CITY OF ANN ARBOR, MICHIGAN
STORMWATER MANAGEMENT PLAN
FOR
ALLEN'S CREEK DRAINAGE SYSTEM

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SUMMARY AND RECOMMENDATIONS

The City of Ann Arbor Stormwater Management Plan for Allen's Creek Drainage System addresses the issues of flooding occurrences on the system and alternative means of alleviating flooding problems.

The Allen's Creek Drainage System includes the central and western portions of the City of Ann Arbor, generally bounded on the west by I-94, on the north by M-14, on the east by Washtenaw Avenue and on the south by Scio Church Road. The trunk storm sewers include the Allen's Creek Drain and three major tributaries, West Park - Miller Drain, Murray - Washington Drain, and Eberwhite Drain.

The climate of Ann Arbor is moderate, with an average yearly temperature of 50°F and extremes on record from -21°F to 105°F. Average yearly precipitation is approximately 30-inches, with the greatest daily precipitation event, since 1940, occurring in June 1968 of 4.74 inches. Snowfall averages approximately 30 inches yearly.

The Allen's Creek drainage basin covers approximately 3500 acres, while the major tributaries, West Park - Miller Drain, Murray - Washington Drain and Eberwhite Drain cover 825, 725 and 275 acres, respectively.

The Allen's Creek drainage basin slopes towards the Allen's Creek Drain from the west and east. The Allen's Creek Drain slopes to the north and empties into the Huron River. The topography slopes are generally moderate to steep. Development within the system is mostly residential and commercial with some light industrial and parkland/undeveloped lands.

The overall storm sewer condition is considered to be generally good and suitable for continued use. Storm sewer inspections conducted in the fall of 1968, fall of 1974 and spring of 1982 revealed the need for minor structural improvements, repairs and general cleaning and maintenance. Joint repairs, invert replacement and patching of exposed reinforcing steel are the most significant improvements and repairs needed. Several suspected sanitary sewer connections were observed during the inspections. They should be verified and removed. Existing sediment, rock, gravel and other debris should be removed as part of the general cleaning and maintenance program. The project cost for this work was estimated to be $1,100,000.

Historical data relating hydrologic and hydraulic information to the Allen's Creek Drainage System include various reports prepared by McNamee, Porter and Seeley, the City of Ann Arbor Flood Insurance Study, and the Special Flood Hazard Information Report on the Huron River. In addition, precipitation records for the City of Ann Arbor and stream flow records for the Huron River were investigated. The June 1968 flood was investigated to determine flooding extent and water surface elevations. Private citizens were interviewed to verify flooding and water surface elevations observed during the 1968 flood. This information was utilized to calibrate the hydrologic and hydraulic models used in the study.

Existing storm sewer capacities were evaluated by utilizing storm sewer profiles and sizes from construction drawings and physical conditions determined from field investigations. Capacities were determined for flowing full conditions and surcharged conditions. Capacities were utilized in the overall hydrologic model to determine overland flows from the 10 and 100 year flood frequencies.

Peak flow predictions were developed at selected locations along Allen's Creek Drain and the main tributaries for the 10 and 100 year frequencies. Resultant overland flows were developed by subtracting surcharged storm sewer capacities from the peak flow rates. Overland flows occur in natural channels/swales and in roads, often flowing between and around man-made structures, thus flooding many areas in the Allen's Creek drainage basin. The 10 and 100 year frequency floods were input into hydraulic models of the various drains to determine water surface elevations at specific locations along the respective drain. Flood profiles were developed to show water surface elevations, and flood plain delineations were drawn on contour maps to show the extent of flooding. Comparison of historical data to modeled floods indicated that channel storage due to the overland flow was significant along Allen's Creek. The storage acts to decrease peak flow rates and resultant water surface elevations near the downstream end. The results indicated good agreement with conditions experienced during the 1968 flood event. A set of profiles was also computed to show the effect of removal of the channel storage. This could occur by further filling or development in the flood plain areas. The profiles indicate that significant increases in flooding depths would occur, especially in the vicinity of Summit Park.

Three general categories of alternatives for handling flooding problems were studied; reduction of peak flows, reduction in flood stages and reduction in damages. Four alternatives were deemed viable and were investigated. Each alternative includes the recommended storm sewer rehabilitation program, involving repair and cleaning of existing storm sewers at an estimated project cost of $1,100,000.

Alternative 1 would provide storm sewer relief sized such that the 10 year flood flows would be handled by the storm system. The estimated cost of construction of this alternative is $18,000,000.

Alternative 2 would provide overland flow channels to transport the 10 year frequency flows. This alternative would involve constructing a grassy swale in low areas and around existing structures. In addition, several culverts are included to pass flow beneath roads. Several retention basins would partially offset storage lost due to channelization. The estimated project cost of this alternative is $7,700,000.

Alternative 3 involves utilizing nonstructural measures to maximize the existing system, to prevent further increases in flood elevations, and to adapt to the flooding. The measures would involve rehabilitating the existing storm sewers and informing residents of the availability of federally subsidized flood insurance, flood-proofing basements, and keeping valuables above flood levels. The estimated project cost of rehabilitating the storm sewers is $1,100,000, with an additional cost for administration and engineering of $50,000 for providing residents with nonstructural flood prevention techniques.

Alternative 4 is a combination of nonstructural measures, rehabilitating the storm sewers and the overland flow channel improvements. Reaches which are determined to be most affected by flooding would be selected to be improved by the overland flow improvements. This would reduce flooding in critical areas. The estimated project cost is $5,300,000.

The recommended alternative is Alternative 4. The storm sewer rehabilitation work of $1,100,000 would optimize the existing system and prevent further deterioration. It would provide a modest beneficial impact on the carrying capacity of the drain, and also greatly reduce the potential of obstructions in the flow during a storm. The study shows that significant increases in flooding elevations will occur if further development or filling is allowed in the floodplain areas. Nonstructural measures can be taken to prevent further increases in flood elevations and to educate citizens of the study results so that mitigating measures can be taken in affected areas.

Nonstructural improvements would involve some administration and engineering costs, estimated to be $50,000. In order to reduce the flooding levels in areas experiencing the worst conditions, channelization and culvert improvements are recommended on Allen's Creek from the Huron River to West Jefferson St., and on Eberwhite Drain from 2nd St. to 4th St. The estimated project cost is $4,100,000. In most reaches, the maximum depth of flooding adjacent to structures for the 10 year event is expected to be less than 3 feet. The total estimated project cost for the recommended Alternative 4 is $5,300,000.

Whichever alternative is chosen, it is recommended that as new development occurs, retention of storm runoff be required in order to restrict such runoff to the amount there would be under existing conditions. This is currently practiced in Ann Arbor and should be continued.

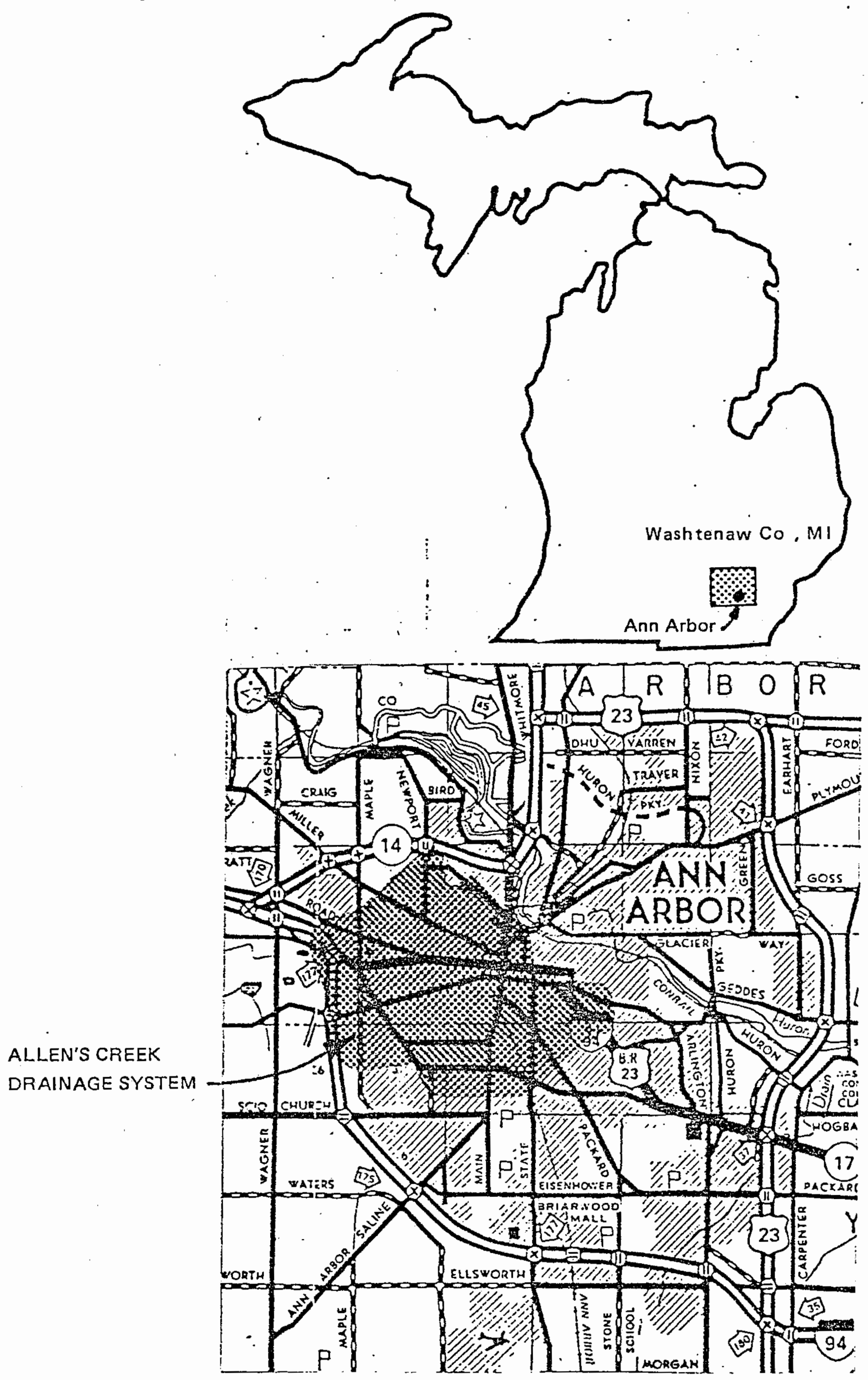
INTRODUCTION

The City of Ann Arbor Stormwater Management Plan for Allen's Creek Drainage System addresses the stormwater flooding problems in the central and western portions of the City. Figure 1 indicates the location of the study area. The Allen's Creek Drainage System, particularly the Allen's Creek Drain, West Park-Miller Drain, Murray-Washington Drain and Eberwhite Drain, was studied with respect to overall watershed characteristics including natural and man-made conditions, historical data concerning storm and flood information and existing conditions relating hydrologic conditions to yield 10 and 100 year frequency flood flows and resulting hydraulic water surface elevations. Finally, recommended alternatives to alleviate the flooding problems were developed along with cost estimates.

The current study is a continuation of preceding studies prepared by this firm in conjunction with the firm of Johnson, Johnson and Roy in 1972 and 1974. The purpose of these studies was to determine the extent of flooding in the June 1968 event, suggest various alternatives for reducing the damages, and develop costs for the pipe relief alternative. The current study expands on the flooding analysis and investigates other viable alternatives.

FIGURE 1

LOCATION MAP



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HISTORICAL DATA

PREVIOUS REPORTS

Information regarding the Allen's Creek drainage basin, including hydrologic and hydraulic data, was investigated. This information is in the form of previous reports. The reports and the contributing information are summarized in the following paragraphs.

The first two reports (McNamee, Porter and Seeley, Consulting Engineers. "Report on Allen's Creek Drain for Washtenaw County Drain Commission and City of Ann Arbor," September 1972; and McNamee, Porter and Seeley and Johnson, Johnson and Roy, Inc., Planners. "Allen's Creek Drain - Analysis of Preliminary Alternatives for Relief: Interim Report Prepared for the Washtenaw County Drain Commission," February 1974.) discussed the flooding event of June 1968. The Allen's Creek drainage basin experienced severe flooding due to this event. The storm sewer system was investigated, flood plain delineations based on observed water elevations were estimated, and a preliminary design for relief sewers along with cost estimates was developed. Physical characteristics of existing storm sewers, hydrologic information and physical characteristics of the Allen's Creek drainage basin were utilized from these reports.

