexible turf grasses or weeds, of which Bermuda and blue grasses are examples, where the average depth of flow is 2 to 3 times the height of vegetation, and (*b*) supple seedling tree switches, such as willow, cottonwood, or salt cedar where the average depth of flow is 3 to 4 times the height of the vegetation.

(2) *Medium* for conditions comparable to the following: (*a*) turf grasses where the average depth of flow is 1 to 2 times the height of vegetation, (*b*) stemmy grasses, weeds, or tree seedlings with moderate cover where the average depth of flow is 2 to 3 times the height of vegetation, and (*c*) brushy growths, moderately dense, similar to willows 1 to 2 years old, dormant season, along side slopes of a channel with no significant vegetation along the channel bottom, where the hydraulic radius is greater than 2 ft.

(3) *High* for conditions comparable to the following: (*a*) turf grasses where the average depth of flow is about equal to the height of vegetation,

(*b*) dormant season—willow or cottonwood trees 8 to 10 years old, intergrown with some weeds and brush, none of the vegetation in foliage, where the hydraulic radius is greater than 2 ft, and (*c*) growing season—bushy willows about 1 year old intergrown with some weeds in full foliage along side slopes, no significant vegetation along channel bottom, where hydraulic radius is greater than 2 ft.

(4) *Very high* for conditions comparable to the following: (*a*) turf grasses where the average depth of flow is less than one-half the height of vegetation, (*b*) growing season—bushy willows about 1 year old, intergrown with weeds in full foliage along side slopes, or dense growth of cattails along channel bottom, with any value of hydraulic radius up to 10 or 15 ft, and (*c*) growing season—trees intergrown with weeds and brush, all in full foliage, with any value of hydraulic radius up to 10 or 15 ft.

In selecting the value of m_5, the degree of meandering depends on the ratio of the meander length to the straight length of the channel reach. The meandering is considered *minor* for ratios of 1.0 to 1.2, *appreciable* for ratios of 1.2 to 1.5, and *severe* for ratios of 1.5 and greater.

In applying the above method for determining the n value, several things should be noted. The method does not consider the effect of suspended and bed loads. The values given in Table 5-5 were developed from a study of some 40 to 50 cases of small and moderate channels. Therefore, the method is questionable when applied to large channels whose hydraulic radii exceed, say, 15 ft. The method applies only to unlined natural streams, floodways, and drainage channels and shows a minimum value of 0.02 for the n value of such channels. The minimum value of n in general, however, may be as low as 0.012 in lined channels and as 0.008 in artificial laboratory flumes.

5-9. The Table of Manning's Roughness Coefficient. Table 5-6 gives a list of n values for channels of various kinds.[1] For each kind of channel the minimum, normal, and maximum values of n are shown. The normal values for artificial channels given in the table are recommended only for channels with good maintenance. The boldface figures are values generally recommended in design. For the case in which poor maintenance is expected in the future, values should be increased according to the situation expected. Table 5-6 will be found very useful as a guide to the quick selection of the n value to be used in a given problem. A popular table of this type was prepared by Horton [34] from an examination of the best available experiments at his time.[2] Table 5-6 is compiled

[1] The minimum value for Lucite was observed in the Hydraulic Engineering Laboratory at the University of Illinois [33]. Such a low n value may perhaps be obtained also for smooth brass and glass, but no observations have yet been reported.

[2] A table showing n values and other elements from 269 observations made on many existing artificial channels is also given by King [35].

TABLE 5-5. VALUES FOR THE COMPUTATION OF THE ROUGHNESS COEFFICIENT BY EQ. (5-12)

Channel conditions			Values
Material involved	Earth	n_0	0.020
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of channel cross section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.010–0.015
Relative effect of obstructions	Negligible	n_3	0.000
	Minor		0.010–0.015
	Appreciable		0.020–0.030
	Severe		0.040–0.060
Vegetation	Low	n_4	0.005–0.010
	Medium		0.010–0.025
	High		0.025–0.050
	Very high		0.050–0.100
Degree of meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n
(Boldface figures are values generally recommended in design)

Type of channel and description	Minimum	Normal	Maximum
A. CLOSED CONDUITS FLOWING PARTLY FULL			
A-1. Metal			
a. Brass, smooth	0.009	0.010	0.013
b. Steel			
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

from up-to-date information collected from various sources ([34,36,38], and unpublished data); hence it is much broader in scope than the Horton table.