The third report (McNamee, Porter and Seeley, Consulting Engineers. "Report on Liberty Street Retention Basin on the Murray-Washington Drain," September 1978.) provided specific information regarding the hydrology and related characteristics for a proposed retention site on the Murray-Washington Drain. A preliminary design basis was provided with respect to retention basin size and inlet/outlet structures with respect to existing storm sewers.

A flood insurance study (Federal Emergency Management Agency, "Flood Insurance Study, City of Ann Arbor, Washtenaw County, Michigan," Wade, Trim and Associates, 1981.) provided hydrologic and hydraulic information for the Huron River at the outlet of Allen's Creek and the Allen's Creek Drain itself from the Huron River upstream to Hoover Avenue. The flood insurance study provided starting water surface elevations for Allen's Creek at the Huron River. The current study on Allen's Creek Drain provides an extensive hydrologic and hydraulic analysis of the Allen's Creek Drain, much more so than the flood insurance study. However, the flood insurance study provides an excellent comparison base in order to ensure that the current study is compatible with previous engineering analyses.

In addition, the Corps of Engineers' 1975 report, "Special Flood Information Report - Huron River," was utilized as backup data for the above mentioned flood insurance study. This report provides hydrologic and hydraulic data relative to the Huron River for the 1968 flood event and various flood frequency events, specifically the 50, 100 and 500 year events.

STORMS AND FLOODS

An investigation of precipitation and Huron River flows from 1965 to the present was conducted. This information was obtained to determine occurrence of intense rainfall events and subsequent flooding in the recent past. Precipitation data (National Oceanic and Atmospheric Administration, "Hourly Precipitation Data, Michigan", 1965-present.) was tabulated for 24-hour rainfalls of greater than 1.5 inches. Concurrent Huron River flows were then tabulated for a time period preceding and following those rain events (U.S. Geological Survey, "Water Resources Data for Michigan," 1965-present).

Table 1 shows that there have been 16 rain events with 24 hour rainfalls of greater than 1.5 inches since 1965. These larger rainfalls have caused some flooding to various degrees in the Allen's Creek drainage basin. However, the magnitude of the flooding problem for a particular event may be substantially different when compared with another event. This is shown by the tabulation of the Huron River flows at Ann Arbor for the respective rain events. River flows are tabulated in Table 1 also.

Table 1

Historical Rain Events and Huron River Flows

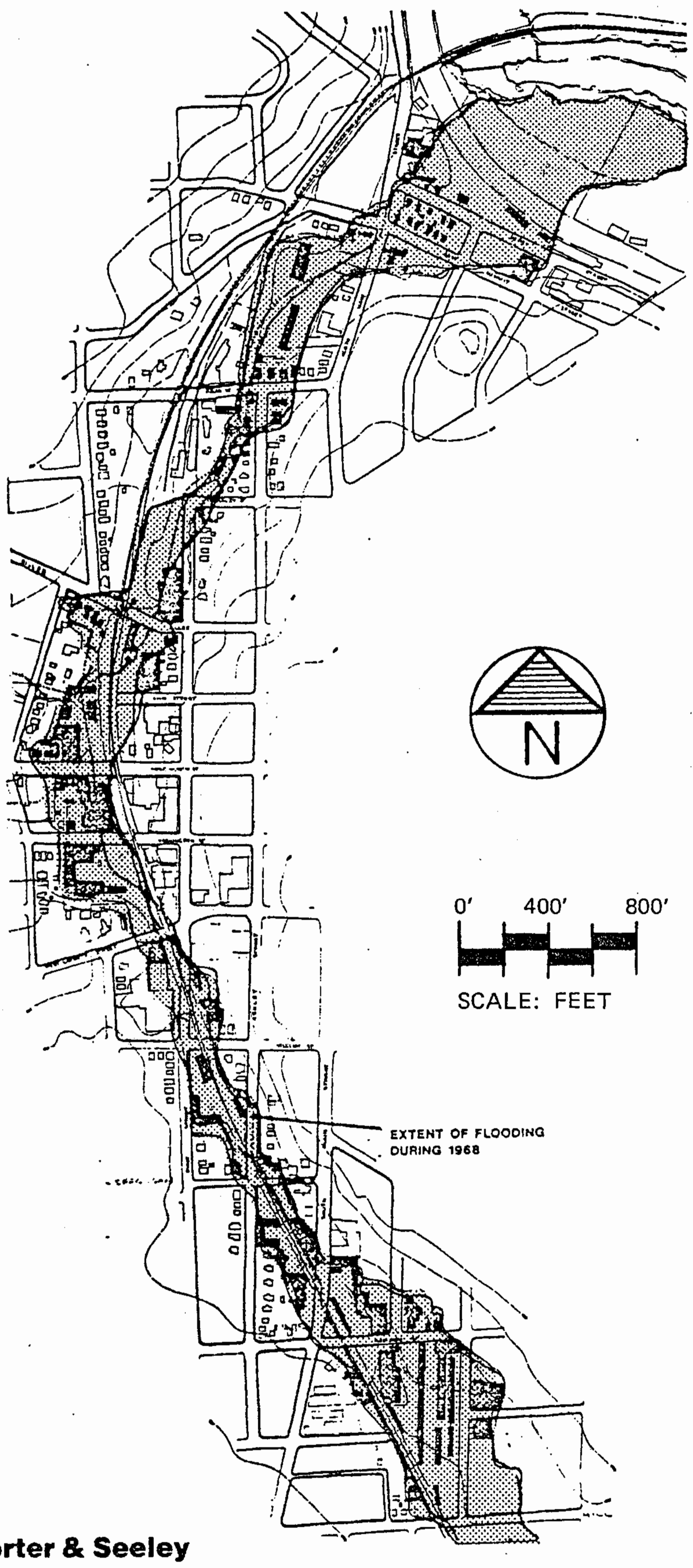
Date	Total Rainfall (inches)	Average Daily Huron River Flow (cfs) Before Rain	After Rain
12/24/65	2.36	265	1150
7/12/66	2.39	150	285
6/28/67	1.90	390	640
7/19/67	1.97	195	405
12/21/67	2.64	419	1250
5/26/68	2.66	435	2080
6/21/68	2.03	345	1010
6/25/68	4.74	1010	4010
8/16/68	2.00	570	650
11/20/69	1.55	680	925
9/27/71	3.94	95	350
8/29/75	1.9	405	1150
8/30/75	2.4	1150	2430
5/6/76	2.4	950	2160
9/3/81	2.2	415	1190
6/29/82	3.46	460	610

As is evidenced by the data, Huron River flow is not directly proportional to rainfall at Ann Arbor. Other factors which contribute significantly to resultant river discharges following a rain event are antecedent precipitation and resultant soil saturation, rainfall intensity and rainfall on the total drainage basin. These factors are illustrated by observation of several events. For example, extreme flooding occurred on the Allen's Creek basin in June 1968 as compared with a heavy flow condition observed in the Huron River. On the other hand, minimal flooding was observed on the Allen's Creek basin in June 1982, as compared with a minimal flow increase in the Huron River. These large extremes in flooding and Huron River flows were observed even though rainfall for the 1968 and 1982 rain events was 4.74 inches and 3.46 inches, respectively. Probable major causes for the different resultant flows were the extent of rainfall and the preceding rainfall. In 1968, the rainfall occurred throughout the basin. Rainfall on preceeding days had increasd soil moisture. In 1982, the rainfall was localized with some areas receiving more intense thundershowers. Also the ground was very dry, allowing greater infiltration into the soil than during the 1968 event.

The approximate high-water line during the June 1968 flood along the Allen's Creek Drain was determined by (1) analysis of the City complaint record of flooding and City observations of water levels at bridges and culverts, (2) establishment of known high water marks by interviews with property owners, and (3) interpolation of flood contours based on data from (1) and (2).The area enclosed by the 1968 high water mark as shown on Figure 2 represents the best available determination of the floodplain for the Allen's Creek Drain from this flooding event. It is estimated that the June 25, 1968 event had a recurrence interval of 50 to 100 years.

The most serious flooding in the Allen's Creek drainage basin during the 1968 flood and other less severe flooding events occurred along the entire length of the Allen's Creek Drain beginning at Hoover Street and continuing downstream to the outlet just below Argo Dam. The West Park-Miller Drain experiences flooding in the vicinity of Arborview and Maple Ridge and also in West Park between Miller Avenue and Huron Street west of Chapin. Manhole covers on the West Park sewer have been lifted off on numerous occasions due to the pressure within the overloaded sewer. The Murray-Washington Drain experiences serious flooding between Washington Street and Murray Street, primarily in the area surrounding the former Washtenaw County Road Commission office and yard, which is presently owned and used by the City of Ann Arbor. The flooding which occurs in the Allen's Creek Drain has on different occasions flooded first floor levels of residences and businesses in the low lying areas. These include the Fingerle Lumber Company, the City offices on Washington Street, the Glacier Corp. Building which is located between Washington and Huron Streets east of Chapin, the City yard located on North Main and Summit Street, and the business establishments in the vicinity of North Main, Depot and Summit Streets. The residences in the vicinity of Depot and Summit Streets receive the most severe flooding by comparison to other flooded upstream residences.

FIGURE 2
1968 FLOODING EXTENT



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WATERSHED CHARACTERISTICS

GENERAL

The Allen's Creek basin is bounded on the west by Interstate 94; on the north by a line extending northwesterly from the Jackson Road and Interstate 94 intersection to Newport Road and the M-14 bypass, then southwesterly along Sunset Road to the Huron River; on the east by a line which extends south and southeasterly from the Huron River and Argo Dam to the intersection of Washtenaw Street and Geddes Road, on Geddes to Berkshire, south on Berkshire to Vinewood Blvd; and on the south by a line which extends southwesterly from Washtenaw Street and Vinewood Blvd. to South Main and Scio Church Road and westerly from South Main and Scio Church Road to Interstate 94.

The Allen's Creek Drain together with its three westerly branches follows natural drainage courses. At one time there were open drainage courses, but as the area developed, it became necessary to construct culverts for street crossings. The total enclosure of the drainage courses followed as development of the area increased.

The Allen's Creek Drain generally parallels the Ann Arbor Railroad through the City of Ann Arbor and the established drain extends downstream from Stadium Blvd. along the Ann Arbor Railroad to its outlet into the Huron River immediately downstream of Argo Dam. The Allen's Creek Drain was constructed in 1925 from plans prepared by Russell A. Dodge, Civil Engineer. The drain varies in size from a 120-inch by 96-inch box culvert at its outlet to special horseshoe section sewers that vary in size from the largest of 166-inch by 102-inch down to 119-inch by 66-inch, and terminates upstream (west) of Seventh Street as a 21-inch diameter sewer.

The Murray-Washington Drain was constructed in 1927 from plans prepared by Menefee and Dodge, Engineers. This drain extends from its junction with the Allen's Creek Drain at the foot of Ann Street southwesterly and then parallel to the north side of Liberty Street to an open area at Liberty and Dartmoor. It then crosses Liberty Street and extends to Stadium Blvd. just south of Arbordale and then southwesterly to Maple Road. The Murray-Washington Drain varies in size from 66-inches in diameter at its junction with the Allen's Creek Drain to 24-inches in diameter at Maple Road.

The Eberwhite Drain was also constructed in 1927 from plans prepared by Menefee and Dodge, Engineers. This drain extends from its junction with the Allen's Creek Drain at First and Williams Streets westerly in Williams to Fourth Street and then across lots in a southwesterly direction to its termination at Ridgemor approximately 1200 feet south of Liberty Street. The drain branches to the south just west of Seventh Street and continues to just north of Pauline Blvd., where it heads east, ending at Arbordale as a 24-inch diameter storm sewer. This drain varies in size from 54-inch in diameter at its junction with the Allen's Creek Drain to 24-inches in diameter at its termination at Ridgemor and at Arbordale.

The West Park-Miller Drain was constructed in 1928 from plans prepared by Menefee and Dodge, Engineers. This drain extends from its junction with the Allen's Creek Drain at the foot of Ann Street into two branches. The north branch extends northwesterly through West Park to the intersection of Maple Ridge and Arborview Blvd. with a branch extending northerly in Red Oak and a branch extending westerly in Arborview Blvd. to Doty Street. The north branch varies in size from 66-inches in diameter at its junction with the Allen's Creek Drain to 24-inches in diameter at its termination point in Arbor View Blvd. at Doty Street. The south branch of this drain extends southwesterly in West Park to Seventh Street approximately 400 feet north of Huron Street and follows the natural drainage course westerly and parallel to Huron Street and Dexter Road to its termination point at Doty Street. This drain varies in size from 54-inches at its junction with the north branch of the West Park-Miller Drain to 42-inches in diameter at its termination in Doty Street. In 1950 the West Park-Miller south branch was extended from Doty Street as a 72-inch sewer and terminated as a 30-inch sewer at I-94 and Jackson Road. This extension was known as the West Park-Miller Fairgrounds Extension.

CLIMATOLOGY AND METEOROLOGY

The climate of Ann Arbor is generally mild as the area seldom experiences prolonged periods of hot, humid weather or extreme cold. Temperature variations during the year occur between 0^{o}F and 90^{o}F. A high of 105^{o}F was recorded on July 24, 1934 while a low of -21^{o}F was recorded on February 10, 1912. Average yearly precipitation is approximately 30.1 inches. May is the wettest month, averaging 3.25 inches, while February is the driest month, averaging 1.65 inches. The greatest daily precipitation, since 1940, of 4.74 inches fell on June 25, 1968. Average annual snowfall is approximately 29.5 inches. The heaviest single-day snowfall of 15.8 inches occurred December 1, 1974. Additional climatological and meteorological information is contained in Appendix A.

DRAINAGE AREA

The Allen's Creek drainage basin contains approximately 3500 acres. The drainage basin consists of the Allen's Creek Drain, which is the main trunk storm sewer discharging to the Huron River, and three major tributary storm sewers, West Park-Miller Drain, Murray-Washington Drain, and Eberwhite Drain. The location of the Allen's Creek drainage basin is shown in Figure 3. Also shown in Figure 3 are the drainage boundaries of the major tributaries. West Park-Miller Drain contains approximately 825 acres, Murray-Washington Drain contains approximately 725 acres and Eberwhite Drain contains approximately 275 acres.

The drainage areas were determined utilizing storm sewer plans and topographic maps. In some cases, storm sewers transport runoff to adjacent sub-drainage basins within the Allen's Creek basin and in other cases transport runoff to drainage basins outside of the Allen's Creek basin. The amount of flow diverted in this manner is not considered to be significant under large rain events, which are the events causing flooding problems. Therefore, drainage boundaries are based mainly on natural drainage divides rather than strictly on storm sewer routes.

TOPOGRAPHY AND DEVELOPMENT

The Allen's Creek drainage basin generally slopes toward the north-south centerline of the basin, which slopes to the north towards the Huron River. The drainage basin lies within the City limits of Ann Arbor. The topography is mild to steep with numerous valleys forming natural overland flow channels which conduct surface runoff to the Huron River.

Allen's Creek is the main trunk storm sewer and flows in a northerly direction along the Ann Arbor Railroad to the Huron River. The Allen's Creek overland slope ranges from 0.7% to 0.9%. West Park-Miller Drain encompasses the area between Jackson-Huron Street and Miller Road with an overland slope ranging from 1.5% to 7.0%. Murray-Washington Drain services the area between Jackson-Huron Street and Liberty Street and the area between I-94 and east of Stadium Blvd. The overland slope ranges from 1.5% to 5.0%. Eberwhite Drain encompasses the area from Liberty St. to Pauline Blvd. and from east of Stadium Blvd. to Seventh Street. The overland slope ranges from 1.5% to 5.0%. The area east of Allen's Creek, encompassing the University of Michigan main campus area, has an overland slope of approximately 2.0%. The area north of Huron Street, servicing the business district of Ann Arbor, has an overland slope of approximately 7.5%.

Recent topography mapping was studied to determine storage areas where surface runoff would pond up and not contribute to downstream flows. Significant storage areas were found to exist in the West Park-Miller and Murray-Washington drainage basins.

Development within the Allen's Creek drainage basin consists of mostly residential and commercial/business with some light industrial and parkland/undeveloped lands. The areas directly tributary to Allen's Creek contain the downtown business district, light industrial, residential areas, University of Michigan athletic facilities, golf course and Ann Arbor Pioneer High School. The West Park-Miller area consists of mainly single family residential areas with some commercial, multiple family residential and parkland. The Murray-Washington area consists of mostly residential areas, a substantial amount of commercial areas and some multiple family residential and parkland. The Eberwhite Drain area consists of primarily residential with some parkland. The area east of Allen's Creek Drain consists of residential areas, and the University of Michigan main campus, which consists of large parking lots, multiple residential units and parkland and commercial areas. The area north of Huron Street consists of residential areas, parkland and some commercial areas.

STORM SEWER CONDITION

General

The Allen's Creek Drain and Branches (48-inch diameter and larger) are considered to be in generally good condition and suitable for continued use with only minimal improvements. Three different internal inspections have been conducted by McNamee, Porter and Seeley over the past fourteen years. The results of the first inspection in the fall of 1968 were published in the September 1972 "Report - Allen's Creek Drain" for the Washtenaw County Drain Commission and City of Ann Arbor as general comments. The second inspection in the fall of 1974 provided a detailed inspection report identifying interval visual conditions related to the original construction drawings. The drains were inspected a third time in the spring of 1982 as part of this current work for the City of Ann Arbor.

This third inspection revealed very minor deterioration within the drain during the eight years elapsed since the 1974 inspection.

Information gathered during the previous inspections, and supported by the 1982 inspection, indicates that the drain is in need of minor structural improvements, repairs and some general cleaning and maintenance work to assure continued efficient performance. In addition, utility relocations and sanitary sewer cross-connection removals and relocations may be required.

The Sewer Inspection Notes (Supplement 1) are available as a supplement to this study and are also available for inspection at the Ann Arbor office of McNamee, Porter and Seeley, Consulting Engineers, and at the offices of the City of Ann Arbor Engineering Department. Copies of the original construction drawings marked to reflect the recommended improvements, repairs and maintenance work are available at the above referenced locations.

Allen's Creek Drain - Huron River To Stadium Blvd.

The internal inspections revealed that some internal improvements, repairs, cleaning and general maintenance work should be undertaken along the main Allen's Creek Drain. Joint repairs, invert replacement and patching of exposed reinforcing steel are the most significant improvements and repairs needed. Six potential sanitary sewer connections were noticed and should be investigated and eliminated if they are, in fact, sanitary sewers. Existing sediment, rock, gravel and other debris should be removed as part of general maintenance and cleaning.

Each item of work has been identified in Supplement 1 attached to this report.

Hill and Hoover Street Branches

Hill and Hoover Street Branches are in very good shape except for some joint repairs and invert improvements.

Murray-Washington Drain

Internal inspection revealed that the Murray-Washington Drain was in good condition but needs some general maintenance to sustain continued use. Crown cracks are present along the 66-inch and 60-inch precast concrete sewer pipe. The sections with severe cracks should be replaced immediately, while the less severe sections should be grouted. Other recommended improvements consist of joint repairs, grouting of exposed reinforcing and removal of debris.

Eberwhite Drain

Internal inspection revealed that the Eberwhite Drain was in generally good condition but requires some general improvement to remain reliable. The major work recommended involves grouting of exposed concrete reinforcing steel along the concrete pipe. A short section of sewer needs concrete reinforcement to insure structural stability. Other required improvements consist of joint repairs and removal of debris.

West Park-Miller Drain

Internal inspection of the West Park-Miller Drain revealed that major structural problems exist along both the north and south branches of the drain. Precast concrete pipe has cracked and is thought to be near collapse in several reaches, and therefore should be replaced. Joint repair will be a major effort in this drain. Other needed improvements involve grouting exposed reinforcing steel, removal of debris and manhole invert repairs. A possible sanitary sewer connection should be removed immediately.

West Park-Miller Drain has the most structural degradation of the Allen's Creek drainage basin and should be given first priority. Immediate steps should be taken to repair or replace the collapsed sewer sections.

EXISTING CONDITIONS

GENERAL

The Allen's Creek drainage basin lies in the major portions of the central and western areas of the City of Ann Arbor. Rainwater is collected in storm sewers which run in generally east-west, west-east directions to the Allen's Creek Drain storm sewer, which collects the storm water and carries it northward to the Huron River outlet. Storm water which is in excess of the storm sewer capacity flows overland along natural valleys, along roads and between existing structures.

The following sections detail the hydrologic and hydraulic determination and modeling of various frequency flooding events. The analyses include investigating the storm sewers and the overland flow path with results yielding floodplain delineations and water surface elevations.

STORM SEWER CAPACITIES

Profiles and sizes of existing storm sewers along the study reaches for Allen's Creek, West Park-Miller Drain, Murray-Washington Drain, and Eberwhite Drain were analyzed from existing as-built drawings to determine their capacities while flowing full and while flowing surcharged at or near the ground surface. Physical condition and obstructions, such as crossover sewers and water mains, were included in the analysis in order to determine their impact on the capacities.

Manning's formula was utilized to calculate the flow capacities. Roughness values were determined from field inspection of the storm sewers. Table 2 presents the storm sewer capacities.

Storm sewer capacities are used in subsequent sections in conjunction with the hydrologic investigation in order to determine overland flows for the 10 and 100 year flood frequencies.

Table 2

Storm Sewer Capacities

Location	Flow (cfs) Pipe Full	Flow (cfs) Pipe Surcharged	Manning's Roughness (n)
Allen's Creek Drain			
Outlet to N. of Miller	980	1025	.02
N. of Miller to W. Park-Miller Drain	785	840	.02
W. Park-Miller Dr. to Murray-Washington Dr.	785	855	.02
Murray-Washington Dr. to Washington St.	500	520	.02
Washington St. to Eberwhite Drain	450	500	.02
Eberwhite Drain to Liberty St.	325	360	.02
Liberty St. to Hill St.	335	345	.02
Hill St. to Hoover St.	315	335	.018
Hoover St. to Stadium Blvd.	185	195	.013
u/s Stadium Blvd. to Dam - open channel	150	195	.035
Bypass to D/S 48" dia. culvert	100	150	.013
48" diameter culvert	140	210	.013
53" x 83" culvert	300	450	.014
Dam to 66" dia. culvert - open channel	115	685	Dam Outlet
D/S headwall of 66" dia. to D/S S. Main	315	345	.014
D/S S. Main to u/s S. Main	200	225	.014
West Park-Miller Drain			
At Allen's Cr. to junc. of N & S Branches	220	255	.015
South Branch at junct. w/N. Branch to 7th	245	260	.015
Seventh to Arbana	150	160	.015
North Branch			
At junction to Seventh	260	265	.015
Seventh to Maple Ridge	220	230	.015
Murray-Washington Drain			
Allen's Creek to Murray	245	255	.02
Murray to Seventh	340	360	.015
Seventh to Buena Vista	270	275	.015
Buena Vista to 72" storm	240	245	.015
72" Storm to Dartmoor	350	360	.015
Dartmoor to Ivywood Dr.	255	265	.015
Ivywood Dr. to Stadium Blvd.	190	200	.012
Eberwhite Drain			
Allen's Creek to Liberty	135	145	.015
Liberty to Second	180	190	.015
Second to Fourth	155	160	.015
Fourth to Elder	155	160	.015

HYDROLOGY

Peak flows were developed at selected locations along Allen's Creek and the main tributaries of West Park-Miller Drain, Murray-Washington Drain and Eberwhite Drain for the 10 and 100 year frequencies in order to determine water surface elevations and extent of flooding. Discharge hydrographs were computed at each location, adjusted to take into account storm sewer capacities, routed via channel routings and/or storage routings in order to account for storage volumes and timing, and were added to subsequent downstream hydrographs. Appendix B contains details of the methodologies and techniques utilized to conduct this portion of the study.

Hydrologic analyses were carried out to establish the peak discharge-frequency relationships for floods of the selected recurrence intervals for each watercourse.

Peak discharge values for the Allen's Creek Drain, West Park-Miller Drain, Murray-Washington Drain and Eberwhite Drain were obtained from an analysis utilizing Braters' Unit Hydrograph Method (Brater, E.F. and J.D. Sherill. "Rainfall-Runoff Relations on Urban and Rural Areas," EPA-670-12-75-46, May 1975.), and a routing of the resultant hydrographs using the HEC-1 computer program. (U.S. Army Corps of Engineers, Hydrologic Engineering Center. "HEC-1 Flood Hydrograph Package Users Manual," September 1981; and "HEC-1 Flood Hydrograph Package Programmers Manual - Draft," October 1981.) Braters' method consists of developing flood hydrographs of various frequencies based on such factors as rainfall intensity duration, infiltration capacity, watershed area, and population density. The HEC-1 computer program is used for both stream and lake routing of flood hydrographs. The procedures used do not account for dynamic storage effects, and are, therefore, somewhat conservative. However, as alternatives to reduce flooding levels are implemented, dynamic storage effects would be essentially eliminated.

Peak discharges for the 10 and 100-year floods of each flooding source are shown in Table 3, in addition to relevant drainage area and location information.