5-10. Illustrations of Channels with Various Roughnesses. Photographs of a number of typical channels, accompanied by brief descriptions of the channel conditions and the corresponding n values, are shown in Fig. 5-5. These photographs are collected from different sources and arranged in order of increasing magnitude of the n values. They provide a general idea of the appearance of the channels having different n values and so should facilitate selection of the n value for a given channel condition. The n value given for each channel represents approximately the coefficient of roughness when the photograph was taken.

The above type of visual aid is also employed by the U.S. Geological Survey. The Survey has made several determinations of channel roughness in streams, mostly in the northwestern United States. These include measurements of cross-sectional area, width, depth, mean velocity, slope, and computation of the roughness coefficient. The reaches were photographed in stereoscopic color, and the photographs have been circulating among the district offices of the Survey as a guide in evaluating n.

WATER SUPPLY AND POLLUTION CONTROL

changes in grade, pipe size, direction of flow, and quantity of flow. Figure 6-14 illustrates the details of a typical manhole. Storm drains are usually not built smaller than 12 in. in diameter, because pipes of lesser size tend to clog readily with debris and therefore present serious malfunction problems. Maximum manhole spacing for pipes 27 in. and under should not exceed about 600 ft. For larger pipes, no maximum is prescribed and the normal requirements for structures should provide access to the drain for inspection, cleaning, or maintenance.

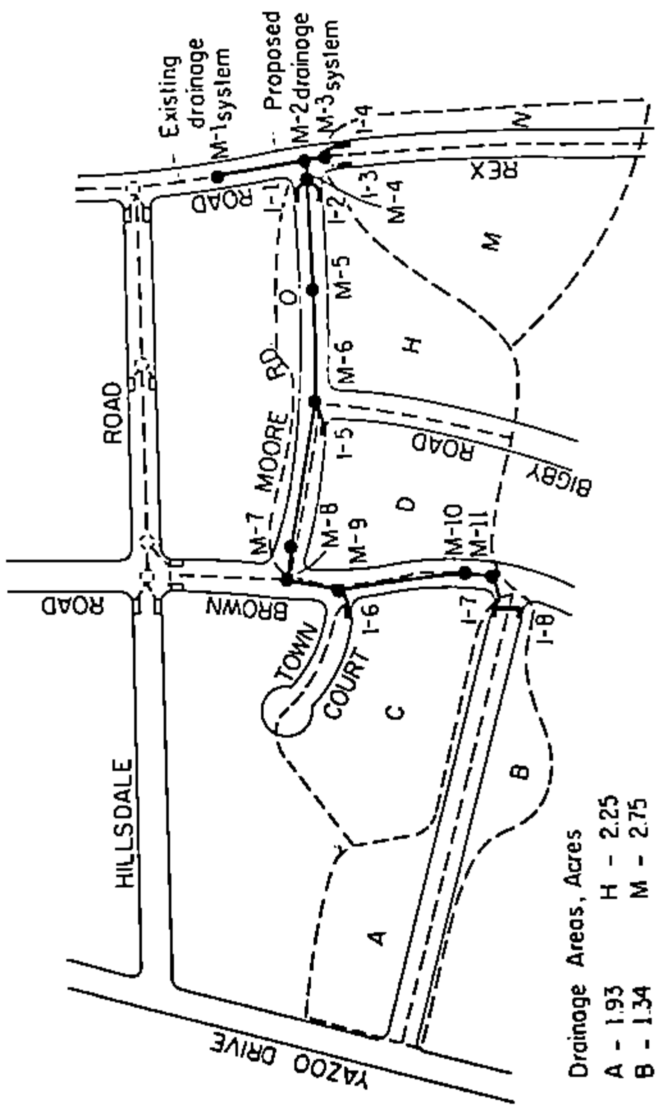
6-18. HYDRAULIC DESIGN OF URBAN STORM-DRAINAGE SYSTEMS

Basic principles outlined at the beginning of the chapter are sufficient to adequately design an urban drainage system. Calculations for the drainage area shown in Fig. 6-15 are presented in detail to illustrate the mechanical procedure and the rational method as applied to an actual design problem. The overall Mextex area is an urban residential area made up of single-family dwellings and is divided into 8 subareas which are tributary to individual storm-water inlets.