Table 3

Peak Discharges

Allen's Creek	Cross section Location	Drainage Area (sq.mi.)	Overland Flow/Total Flow (cfs) Frequency (yrs.) 10		100	
Huron River	9	5.5	1050	(2075)	2355	(3380)
Kingsley	50	5.0	985	(1825)	2100	(2940)
West Park-Miller Drain	61	3.7	610	(1450)	1470	(2310)
Murray-Washington Drain	69	2.6	510	(1030)	1240	(1760)
Eberwhite Drain	85	2.2	415	(775)	920	(1280)
Williams	90	1.8	200	(545)	690	(1035)
Hill Street	115	1.6	125	(460)	565	(900)
Hoover	130	1.0	85	(280)	335	(535)
Upstream Stadium Blvd. (d/s end of open channel)	155	0.7	260	(260)	480	(480)
Downstream of Main Street (u/s end of open channel)	170	0.5	0	(260)	135	(480)
Upstream South Main	200	0.2	0	(100)	0	(180)
Upstream End	—					
Eberwhite Drain						
Allen's Creek	545	0.4	115	(265)	320	(470)
Upstream End	—					
Murray-Washington Drain						
Allen's Creek	695	1.1	175	(430)	325	(580)
Crest	740	0.8	0	(245)	0	(255)
Virginia and Bemidji	750	0.8	0	(250)	0	(255)
Liberty	770	0.5	40	(295)	270	(525)
Dartmoor	790	0.3	20	(220)	195	(395)
Upstream End	-					
West-Park Miller Drain						
Allen's Creek	950	1.3	165	(425)	390	(650)
Junction of N/S Branches	965	1.2				

(continued)

Table 3 (continued)

Allen's Creek	Cross section Location	Drainage Area (sq.mi.)	Overland Flow/Total Flow (cfs) Frequency (yrs.) 10		100	
South Branch						
Junction with N. Branch	823	0.7	0	(155)	145	(305)
Ravena	865	0.6	0	(155)	295	(455)
Upstream End	—					
North Branch						
Junction with S. Branch	965	0.5	0	(260)	205	(465)
Upstream End	—					

COMPARISON WITH PREVIOUSLY CALCULATED FLOWS

The report entitled "Allen's Creek Drain - Analysis and Preliminary Alternatives for Relief: Interim Report Prepared for the Washtenaw County Drain Commission" (McNamee, Porter and Seeley, Sept. 1972), presented estimates of flow for the 10 year flood at the downstream end of Allen's Creek Drain, West Park-Miller Drain, Murray-Washington Drain and Eberwhite Drain. Peak discharges calculated in the current study are substantially lower than the previously calculated flows due to several factors. Drainage areas have been revised in this current study based on updated storm sewer maps and latest topography information. Population densities have been determined accurately from the 1980 United States census tracts, whereas the 1970 total population data for the City of Ann Arbor was utilized for the 1974 study. The major difference was due to the fact that the 1974 study calculated flows assuming the storm runoff would all be carried by storm sewers. Therefore, overland channel storage and retention, timing of each hydrograph as it travelled overland, and the manner in which hydrographs combined downstream, were not taken into account. It was assumed that storm runoff quantities added simultaneously at all locations as storm sewers would convey the runoff quickly and efficiently with little time delay between the peak discharges. However, the current study proposes overland flow relief. Therefore, overland storage, retention and hydrograph timing cause peak discharges not to add directly, thereby reducing downstream peak discharges.

HYDRAULICS

The 10 and 100 year frequency floods on the Allen's Creek basin create overland flows as existing storm sewer capacities are not adequate to carry the full flow. Overland flows occur in natural channels/swales and in streets. The natural channels/swales very often are located between private homes, businesses and other structures. When this occurs, the overland flow becomes "flooding" as the water elevation rises above the base of numerous structures.

Overland flow paths in the Allen's Creek basin act as natural stream channels during flooding events but are essentially dry the majority of the time. Normally, the flow paths are wide and bowl shaped without a well defined stream channel. In some cases, the flow path becomes narrow due to structures obstructing the flow. In other cases, the flow path becomes very wide due to a wide extent of flat land.

Analyses of the hydraulic characteristics of the flooding sources studied in detail in the Allen's Creek System were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along each of the flood sources. Overland flows were used to determine flooding elevations.

Cross sections for the backwater analyses of Allen's Creek, West Park-Miller Drain, Murray-Washington Drain and Eberwhite Drain were made from topographic mapping obtained photogrammetrically from aerial photographs flown in the spring of 1982 (Abrams Aerial Survey Corp., Lansing, Michigan. "Aerial Photographs of the City of Ann Arbor"). Topographic mapping was prepared at a scale of 1-inch equal to 100 feet with 1 foot elevation contours.

Channel roughness factors (Manning's "n") used in the hydraulic computations were based on field observations of the streams and flood plain areas. Roughness values for the overland flow channels range from .07 to .10.

Water surface elevations of floods of the selected recurrence intervals were computed through use of the HEC-2 step-backwater computer program (U.S. Army Corps of Engineers. "HEC-2 Water-Surface Profile," User's Manual, February 1972; Application of the HEC-2 Bridge Routines, June 1974; and HEC Training Document 5 Floodway Determination Using Computer Program HEC-2, Vernon R. Booner, May 1974.). Flood profiles were drawn showing computed water surface elevations for floods of the selected recurrence intervals. The starting water surface elevation for Allen's Creek was obtained from the Ann Arbor Flood Insurance Study. The Huron River water surface elevations at the junction with Allen's Creek were used. Starting water surface elevations for West Park-Miller Drain and Murray-Washington Drain were obtained from Allen's Creek hydraulics. The starting water surface elevation for Eberwhite Drain was calculated using the slope-area method.

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures or physical structures remain unobstructed, operate properly, and do not fail.

The hydraulic losses that will occur due to the addition of bridges and other structures on the overland flow channels because of population growth or shifts have not been considered.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles and the Flood Plain Delineation Maps (Supplement 2). The 10 and 100 year flood elevations are plotted on the profiles and the floodplain maps. It was found that channel storage along Allen's Creek is a significant factor in reducing peak flow rates under existing conditions. If this storage is removed by further development or filling in of the floodplain, the flood elevations would increase. The profiles and floodplain for the reduced storage conditions are also shown and are referred to as Existing Conditions with Normal Floodplain Encroachment. Figure 4 (back pocket) shows the flooding extent for the 10 year flood at a scale of 1-inch equal to 1200 feet for the reduced storage condition.

COMPARISON WITH HISTORICAL DATA

Results of the hydraulic analysis were compared to observed conditions during the 1968 flood. The physical survey conducted following the 1968 flood resulted in determining spot water surface elevations and flooding extent along the Allen's Creek Drain from the Huron River to Hoover Avenue. In addition, flooding which occurred on the tributaries was noted. It is estimated that the 1968 flood was approximately a 50 year to 100 year frequency event. The hydraulic modeling results agreed favorably with the 1968 flooding extent. Preliminary results were presented to the citizens of the City of Ann Arbor at two public meetings (refer to Appendix C) during the course of the study. Citizens reviewed the results and were in reasonable agreement with flooding extent based on previous observations during past rain and flood events, particularly the large flood in June 1968. The 100 year frequency water surface elevations are slightly higher than water surface elevations recalled by citizens during the 1968 flood.

The differences noted tended to be small, except near the downstream end of Allen's Creek in the vicinity of Depot Street, Fifth Avenue and Summit Street. The previous delineation of the extent of flooding, based on high water data, showed water flowing over the top of the railroad at the downstream end. The model also indicated that water would flow over the railroad. Based on the detailed topographic information obtained for this study, residents were also able to compare the depth of flooding as well as the extent. Interviews in September 1982 with the residents of the area indicated that the water level did not rise quite high enough to flow over the top of the railroad. Therefore, a further analysis was made of the model used and the underlying assumptions made. The physical factors, hydraulics and hydrology were re-examined and evaluated.

Physically, if the railroad elevation is incorrect, if the elevation has been altered since 1968, or if additional outlets under the railroad exist, the results of the model could be incorrect. A site visit was made and it appears that the railroad elevation is correct and that no additional outlets exist. One resident indicated that no significant changes have been made to the railroad since 1968. Therefore, it does not appear that the physical factors are at fault.

The hydraulics indicate that the maximum water surface elevation would be experienced for a given flow rate. The peak 50 year flow rate was estimated to be 1865 cfs at this location. The maximum flow rate which could be handled by the catch basins at the outlet would be on the order of 100 cfs or less. Therefore, the hydraulics would indicate that flow would occur over the top of the railroad. Since the estimated peak flow is so much greater than the calculated outflow, it does not appear that the hydraulics would have a significant effect.

The two most important factors influencing the hydrology at the downstream end of the system are the storage available and the infiltration capacity. Since the results throughout most of the reaches of study agreed well with historical data, it appears that the infiltration capacity is not a major factor in the discrepancy. Storage in the existing system was modeled using reservoir routing techniques in the HEC-1 computer model. It was estimated that there is 120 acre-feet of storage available in the natural storage areas along the drains. The individual locations and volumes are shown in Table 4. Additionally, a similar routing technique was used to account for storage in the open channels. The area near the outlet was recognized as a storage area, but it was expected that the volume available would be insufficient to significantly reduce the peak outflow. The estimated volume of overland flow for the various events analyzed is also shown in Table 4.

Based on the concerns raised, a further investigation was made of the channel storage available. The figures are shown in Table 4. It can be seen that the channel and natural open area storage is quite significant compared to the total overland flow volume. The experienced results in 1968 appear to be consistent with this finding. Although a dynamic routing model would be required to completely model the channel storage condition, an approximate technique was used for estimating the profiles and floodplain. A ratio was determined by subtracting the channel storage volume, translated to inches of surface runoff on the tributary area, for the total surface runoff for the respective frequency storm, over the total surface runoff for the respective frequency storm:

$$\frac{\text{Total Surface Runoff} - \text{Channel Storage on Drainage Area}}{\text{Total Surface Runoff}}$$

A revised flow value was then computed by multiplying the original total flow (Table 3) by the ratio and then subtracting the appropriate storm sewer capacity. Table 5 contains the revised peak flows as well as previous peak flows (Table 3) for comparison purposes. The 10-year and 100-year profiles and the 10-year floodplain delineation allowing for the floodplain storage are shown in Supplement 2 and are referred to as Existing Conditions with Floodplain Reserved for Storage.

It is important to note that as the flooding is reduced by any alternative chosen, except "no action," the channel storage will be reduced, thereby making the normal encroachment flows more accurate.

Table 4

Existing Storage Volumes

Natural Storage Areas (acre-feet)		
Liberty Street just east of Dartmoor	50	
Virginia at Bemidji	14	
North of Huron from Wildwood to Arbana	41	
	119	
Channel Storage (acre-feet)		
	10 year event	100 year event
Murray-Washington Drain	16	24
Eberwhite Drain	5	10
West Park-Miller Drain	14	31
Allen's Creek	64	131
Total	99	196
Estimated Total Overland Flow Volume (acre-feet)		
	132	429

Table 5

Channel Storage Impact on Peak Discharges

Location	Overland Flow (cfs) - Existing Condition Frequency (yrs) 10: With Normal Floodplain Encroachment	10: With Floodplain Reserved for Storage	100: With Normal Floodplain Encroachment	100: With Floodplain Reserved for Storage
Huron River	1050	0	2355	1410
Railroad upstream of Huron River	1050	595	2355	1410
Kingsley	985	655	2100	1425
West Park-Miller Drain Confluence	610	350	1470	940
Murray-Washington Drain Confluence	510	315	1240	800
Eberwhite Drain Confluence	415	285	920	615
Williams	200	95	690	420
Hill Street	125	60	565	385
Hoover	85	40	335	240
Stadium Blvd.				

The results indicate close agreement with conditions experienced in the 1968 flood.

The City of Ann Arbor Flood Insurance Study analyzed Allen's Creek from the Huron River upstream to Hoover Avenue. The current hydraulic modeling agrees reasonably well with the Flood Insurance Study results. Discrepancies between the two studies are related to the detailed 1-inch equal to 100-ft., 1 foot contour topography utilized in the current study in addition to the detailed cross section data utilized in the hydraulic modeling.

ALTERNATIVES FORMULATION

There are three general categories of alternatives for handling flooding problems: a) reduction of peak flows, b) reduction in flood stages, and c) reduction in damages.

Reduction in flow can be accomplished by increasing the amount of rainfall infiltration into the soil or by storing part of the runoff in such a way that the peak outflow rate is lower than the peak inflow rate. The soils in the Allen's Creek basin consist mainly of loam and sandy loam, which are generally not suitable for providing recharge ponds to increase infiltration into the soil. The City of Ann Arbor currently has an ordinance which requires that retention be provided at new developments such that the peak flow rate is not increased over predevelopment conditions. Also, there are several locations where storm flows are naturally retained under existing conditions. A few open areas which could be used to retain storm flows are available.