EXAMPLE 6-4. Design a storm-drainage system to carry the flows from the 8 inlet areas given in Fig. 6-15. It will be assumed that a 10-year frequency rainfall satisfies the local design requirements. Assume clay soil to be predominant in the area with average lawn slopes.

Solution: The steps for solving are as follows:

1. Prepare a drainage-area map showing drainage limits, streets, impervious areas, and direction of surface flow.
2. Divide the drainage area into subareas tributary to the proposed storm-water inlets (Fig. 6-15).
3. Compute the acreage and imperviousness of each area.
4. Calculate the required capacity of each inlet, using the rational method. Assume a 5-min inlet time to be appropriate and compute inlet flows for a rainfall intensity of 7.0 in./hr. This is obtained by using the 10-year frequency curve on Fig. 6-11 with a 5-min concentration time. Appropriate C values are obtained from Fig. 6-10 by entering the graph with the calculated percentage imperviousness (the percent of the inlet area which is covered by streets, sidewalks, drives, roofs, etc.), projecting up to the average lawn slope curve and reading C on the ordinate. Computations for the inlet flows are tabulated in Table 6-7.
5. Select the type inlets required to adequately drain the flows in Table 6-7. The choice will be based on a knowledge of the street slopes and their relation to various inlet capacities. Inlet capacity curves such as given in Fig. 6-13 would be used. For the purposes of this example, no actual selections will be made but the student should recognize that this is the next logical step and an exceedingly important one.
6. Beginning at the upstream end of the system, compute the discharge to be carried by each successive length of pipe, moving downstream. These calculations are summarized in Table 6-8. Note that at each point downstream where a



Drainage Areas, Acres

A - 1.93	H - 2.25
B - 1.34	M - 2.75
C - 319	N - 1.27
D - 2.66	O - 0.40

FIG. 6-15. Plan view of Mextex storm-drainage system.

TABLE 6-7
REQUIRED STORM-WATER INLET CAPACITIES FOR EXAMPLE 6-5

1	2	3	4	5	6	7
Inlet	Area Designation	Area Acres	Percent Impervious	C	Rainfall Intensity	Q, cfs (3) × (5) × (6)
I-1	O	0.40	49	0.57	7.0	1.59
I-2	H	2.25	20	0.35	7.0	5.52
I-3	M	2.75	26	0.40	7.0	7.70
I-4	N	1.27	26	0.40	7.0	3.55
I-5	D	2.66	26	0.40	7.0	7.43
I-6	C	3.19	24	0.38	7.0	8.48
I-7	A	1.93	23	0.37	7.0	4.99
I-8	B	1.34	29	0.42	7.0	3.93

new flow is introduced, a new time of concentration must be determined as well as new values of C and drainage area size. As the upstream inlet areas are combined to produce a larger tributary area at some design point, a revised C value representing these combined areas must be obtained. Usually the procedure is to take a weighted average of all the individual C values of which the larger area is composed. For example, when computing the flow to be carried by the pipe from M-9 to M-8, the tributary area is A + B + C = 6.46 acres, and the composite value of C will be

$$C = \frac{\Sigma c_i a_i}{\Sigma a_i} = \frac{0.37 \times 1.93 + 0.42 \times 1.34 + 0.38 \times 3.19}{6.46} = 0.38$$

At the design location the value of t_c will be equal to the inlet time at I-8 plus the pipe flow time from I-8 to M-9 (see Table 6-8), which must be known to permit solving the rainfall intensity to be used in computing the runoff from composite area A + B + C.

7. Using the computed discharge values, select tentative pipe sizes for the approximate slopes given in column 8 of Table 6-8. Once the pipe sizes are known, flow velocities between input locations can be determined. Normally these velocities are approximated by computing the full flow velocities for maximum discharge at the specified grade. These velocities are then used to compute channel flow time for estimating the time of concentration. If upon completing the hydraulic design enough change has been made in any concentration time to alter the design discharge, new values of flow should be computed. Generally this will not be the case.

8. Using the pipe sizes selected in Step 7, draw a profile of the proposed drainage system. Begin the profile at the point farthest downstream, which can be an outfall into a natural channel, an artificial channel, or an existing drain as in the case of the example. In constructing the profile, be certain that the pipes have at least the minimum required cover. Normally 1.5 to 2 ft is sufficient. Pipe slopes should conform to the surface slope wherever possible. At all manholes indicate the necessary change in invert elevation. In this example where there is no change in pipe size through the manhole, a drop of 0.2 ft will be used. Where the size decreases upstream through a manhole, the upstream invert

TABLE 6-8
COMPUTATION OF DESIGN PIPE FLOWS FOR THE STORM-DRAINAGE SYSTEM OF EXAMPLE 6-4

1	2	3	4			5	6	7	8			
Pipe Section	Tributary Area	Area, acres	Flow Time, min			Rainfall Intensity,	C	Q, cfs	Pipe			
			Inlet	Pipe	Total				Slope, percent	Size, in.	Full-Flow Velocity, fps	Length, ft
I-8-I-7	B	1.34	5	0.18	5	7.0	0.42	3.93	1.0	15	5.2	30
I-7 M-11	A + B	3.27	5	0.13	5.10	7.0	0.39	8.93	1.0	18	5.9	46
M-11 M-10	A + B	3.27	—	0.24	5.23	—	0.39	8.93	1.0	18	5.9	85
M-10-M-9	A + B	3.27	—	0.37	5.47		0.39	8.93	2.0	18	8.1	178
I-6-M-9	C	3.19	5	---	5	7.0	0.38	8.48	1.0	18	5.9	40
M-9-M-8	A + B + C	6.46	—	0.21	5.80	6.9	0.38	16.90	1.8	21	8.5	110
M-8-M-7	A + B + C	6.46	—	0.11	6.01	—	0.38	16.90	1.8	21	8.5	57
M-7-M-6	A + B + C	6.46	—	0.47	6.12		0.38	16.90	1.6	21	8.1	230
I-5-M-6	D	2.66	5	—	5	7.0	0.40	7.43	2.0	15	7.4	19
M-6-M-5	A + B + C + D	9.12	—	0.38	6.59	6.8	0.39	24.20	2.0	24	10.0	230
M-5-M-4	A + B + C + D	9.12	---	0.42	6.97	-	0.39	24.20	1.9	24	9.8	247
I-1-M-4	O	0.40	5	--	5	7.0	0.57	1.59	3.0	15	9.0	19
I-2-M-4	H	2.25	5	-	5	7.0	0.35	5.52	3.0	15	9.0	17
M-4-M-2	A + B + C + D + O + H	11.77	--	0.05	7.39	6.6	0.39	30.3	1.5	27	9.4	29
I-3 M-3	M	2.75	5	·	5	7.0	0.40	7.70	2.0	15	7.4	15
I-4 M-3	N	1.27	5	—	5	7.0	0.40	3.55	2.0	15	7.4	20
M-3-M-2	M + N	4.02	—	--	5	7.0	0.40	11.30	1.8	18	7.8	37
M-2-M-1	A + B + C + D + O + H + M + N	15.79	—	---	7.44	6.6	0.39	40.6	1.4	30	9.8	176

will be set above the downstream invert a distance equal to the difference in the two diameters. In this way the crowns are kept at the same elevation. A part of the profile of the drainage system in the example is given in Fig. 6-16.