Table 6 lists the sites considered for retention development. These sites are in the low-lying areas along Allen's Creek or its tributaries and are at locations where a significant amount of drainage area is tributary. Table 6 also includes the volume of overland flow which would be tributary to these basins for the 10 and 100 year events, as well as the retention volume which could be developed at each of the sites. It can be seen that the retention volume available is much smaller than the overland flow volume. Therefore, retention of the stormwater in new basins would not produce a large reduction in the amount of flooding currently being experienced. Retention at numerous locations throughout the basin by providing catch basin restrictors is not feasible in Ann Arbor due to the hilly terrain. Water would tend to flow overland rather than ponding around the catch basin.

Table 6

Retention Areas

		Storage (acre-feet)	100 Year Overland Flow Volume (acre-feet)
1)	Athletic field south of Liberty and east of Seventh	3	26
2)	Slauson School athletic field (to offset storage in residential area at Virginia and Bemidji)	14	13
3)	Elbel athletic field	13	188

Reduction in flood stages can be accomplished by increasing the efficiency of the flow channel. Under existing conditions, the runoff will flow in the storm sewer until its' capacity is exceeded, then it will flow overland. The efficiency of the storm sewer as well as the overland flow path can be increased. As described more fully in the section on the condition of the storm sewer, the sewer is in poor condition in several areas. Rehabilitation of these reaches can increase the efficiency of the sewer and are recommended to be included in the chosen alternative. Also, a parallel storm sewer can be provided to increase the capacity of the sewer system. Similarly, the overland flow channel efficiency can be improved. Obstructions and restrictions in the flow path can be removed. For example, a culvert can be placed under a road that is high compared to the surrounding land and solid fences can be replaced with more open ones. These options are viable for the Allen's Creek basin and are considered further.

Alternatives that reduce damages are often called nonstructural alternatives. Various options, such as removal of structures in flood prone areas, flood proofing, and procurement of flood insurance can help to reduce the economic losses due to flooding. The City of Ann Arbor is currently participating in the National Flood Insurance Program. Therefore, insurance subsidized by the federal government is available to anyone in the City. Floodplain regulations are also in effect to restrict new development from occurring in the flood plain. These options were also considered viable for the Allen's Creek basin and have been developed further.

ALTERNATIVES DEVELOPMENT

Four alternatives to alleviate flooding problems in the Allen's Creek system were deemed viable and are presented in this section. Each alternative includes the recommended existing storm sewer rehabilitation program involving repair and cleaning of existing storm sewers at an estimated project cost of $1,100,000.

Alternative 1: Storm Sewer Relief

For this alternative, the existing storm sewers would be rehabilitated to maximize their capacities. In addition, relief sewers would be constructed. The relief would be sized such that a 10 year flow could be handled by the sewers. The relief sewers would be located adjacent to the existing sewers wherever room is available. Where there is not sufficient area available, the sewer would be constructed in adjacent streets. In some sections, mainly along the tributary branches, the adjacent streets are much higher than the existing sewers and would require excessively deep sewers at a corresponding increased cost of construction.

Therefore, it will be necessary to acquire a few houses in order to provide the necessary easement and construction right-of-way for the relief sewers. The sizes of the proposed relief sewers and the suggested routes are shown in Figure 5 (back pocket). The current estimated project cost of this alternative is approximately $18,000,000, which includes legal, engineering and administrative fees.

The advantages of this alternative are that flooding due to flows up to a 10 year frequency would be eliminated. However, the cost of construction is higher than the other alternatives.

Alternative 2: Overland Flow Channels

In order to transport the storm flows in excess of the pipe capacity, an overland flow channel would be provided in this alternative. The existing pipes would be rehabilitated to minimize the amount of overland flow. To remain unobtrusive, the channel would be a grassy swale with four horizontal to one vertical side slopes and a 10 year depth of less than two feet, where it could be accommodated. Steep side slopes or a slightly larger depth may be needed in some cases to avoid existing structures. Some structures would have to be removed where there is no room for an overland channel. The swale would be dug down in the low areas as shown in Figure 6. Supplement 2, Floodplain Delineation Maps and Flood Profiles, contains drawings of the 10 and 100 year overland flow channel alternative profiles and delineations. Therefore, the 10 year hydraulic grade line would be below the existing ground in most areas and no overland flow would be present next to existing structures for the 10 year event. At road crossings, either the road would have to be lowered or a culvert below the road provided. Because of the low depths of flow, culverts could not be very tall and therefore, would have to be relatively wide to provide open waterway required. Four culverts are recommended for this alternative: on Allen's Creek at the railroad crossing near the Huron River, at Huron Street, at Liberty Street, and on Eberwhite Drain at Second Street.

The intent of this alternative is to recognize that there is a low-lying area where flood flows will occur and to accommodate these flood flows with minimial disruption to the area. Therefore, it would be desirable for the overland channel to usually remain dry and to serve multiple purposes. Where there is a wider floodplain and no further development is anticipated, or where some flooding can be tolerated, the swale would not have to be provided. The effect on upstream areas would have to be evaluated, but due to the relatively steep slopes along the flow path, the effects would usually be quite localized.

Reducing the extent of flooding would also reduce the channel storage. To partially offset this effect, a portion of the flow in excess of the existing pipe capacity would be stored in retention basins. The basins would be located on existing open lands shown in Table 6. Since the open lands are currently recreational areas, a dual use would be maintained. The basins would only be used when the pipe capacity is exceeded.

The current estimated cost of construction of this alternative is $6,200,000, including existing storm sewer rehabilitation, channelization, culverts and retention ponds. The total project cost, including legal, engineering and administrative fees, is estimated to be $7,700,000.

The advantage of this alternative is that flooding due to the 10 year event would be eliminated. The 100 year flooding would also be significantly reduced. Implementation of this alternative may be quite difficult since much of the work would be across back lots and around existing structures.

Alternative 3: Nonstructural Alternative

The emphasis of this alternative is to maximize the existing system and to adapt to the flooding conditions rather than attempting to change it. The existing sewer system would be rehabilitated to maximize its capacity. The residents would be informed of the flooding problems and encouraged to purchase federally subsidized flood insurance. The City's engineering department would provide data on existing flooding potential and could help the residents understand the factors which could reduce the potential for damages such as keeping valuables above flood levels, and not further obstructing the flow path. The estimated project cost of rehabilitating the existing storm sewers is $1,100,000, in addition to an estimated cost of $50,000 for providing residents with nonstructural flood prevention techniques.

The advantage of this alternative is the low capital expenditure required and the minimal disruption due to construction. However, residents in low-lying areas would continue to experience flooding problems.

Alternative 4: Combination of Nonstructural Measures and Overland Flow Channels

For this alternative, all the measures noted in Alternative 3, the Nonstructural Alternative, and the storm sewer rehabilitation would be implemented. In addition, Alternative 2, Overland Flow Alternative, is recommended to be implemented in those reaches which are determined to be most affected by flooding. The design of work in the selected reaches should be such that the flooding in surrounding areas is not significantly worsened. Table 7 lists the number of structures affected along different sections of Allen's Creek and its tributaries for the 10 year frequency flood assuming existing conditions with the floodplain reserved for storage; i.e., results consistent with the 1968 flooding event. The estimated project cost is $5,300,000.

The advantage of this alternative is that it is responsive to areas where the need is greatest and keeps disruptions to a minimum in other areas.

Table 7

Allen's Creek Structure Flooding - 10 Year Frequency

Reach	0 - 1/2	1/2 - 1	1-2	2-3	3-4	4-5	5-6	Total Struc.
	Depth of Flooding (ft.)							
Allen's Creek Drain								
Huron River to Main	9	8	11	12	-	-	-	40
Main to Felch	4	1	4	5	1	-	-	15
Felch to Miller	3	4	15	8	-	-	-	30
Miller to Huron St. (W. boundary Chapin)	4	7	10	3	-	-	-	24
Huron St. to Liberty (W. boundary 3rd)	-	1	5	2	2	-	-	10
Liberty to W. Jefferson (W. boundary 2nd)	2	2	1	3	1	-	-	9
W. Jefferson to W. Madison	6	4	3	1	-	-	-	13
W. Madison to Hill	4	8	1	-	-	-	-	13
Hill to Hoover	14	9	-	-	-	-	-	23
Hoover to McKinley	2	20	24	-	-	-	-	46
McKinley to Stadium	-	-	-	-	-	-	-	0
Stadium to Main	1	-	-	-	-	-	-	1
Subtotal	49	64	74	33	4	0	0	224
West-Park Miller Drain								
Chapin to N/S Junction	4	2	-	-	-	-	-	6
Subtotal	4	2	0	0	0	0	0	6
Murray-Washington Drain								
3rd to Murray	9	22	2	2	-	-	-	35
Murray to 7th	-	11	5	3	-	-	-	19
7th to Crest	-	-	4	-	-	-	-	4
Crest to Liberty	-	-	-	-	-	-	-	0
Liberty to Stadium	15	7	1	-	-	-	-	23
Subtotal	24	40	12	5	0	0	0	81
Eberwhite Drain								
2nd to 4th	4	5	2	7	1	2	-	21
4th to 7th	2	11	2	1	-	-	-	16
7th to Lutz	4	3	3	2	-	-	-	12
Subtotal	10	19	7	10	1	2	0	49
Total Structures Flooded	87	125	93	48	5	2	0	360

RECOMMENDED IMPROVEMENTS AND COST ESTIMATE

The recommended alternative is Alternative 4. Nonstructural measures and the storm sewer rehabilitation are included. In addition, channelization and culvert improvements for various reaches are included. The reaches recommended for such improvements were chosen based on severity of flooding with respect to structures flooded above a depth of two feet for the 10-year flood. These improvements are recommended to be implemented in the near future in order to reduce future severe flooding in these critical areas. Table 8 contains the recommended prioritized improvements with associated cost estimates. Recommended improvements are shown in Figure 6 in conjunction with the complete overland flow channel recommendations.

Future development in the Allen's Creek system is very likely to occur. It is recommended that retention basins be constructed for all new developments in order to restrict storm runoff to the amount it would be under undeveloped conditions.

Table 8
Recommended Improvements with Cost Estimate

	Estimated Cost
Nonstructural Improvements	$ 0
Legal, Engineering and Administrative	50,000
Subtotal Project Cost	$ 50,000
Storm Sewer Rehabilitation	
(Correct structural problems, joint repairs, pipe wall and invert protection, removal of debris)	
Allen's Creek Drain	$ 370,000
Legal, Engineering and Administrative	110,000
Subtotal	$ 480,000
Hill and Hoover Streets	$ 15,000
Legal, Engineering and Administrative	5,000
Subtotal	$ 20,000

(continued)

Table 8 (continued)

	Estimated Cost
Storm Sewer Rehabilitation (continued) (Correct structural problems, joint repairs, pipe walls and invert protection, removal of debris)	
West Park-Miller Drain	$ 77,000
Legal, Engineering and Administrative	23,000
Subtotal	$ 100,000
Murray-Washington Drain	$ 63,000
Legal, Engineering and Administrative	17,000
Subtotal	$ 80,000
Eberwhite Drain	$ 64,000
Legal, Engineering and Administrative	16,000
Subtotal	$ 80,000
Utility Relocations	$ 75,000
Sanitary Sewer Cross Connection Control	225,000
Legal, Engineering and Administrative	40,000
Subtotal	$ 340,000
TOTAL PROJECT COST	$ 1,100,000
Channelization, Culvert, Retention Basin Improvements	
Allen's Creek Reaches	
Huron River to Main St.	
Channel	$ 230,885
Culvert at RR (6'x45' box - 300' long)	766,000
Legal, Engineering and Administrative	249,115
Subtotal	$ 1,246,000
Main Street to Felch St.	
Channel	$ 126,875
Legal, Engineering and Administrative	31,125
Subtotal	$ 158,000
Felch Street to Miller Street	
Channel	$ 303,070
Legal, Engineering and Administrative	75,930
Subtotal	$ 379,000

(continued)

Table 8 (continued)

	Estimated Cost
Allen's Creek Reaches (continued)	
Miller St. to Huron St. (including West Park-Miller Drain to Upstream of Chapin)	
Channel	$ 410,165
Legal, Engineering and Administrative	102,835
Subtotal	$ 513,000
Huron Street to Liberty St. (including Murray-Washington Drain to 3rd St.)	
Channel	$ 423,735
Culvert at Huron St. (2.5'x60' box - 150' long)	284,000
Legal, Engineering and Administrative	177,265
Subtotal	$ 885,000
Liberty St. to W. Jefferson (including Eberwhite Drain to 2nd Street)	
Channel	$ 75,310
Culvert at Liberty St. (3'x35' box - 250' long)	386,000
Legal, Engineering and Administrative	115,690
Subtotal	$ 577,000
Eberwhite Drain	
2nd Street to 4th Street	
Channel	$ 196,675
Culvert at 2nd Street (2.5'x12' box - 200' long)	94,000
Legal, Engineering and Administrative	72,325
Subtotal	$ 363,000
Subtotal Project Cost	$ 4,121,000
TOTAL PROJECT COST	$ 5,271,000
(Approximately)	$ 5,300,000

APPENDIX A

Climatological Summary

U. S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
ENVIRONMENTAL DATA SERVICE
IN COOPERATION WITH MICHIGAN WEATHER SERVICE

CLIMATOGRAPHY OF THE UNITED STATES NO. 20 - 20

LATITUDE 42° 17'
LONGITUDE 83° 44'
ELEV. (GROUND) 871 Feet

CLIMATOLOGICAL SUMMARY

Revised December 1971

STATION ANN ARBOR, MICHIGAN
WASHTENAW COUNTY

MEANS AND EXTREMES FOR PERIOD 1940-1969

Month	Temperature (°F) Means: Daily maximum	Daily minimum	Monthly	Extremes: Record highest	Year	Record lowest	Year	Mean degree days **	Precipitation Totals (Inches): Mean	Greatest daily	Year	Snow, Ice Pellets: Mean	Maximum monthly	Year	Greatest daily	Year	Mean number of days: Precip. .10 inch or more	Temperatures Max.: 90° and above	Max.: 32° and below	Min.: 32° and below	Min.: 0° and below	Month
(a)	30	30	30	30		30		30	30	30		30	30		30		30	30	30	30	30	
JANUARY	31.8	17.8	24.8	72	1950	-14	1963	1246	1.81	1.59	1967	7.3	23.4	1943	14.0	1967	5	0	16	29	2	JANUARY
FEBRUARY	34.2	19.1	26.7	66	1944	- 9	1951	1082	1.65	2.25	1954	6.6	17.0	1962	6.5	1965	4	0	12	27	1	FEBRUARY
MARCH	43.8	26.4	35.1	78	1946+	- 2	1943	927	2.31	2.35	1954	5.4	17.0	1964	8.0	1968+	6	0	5	23	*	MARCH
APRIL	58.5	37.8	48.2	85	1960+	13	1954	504	3.21	3.70	1947	1.1	6.0	1961	4.0	1947	7	0	*	9	0	APRIL
MAY	69.7	48.1	58.9	92	1962	27	1966+	236	3.25	2.59	1943	.1	1.6	1940	1.2	1940	7	*	0	1	0	MAY
JUNE	79.8	58.2	69.0	99	1956	38	1963	54	3.10	4.74	1968	0	0		0		7	4	0	0	0	JUNE
JULY	83.4	62.0	72.7	102	1966	45	1960	6	2.91	2.85	1951	0	0		0		5	4	0	0	0	JULY
AUGUST	81.5	60.5	71.0	99	1955	40	1965	19	2.78	~~2.40~~ 3.13	~~1947~~ 75	0	0		0		6	4	0	0	0	AUGUST
SEPTEMBER	74.4	53.4	63.9	98	1960+	29	1961	99	2.19	~~1.58~~	~~1962~~ 71	0	0		0		5	1	0	*	0	SEPTEMBER
OCTOBER	63.7	43.9	53.8	89	1963+	23	1969	360	2.49	3.29	1954	T	1.2	1943	1.0	1943	5	0	0	3	0	OCTOBER
NOVEMBER	47.7	32.7	40.2	78	1950	1	1950	744	2.25	1.71	1955	3.0	16.0	1966	7.5	1966	6	0	2	16	0	NOVEMBER
DECEMBER	35.4	22.4	28.9	~~65~~ 64	~~1966~~ 1975	-12	1960	1119	2.15	2.15	1967	6.0	~~24.3~~ 34.0	~~1951~~ 1974	~~6.0~~ 15.8	~~1966~~ 1974	5	0	12	27	*	DECEMBER
Year	58.7	40.2	49.5	102	July 1966	-14	Jan. 1963	6396	30.10	4.74	June 1968	29.5	~~24.3~~ 34.3	~~Dec. 1951~~ JAN 1978	~~14.0~~ 15.8	~~Jan. 1967~~ DEC 1974	68	13	47	135	3	Year

(a) Average length of record, years. 3.94

T Trace, an amount too small to measure.

** Base 65°F (H. C. S. Thom, Monthly Weather Review, January 1954)

\+ Also on earlier dates, months, or years.

* Less than one half.

CLIMATE OF ANN ARBOR, MICHIGAN

Ann Arbor, in east central Washtenaw County and in the Southwest Lower Climatic Division, is 30 miles west of Detroit and 40 miles north of the Ohio border. The Huron River flows southeastward through the city on its journey to Lake Erie. The surrounding terrain is partially wooded and varies from gently undulating to moderately rolling hills. Soils range from sandy to silt loams. The major agricultural activities include corn, wheat, and oats.

Due to the inland location in southeast Michigan, the Great Lakes' influence on Ann Arbor's climate is minimized. The most noticeable influence is the increased cloudiness which moderates minimum temperatures during cold air outbreaks in the late fall and early winter months. The continental character of Ann Arbor's climate is reflected by the larger daily, seasonal, and annual temperature changes experienced at Ann Arbor when compared with stations nearer the Great Lakes and at a similar latitude.

Because the day-to-day weather is controlled largely by the movement of pressure systems across the nation, Ann Arbor seldom experiences prolonged periods of either hot, humid weather in the summer or extreme cold during the winter. Long-term wind and humidity records are not available from Ann Arbor, but these data should be similar to the values observed at Detroit. The prevailing wind at Detroit is southwesterly averaging about 10 mph. The strongest one-minute wind speed, 77 mph, occurred July 1960. The average 1 pm relative humidity varies from 51% in May and July to 70% in December. The percent of possible sunshine varies from 70% in July to 32% in January and December and annually averages 54%.

Temperature data available for Ann Arbor show the following extremes: a high of 105°F on July 24, 1934, and a low of 21°F below zero on February 10, 1912; the warmest monthly mean temperature, 77.6°F was recorded July 1955, while the coldest was January 1918 with 11.4°F. Summers are dominated by moderately warm temperatures with an average of 13 days exceeding the 90°F mark. Between 1940-1969, the thermometer reach 100°F, or higher, 2 times on July 2 and 3, 1966. During the same period, 7 years failed to record a below zero temperature. The lake influence is reflected in the milder minimum temperatures. On an average, 81% of the minimum temperatures from November through March, are 32°F, or below, but only 3 days per year will experience below zero temperatures.

Precipitation is well distributed throughout the year with the crop season, May-October, receiving an average of 16.72 inches or 56% of the average annual total. May, with 3.25 inches, is the wettest month, while February with a 1.65-inch average, is the driest month. Summer precipitation is mainly in the form of afternoon showers and thundershowers. Annually, thunderstorms will occur on an average of 36 days. The greatest monthly precipitation total on record occurred in July 1902 with 10.70 inches. August 1894 was the driest month when only a trace of precipitation was observed. The greatest daily total, 4.74 inches, fell on June 25, 1968.

GREATEST ANNUAL PRECIP. 41.55 1902
LEAST " " 16.84 1963

Evaporation from the class "A" pan during the crop season averages about 35.1 inches for the Ann Arbor area based on data taken at Dearborn. Because potential moisture evaporation is more than double the average precipitation during the crop season, soil moisture replenishment during the fall and winter months plays an important role in the success of agriculture for this area. While drought may be periodically experienced, only 10% of the time will drought conditions reach extreme severity as indicated by the Palmer Drought Index.

The average annual snowfall for Ann Arbor is 29.5 inches. The heaviest single-day snowfall, ~~14.0~~ 15.8 inches, occurred ~~January 27, 1967~~. 12-1-74 The average date for Ann Arbor's first 1-inch snowdepth is November 26; first 3-inch snowdepth, December 18; and first 6-inch snowdepth, January 16. Ann Arbor averages 55 days per season with 1 inch or more of snow on the ground, but this will vary greatly from season to season. The greatest snowdepth on record, ~~19~~ 20" inches, was recorded ~~January 27, 1967~~ 1-26-78. Snowfall during the 1965-66 season totaled only 6.5 inches while the greatest total, ~~72.0~~ 73.5 inches, was recorded during the ~~1966-67~~ season. 1977-78

The average date of the last freezing temperature in the spring is May 2, while the average date of the first freezing temperature in the fall is October 17. The freeze-free period, or growing season, averages 168 days annually.

Michigan is located on the northeast fringe of the Midwest tornado belt. The lower frequency of tornadoes occurring in Michigan may be, in part, the result of the colder water of Lake Michigan during the spring months, a prime period of tornado activity. Michigan has averaged 10 tornadoes each year since 1950. Since 1900, only 10 tornadoes are known to have touched down in Washtenaw County.

Degree day data is provided as an index of heating requirements for buildings. The average for May is 236 and for April, 504 degree days. This indicates that twice as much fuel will be required for heat in April as in May. "Degree Days" for a single day are obtained by subtracting the mean temperature from 65°F. When the mean temperature is 65°F, or higher, the need for heat is considered slight or none.

Probability of First Occurrence of 1-, 3-, and 6-inch Snowdepth By A Given Date:

Snowdepth	10%	50%	90%
1"	Nov 7	Nov 26	Dec 15
3"	Nov 13	Dec 18	Jan 26
6"	Dec 3	Jan 16	*

*Less than 90% probability of observing a 6-inch snowdepth.

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Average Temperature (°F)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Ann'l
1940	18.1	26.3	28.4	43.0	56.5	68.6	73.6	70.3	63.0	52.5	37.2	31.6	47.4
1941	25.9	25.1	30.4	52.8	62.1	69.2	73.6	70.9	66.7	54.8	41.7	34.4	50.6
1942	24.6	22.0	37.1	52.6	60.6	68.8	72.9	69.7	62.0	52.9	40.1	24.9	49.0
1943	21.1	28.0	33.1	43.8	56.9	72.4	72.5	70.9	60.6	50.4	37.5	26.9	47.8
1944	29.7	27.7	30.3	42.8	63.0	71.2	73.3	73.4	64.7	52.3	41.6	24.6	49.6
1945	18.4	27.7	48.1	50.2	52.4	65.2	70.2	70.7	63.9	50.1	40.2	23.1	48.3
1946	27.1	27.1	46.1	47.9	55.5	65.0	71.4	66.7	64.2	56.5	42.8	31.8	50.2
1947	28.4	20.4	29.6	44.6	52.9	64.0	68.6	75.6	63.8	59.5	35.8	27.5	47.6
1948	19.4	24.6	34.4	50.6	56.0	66.7	73.1	71.6	66.1	49.3	44.2	30.6	48.9
1949	30.0	29.8	35.3	47.3	60.4	72.1	75.3	71.4	59.0	57.9	39.3	32.5	50.9
1950	32.6	26.4	30.6	41.5	59.8	67.9	70.6	68.7	62.3	56.4	35.0	24.3	48.0
1951	26.8	26.5	35.6	45.7	60.1	66.9	71.8	68.1	60.9	54.5	33.0	27.4	48.1
1952	28.7	29.6	34.5	49.7	57.5	72.2	75.4	71.1	64.8	48.3	42.8	33.3	50.7
1953	29.9	31.5	37.4	44.4	61.2	71.2	73.4	73.0	63.6	56.7	42.7	31.8	51.4
1954	25.0	33.1	32.7	50.1	54.0	70.3	70.8	68.7	63.3	53.2	38.6	28.3	49.0
1955	25.3	25.3	34.8	53.9	62.0	67.7	77.6	75.9	65.1	54.5	36.9	26.5	50.5
1956	24.9	28.4	32.5	45.6	58.5	71.4	71.8	70.6	60.0	56.9	39.8	34.4	49.5
1957	20.8	31.0	36.9	49.2	57.6	69.4	73.2	70.7	63.4	51.2	40.0	33.9	49.8
1958	26.3	22.3	36.5	50.1	59.9	63.9	72.5	71.0	63.8	54.7	41.9	22.1	48.8
1959	20.8	25.7	34.6	49.4	63.0	70.5	73.4	75.6	67.0	51.3	36.2	33.8	50.1
1960	27.5	27.2	25.7	50.7	57.6	66.8	70.5	71.1	66.8	52.2	41.0	22.9	48.3
1961	21.7	31.4	39.1	43.6	56.1	67.3	72.8	71.7	68.7	55.0	41.6	28.5	49.8
1962	21.9	23.7	35.1	48.2	66.6	70.6	71.3	71.4	62.2	55.4	40.7	26.9	49.5
1963	17.6	21.1	38.0	50.8	58.1	71.6	73.7	69.1	62.9	62.1	44.9	23.2	49.4
1964	29.3	28.0	36.5	50.4	64.5	71.2	75.1	67.5	64.6	52.0	44.7	28.2	51.0
1965	24.2	24.9	29.5	46.6	64.2	68.4	69.9	68.5	65.6	50.5	42.7	34.7	49.1
1966	21.3	28.9	39.8	46.4	56.1	72.6	76.6	70.9	63.8	51.2	43.4	28.1	49.9
1967	28.6	22.7	35.6	49.9	56.0	72.7	71.6	69.1	62.8	53.0	37.2	32.8	49.3/
1968	23.9	24.6	40.2	52.0	57.6	69.1	72.6	73.5	66.1	54.6	41.8	28.3	50.4
1969	24.9	29.4	35.5	51.3	60.2	65.3	73.5	74.3	65.7	53.2	40.2	29.4	50.2