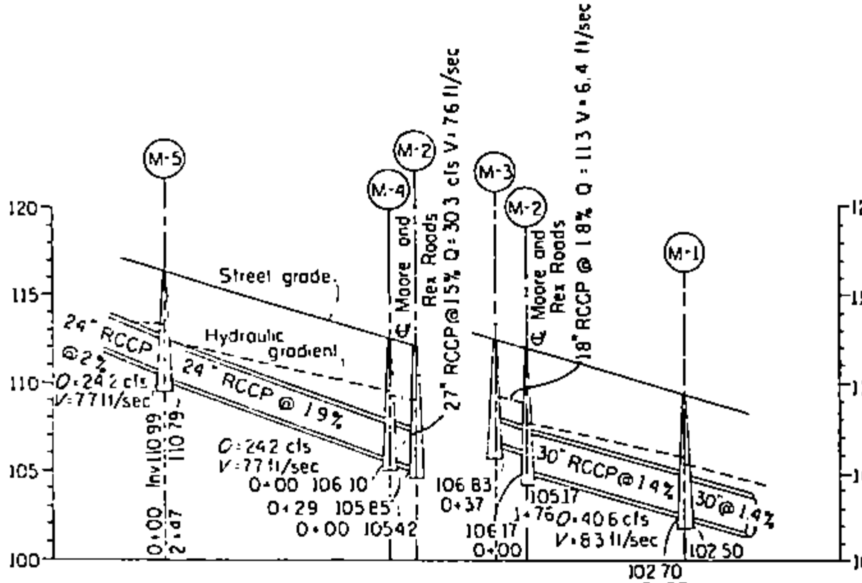


FIG. 6-16. Profile of part of the Mextex storm drain, showing the hydraulic gradient.

9. Compute the position of the hydraulic gradient along the profile of the pipe. If this gradient lies less than 1.5 ft below the ground surface, it must be lowered to preclude the possibility of surcharge during the design flow. Note that the value of 1.5 ft is arbitrarily chosen here. In practice, local standards indicate the limiting value. Hydraulic gradients may be lowered by increasing pipe sizes, decreasing head losses at structures, by designing special transitions, by lowering the system below ground, or by some combination of these means.

Computations for a portion of the hydraulic gradient of the example will now be given. Head losses in the pipes are determined by applying Manning's equation, assuming $n = 0.013$ in this example. Head losses in the structures will be determined by using the relationships defined in Figs. 6-17 and 6-18. These curves were developed for surcharged pipes entering rectangular structures but may be applied to wye branches, manholes, and junction chambers as pictured on the curves.[22] The "A" curve is used to find entrance and exit losses, the "B" curve to evaluate the head loss due to an increased velocity in the downstream direction. The loss is designated as the difference between the head losses found for the downstream and upstream pipe ($V_{h-2} - V_{h-1}$). In cases where the greatest velocity occurs upstream, the difference will be negative and may be applied to offset other losses in the structure. The "C" loss results from a change in direction in a manhole, wye branch, or bend structure. The "D" loss is related