Total Precipitation (Inches)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Ann'l
1940	1.81	1.24	2.15	2.70	3.28	4.76	2.16	5.36	1.18	2.82	2.06	3.42	32.94
1941	1.66	.59	1.54	2.47	2.52	4.01	2.39	2.65	1.83	4.20	3.19	1.03	28.88
1942	2.12	1.80	2.77	1.83	2.65	6.54	4.35	1.75	3.11	3.54	2.71	3.86	37.03
1943	2.40	1.66	2.92	3.20	10.49	3.37	4.15	2.96	2.28	1.39	2.12	.49	37.43
1944	1.50	1.65	3.16	3.24	3.30	4.57	1.71	2.57	1.23	1.06	1.79	1.50	27.28
1945	.52	1.37	4.08	2.88	7.85	4.08	2.77	2.06	5.90	3.13	1.47	1.73	37.84
1946	1.56	1.86	1.33	.55	4.08	3.22	1.02	3.11	1.57	2.65	1.59	2.13	24.67
1947	1.28	.41	2.43	6.27	5.56	2.49	1.13	4.10	1.10	3.40	1.88	1.74	31.79
1948	1.25	2.31	3.51	4.15	3.44	3.04	2.48	.87	2.01	.53	3.55	2.16	29.30
1949	2.96	3.32	2.73	2.24	2.46	2.20	3.20	3.65	3.24	3.22	1.05	3.43	33.70
1950	4.44	3.64	3.68	4.44	2.09	3.13	2.92	2.21	2.95	2.87	3.51	1.12	37.00
1951	1.19	2.88	2.29	2.86	5.01	2.58	3.71	2.36	1.16	4.44	2.67	3.70	34.85
1952	3.68	1.49	2.77	3.55	3.05	1.57	1.50	1.05	2.19	1.48	3.44	2.12	27.89
1953	1.86	.89	2.57	3.20	1.86	3.23	4.10	1.80	2.72	.65	.91	1.46	25.25
1954	1.59	4.65	4.02	2.95	.70	5.86	.76	2.24	1.50	7.29	1.42	1.66	34.64
1955	1.78	.79	2.06	1.61	2.91	1.98	3.38	1.72	1.28	3.55	3.30	.91	25.27
1956	.88	2.05	2.47	5.35	4.36	2.41	2.08	3.47	.62	.46	2.06	1.07	28.08
1957	1.74	1.33	1.28	4.13	3.31	3.45	3.48	2.77	2.93	3.64	2.69	3.62	34.37
1958	.60	.36	.26	2.25	.88	2.62	3.82	3.59	2.60	2.54	2.64	.31	22.47
1959	2.77	1.99	2.85	4.10	2.90	1.21	4.77	3.19	1.88	4.69	3.26	2.26	35.87
1960	2.79	2.26	.86	1.98	2.24	5.89	4.71	2.59	1.04	2.31	1.29	.34	28.30
1961	.18	2.19	2.99	6.57	2.16	3.73	1.61	4.23	4.75	1.78	2.87	.84	33.90
1962	1.48	1.92	.66	.98	1.71	4.33	3.99	3.04	3.45	2.09	.95	.93	25.53
1963	.48	.30	2.25	2.91	2.61	1.40	1.56	1.25	1.15	.59	1.48	.87	16.85
1964	1.45	.38	2.20	5.23	2.26	2.96	2.78	3.47	1.19	.06	.76	1.61	24.35
1965	3.08	1.96	2.65	2.28	2.00	2.38	1.23	4.12	2.05	2.83	1.30	4.79	30.67
1966	.41	.56	2.28	2.86	1.72	2.96	1.54	3.33	2.63	.99	3.05	4.45	26.78
1967	2.49	1.38	.87	3.17	1.00	6.20	3.31	1.89	1.99	3.73	2.64	4.94	33.61
1968	1.69	2.03	2.25	1.88	5.65	8.45	4.51	3.90	3.03	.87	3.06	3.01	40.33
1969	2.75	.16	1.47	4.36	3.51	4.47	6.03	1.95	1.02	1.89	2.77	1.42	31.80

PROBABILITIES FOR SELECTED TEMPERATURES*

Temp. (°F)	Percent Probability of Indicated Temperature, or Lower, Occurring On or After Date in SPRING			Percent Probability of Indicated Temperature, or Lower, Occurring On or Before Date in FALL		
	90%	50%	10%	10%	50%	90%
32	Apr 17	May 2	May 17	Oct 1	Oct 17	Nov 2
28	Apr 3	Apr 18	May 3	Oct 16	Nov 1	Nov 17
24	Mar 20	Apr 4	Apr 19	Oct 28	Nov 13	Nov 29
20	Mar 10	Mar 25	Apr 9	Nov 9	Nov 25	Dec 11
16	Feb 27	Mar 14	Mar 29	Nov 16	Dec 2	Dec 18

*"Michigan Freeze Bulletin", Research Report #26, Michigan State University, May 1965.

STATION HISTORY

Station was established in 1880 at Astronomy Observatory and remained there until July 1944 at which time it was moved to the roof of the Geology Department Building. In October 1956, the station was moved to the roof of the East Engineering Building. The instruments were moved, October 30, 1961, 300 feet north to a new location on the roof and is now 800 feet north of the Eastern University Branch Post Office.

APPENDIX B

Hydrology Methodologies
and
Techniques

CONTENTS

HYDROLOGY METHODOLOGIES AND TECHNIQUES

General

The discharge hydrograph produced by an intense rainstorm is the result of two relatively independent aspects of the runoff process, one controlling the volume of the surface runoff and the other establishing the shape of the surface runoff hydrograph. Various methods of determining the design flood magnitude treat these two aspects somewhat differently. These different methods can be classified as follows:

1. Methods utilizing empirical equations or curves
2. Statistical methods
3. Storage routing procedures
4. Unit hydrograph methods

The methods in the first two categories do not, in general, explicity consider the phase controlling the volume of surface runoff and directly compute design peak rates of discharge, whereas the methods in the latter two categories, in general, utilize the infiltration capacity concept (Ref. 1) to compute the volume of surface runoff.

Empirical methods use one or more empirical constants to represent the combined effect of a number of climatic factors and watershed characteristics that affect the runoff process. It is extremely difficult to determine the necessary constant for a particular location and, furthermore, they do not lend themselves to accounting for changing conditions.

Statistical methods utilize observed flood peaks on a particular basin and assume that they represent a sample of random and independent events from a population whose basic distribution is known. Sometimes methods are applied utilizing regional values for some of the parameters. The length of the available records is usually small for predicting rare floods (having recurrence interval of 100 years or more). Furthermore, statistical methods (Ref. 2) inherently lack the flexibility to incorporate the effects of watershed changes due to urbanization.

Volume of Surface Runoff

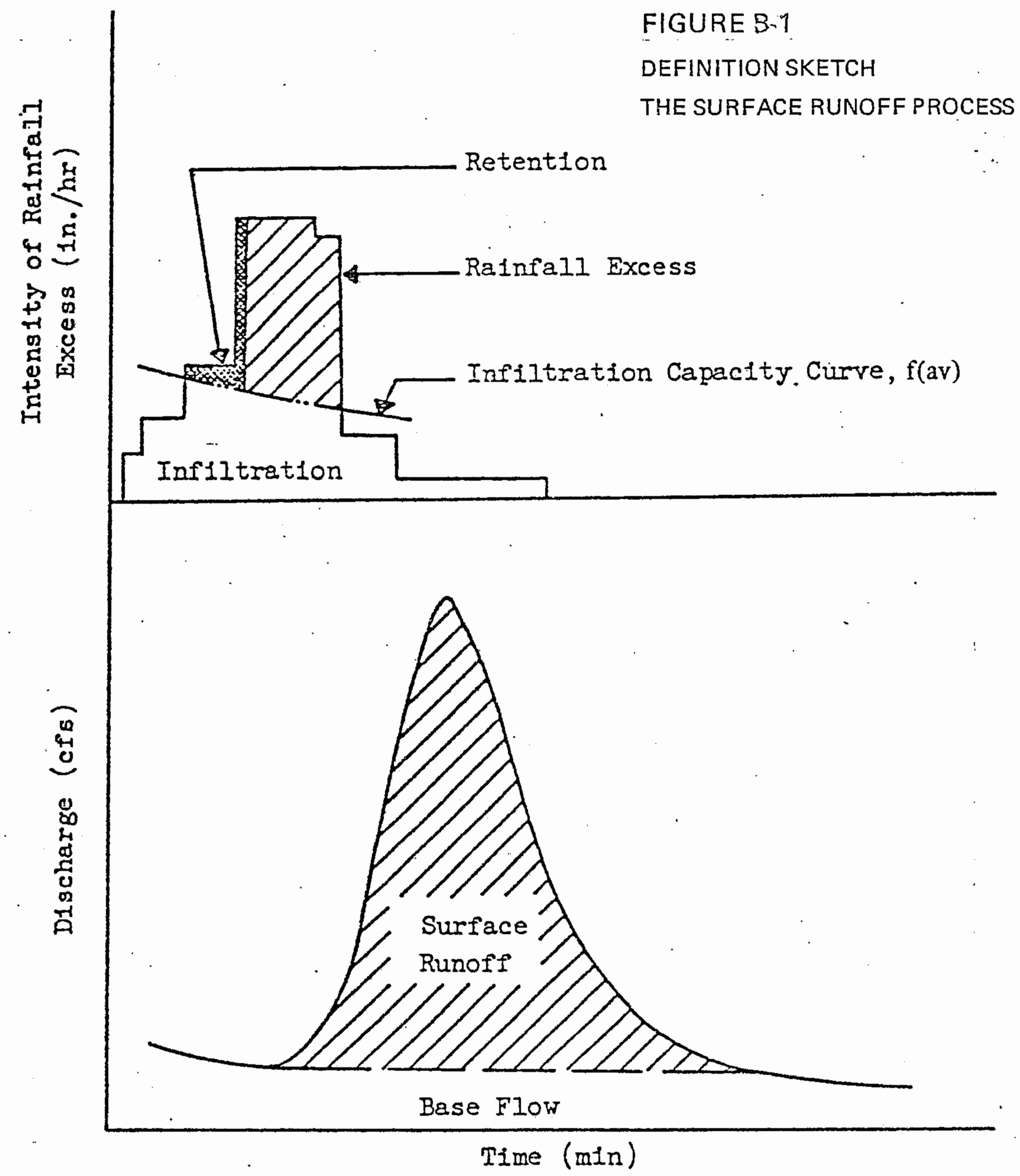
A small portion of the initial rainfall is prevented from reaching the ground by the interception process. Of the rainfall reaching the ground, a small portion is stored in static surface storage (also called depression storage) and is permanently prevented from becoming surface runoff. The sum of these two abstractions is called permanent retention, retention (Ref. 3) or initial losses.

On an impermeable surface (such as pavements, parking lots, etc.), the surface runoff starts as soon as the requirements of the retention are satisfied. The latter's magnitude (measured in terms of average depth over the entire surface) seldom exceeds 1/10-inch. For pavements, Tholin and Keifer (Ref. 4) have suggested a value of 1/16-inch, whereas for small paved areas Viessman (Ref. 5) reported a range 0.04 to 0.10-inch.

On a permeable land surface, the magnitude of the retention is comparatively large. Brater (Ref. 6) has shown that a reasonable limiting value for this quantity for watersheds in Southeast Lower Michigan is about 0.22-inch. The other very important difference between impermeable and permeable surfaces is that on a permeable surface the surface runoff cannot start until the rainfall and/or snowmelt exceeds the prevailing infiltration capacity. Horton (Ref. 1) defined the infiltration capacity as the maximum rate at which a soil can absorb water under given conditions. Thus, after subtracting the infiltration and filling the initial retention, the residual rainfall becomes the input to the surface runoff system. This input is termed "rainfall excess", or "runoff-producing rainfall" and "effective rainfall" (see Figure B-1). In this report, the term "rainfall excess" is invariably used. The volume of the surface runoff is equal to the volume of the rainfall excess and is equal to the total rainfall minus retention and infiltration.