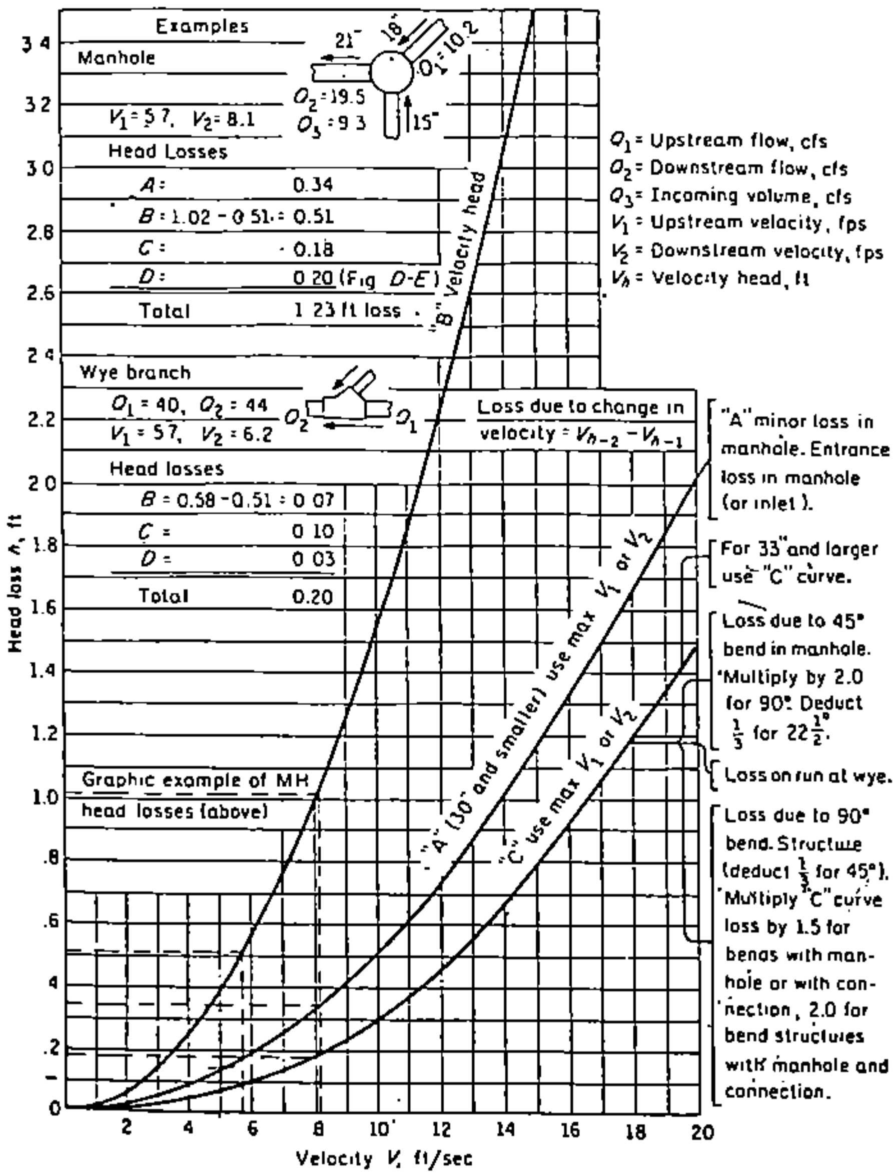


FIG. 6-17. Types A, B, and C head losses in structures. (Courtesy of the Baltimore County Department of Public Works, Towson, Maryland.)

to the effects produced by the entrance of secondary flows into the structure. Several examples of the use of these curves are shown on Figs. 6-17 and 6-18.

Computations for the hydraulic gradient shown in Fig. 6-16 are as follows:

(a) Begin at the elevation of the hydraulic gradient at the upstream end of the existing 30-in. reinforced-concrete culvert pipe (RCCP). This elevation is 105.50. The existing hydraulic gradient is shown on Fig. 6-16.

D loss = 0; no secondary flow
Total = 0.36 ft

The hydraulic gradient therefore rises in the manhole to an elevation of 105.50 + 0.36 = 105.86 ft, plotted in M-1 on Fig. 6-16.

(c) Compute the head loss due to friction in the 30-in. drain from M-1 to M-2. Assume n = 0.013. Using Manning's equation, the head loss per linear foot of drain is

$$S = \frac{(nV)^2}{2.21 R^{4/3}} \qquad (6\text{-}13)$$

and from M-1 to M-2,

$$S = \frac{(0.013 \times 8.3)^2}{2.21 \times 0.534} = 0.0166$$

The total frictional head loss is therefore

$$hf = S \times L = 0.0116 \times 176 = 1.73 \text{ ft}$$

Elevation of the hydraulic gradient at the downstream end of M-2 is thus 105.86 + 1.73 = 107.59 ft. This elevation is plotted on Fig. 6-16, and the hydraulic gradient in this reach is drawn in.

(d) Compute the head losses in M-2.

A = 0.36 (V = 8.3 ft/sec)
B = 1.07 − 0.90 = 0.17 (V_2 = 8.3, V_1 = 7.6 = Q/A for 27-in. drain)
C = 0.20 × 2.0 (multiply by 2 for 90 degree bend in manhole—see Fig. 6-17) = 0.40
D = 0.22 for Q_3/Q_1 = 11.3/30.3 = 37 percent

Total head loss in M-2 equals 1.15 ft and the elevation of the hydraulic gradient in M-2 is therefore 107.59 + 1.15 = 108.74 ft.

(e) Compute the friction head loss in the section of pipe from M-2 to M-3.

$$S = \frac{(0.013 \times 6.4)^2}{2.21 \times 0.272} = 0.0113$$

$$hf = 0.0113 \times 37 = 0.42 \text{ ft}$$

Elevation of the hydraulic gradient at the downstream end of M-3 is therefore 108.74 + 0.42 = 109.16 ft. Plot this point on the profile and draw the gradient from M-2 to M-3.

Students should realize that the hydraulic gradient in this example was computed under the assumption of uniform flow. In closed conduit systems, if the pipes are flowing full or the system is surcharged (the usual design flow conditions), this method will produce good results. Where open conduits are used, or in partial-flow systems, the hydraulic gradient can be determined by computing surface profiles in the manner described in Sec. 6-2.

Computations for the remainder of the hydraulic gradient are identical to those just given and will not be presented. It should be noted that