Infiltration Capacity. When the rate of rainfall exceeds the infiltration capacity, the rate of infiltration will be equal to the infiltration capacity; otherwise, the rate of infiltration will equal the rate of rainfall. The infiltration capacity varies considerably over the drainage area from one place to another depending on the type and condition of the soil and vegetation. This is exemplified by the situations where only a portion of the watershed contributed to runoff. For example, Betson (Ref. 7) has reported cases in which portions as small as 5% of the watershed produced all of the runoff hydrograph at the outlet. However, for flood-producing storms, the variations tend to decrease considerably. For most soils, the infiltration capacity varies greatly from time to time depending primarily on the moisture content of the surface layer of the soil and on the degree and nature of the vegetative cover. Illustrations of the seasonal variation in infiltration capacity are available in the literature (Ref. 3, 6, 8, 9). The factors which control infiltration capacity (Ref. 8) are quite well understood, but quantitative values for any region must be obtained from the analyses of the rainfall and the runoff hydrographs observed on watersheds in the region where design flood magnitudes are required. Values obtained from one region cannot be applied to another region.

FIGURE B-1
DEFINITION SKETCH
THE SURFACE RUNOFF PROCESS



Effects of Urbanization on Volumes of Surface Runoff. The effects of urbanization on flood volumes can be broken into the following two components:

1. Effect on infiltration capacity of the pervious area
2. Effect due to creation of additional impervious areas

1. Effect on Infiltration Capacity of the Pervious Area. The effect of urbanization on the infiltration capacity of the pervious portion of the watershed could be either to reduce or to increase the infiltration capacity. For example, the conversion of a forested area to lawns and parks would most likely reduce the infiltration capacity. On the other hand, if a soil containing some clay is converted from pastures to well-kept lawns, the infiltration capacity might be increased.

   During a study conducted at the University of Michigan, the results of computations of infiltration capacity for more than twenty watersheds of the Clinton River and Rouge River basins did not indicate any significant difference from one watershed to another. These watersheds range from rural, with a population density of less than 100 persons per square mile, to fully urbanized, with a population density of over 7,000 persons per square mile. Therefore, for this study it is assumed that urbanization has no effect on the infiltration capacity of the pervious portion of the watershed.

2. Effect Due to Creation of Additional Impervious Areas. An important effect of urbanization is to create additional impervious areas. This effect must be taken into account in computing the volume of the surface runoff. Even a wild river has some impervious area due to the presence of the water surface area of lakes and the river. One may estimate the area of impervious surfaces from aerial photographs, but it is doubtful that all such impervious areas contribute nearly 100% runoff. For this reason, Brater (Ref. 6) introduced the concept of "Hydrologically Significant Impervious Area" (HSIA), which represents the percent of the total area which always contributes a runoff equal to the total precipitation minus retention, or, in other words, almost 100% runoff.

   This quantity is derived from the analysis of hydrographs resulting from large storms occurring after a long summer dry spell, when the infiltration capacity of the pervious portion is very high and therefore the entire surface runoff occurs from the hydrologically significant impervious area (HSIA). This percentage is believed to be considerably smaller than the percentage of the physical impervious area. The research done at the University of Michigan, under the direction of Professor E.F. Brater (Ref. 7), showed the following correlation between HSIA and population density, P_d, for twelve watersheds in Southeastern Lower Michigan.

$$HSIA = 1.38\ P_d$$

where:

P_d is the population density in thousands of persons per square mile
HSIA is in percent of total area

Flood Peaks and Shape of the Hydrograph. Three different sets of factors affect the flood peaks and the shape of the hydrograph. These may be referred to as climatic, physiographic and urban development factors. In the first category, the intensity and duration of rainfall and the volume of the rainfall excess are the important factors. Those considered important in the second category are the shape and drainage area of the watershed, the length and slope of the main channel, the dynamic storage characteristics and the watershed drainage network. In the last category, the important factors are considered to be population density, percent of impervious area, degree of industrialization and number of dwellings per square mile. Many of these factors are interdependent.

The quantitative relationship between the watershed response parameters and the watershed characteristics has been developed at the University of Michigan (Ref. 10) through the use of a mathematical model and the analysis of data from natural watersheds. It has been shown that the size of the drainage area in the watershed is the single most important watershed characteristic to affect the shape of the unit hydrograph. The quantitative relationship developed between the response parameters and the population density has been reported by Brater and Sherill (Ref. 9).

Unit Hydrograph Theory

The unit hydrograph theory was first proposed by Sherman (Ref. 11) in 1932. This, together with the infiltration concept, formed the first rational basis for analyzing and synthesizing flood hydrographs. Subsequently, the unit-hydrograph theory was expanded and reinforced by a number of investigations, notably those of Horner and Flynt, Bernard, Synder, Brater, and Sangal (Refs. 12, 13, 14, 15 and 10, respectively).

The unit hydrograph theory is based on the following premises:

- The runoff hydrograph for a "unit storm" is an invariant property of a drainage basin and is called the unit hydrograph. In other words, the unit hydrograph is the watershed response to a unit storm input.

- The ordinates of the runoff hydrograph resulting from a storm similar to the unit storm will be directly proportional to those of the corresponding ordinates of the unit hydrograph, the constant of proportion being equal to the depth of the rainfall excess.

- The hydrograph resulting from two unit storms occurring in succession can be obtained simply by adding the corresponding ordinates of the two unit hydrographs after displacing one from the other by an amount equal to the time lapse between the storms.

These, in essence, are the properties of a linear system (Ref. 16); that is why it can be said that the unit hydrograph theory considers the hydrologic system to be a linear system.

Unit Storm. As implied originally by Sherman, a unit storm is a rainfall, uniformly spread over the drainage basin and of almost uniform intensity lasting for a day, which would result in a hydrograph of surface runoff amounting to 1-inch on the drainage area. However, it is currently interpreted by many to represent an effective storm of uniform intensity of 1/H in./hr. lasting for H hours and amounting to 1-inch of runoff. The resulting unit hydrograph is called an H-hr unit hydrograph.

On the other hand, Brater (Ref. 15) suggested as early as 1940 that the duration of precipitation excess has no effect on the unit hydrograph shape so long as the precipitation excess does not exceed a certain critical duration; for small basins the latter approximately equals the period of rise.

Parameters Characterizing the Unit Hydrograph. The unit hydrograph can be represented in terms of the following eight direct and easily definable parameters shown in Figure B-2.

1. Peak rate of discharge, Q_p. It is sometimes expressed as peak discharge per unit depth of rainfall excess, q_p. Also, it may advantageously be expressed as peak discharge per unit depth per unit area, q_{pA}. These are related as follows:

$$Q_p = (q_p)(D) = (q_{pA})(A)(D)$$

where D is the average depth of rainfall excess over the basin, and A is the catchment area of the basin.

2. Response Time Parameter. The following three ways of expressing the response time parameter are used in this study and are shown in Figure B-2.

   a. The Period of Rise, t_R, is defined as the time from beginning of rainfall excess to the peak rate of discharge.

   b. Lag, t_p, is defined as the time interval between the centroid of rainfall excess and the peak rate of discharge. For a rainfall excess of uniform intensity, i_o, and of duration, t_o:

$$t_p = t_R - t_o/2$$

   c. Significant Period of Rise, t_r, is defined as the time from 10% of the peak discharge to the peak rate of discharge.

3. Width of the surface runoff hydrograph at the 75% level of the peak discharge, W_{75}.

4. Width of the surface runoff at the 50% level of the peak discharge, W_{50}.

5. Width of the surface runoff hydrograph at the 25% level of peak discharge, W_{25}.

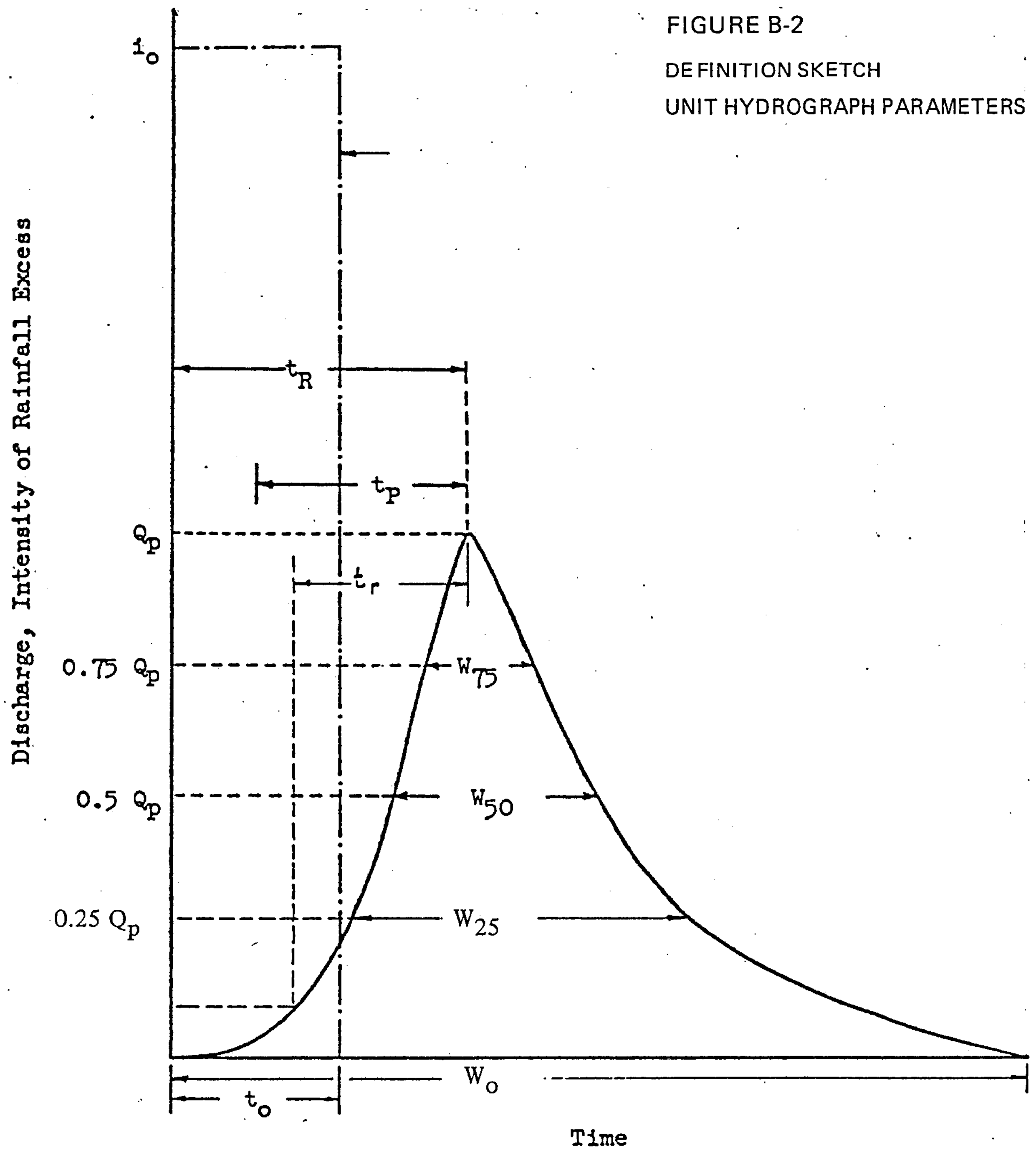
6. Width of the surface runoff hydrograph at the 0% level of peak discharge, W_0.

The volume under the surface runoff hydrograph is known, since by definition, it equals unity.

FIGURE B-2

DEFINITION SKETCH

UNIT HYDROGRAPH PARAMETERS



McNamee Porter & Seeley

CONSULTING ENGINEERS

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Ann Arbor, Michigan Stormwater Management Plan for Allen's Creek Drainage System

Discharge Frequency Relationships

General. The method used in this study to predict flood magnitudes and frequencies is Brater's method, which is based on the infiltration capacity unit hydrograph concept incorporating the factors of retention and HSIA (hydrologically significant impervious area) presented earlier. The season when the flood event occurs has a very significant effect on the resulting flood. Brater's method takes the seasonal effect into account by separately analyzing the flood events for the summer season (June through September) and winter season (remaining eight months). First, flood frequency relationships are developed for any location for a given level of urbanization separately for summer and winter seasons. The two are considered independent of one another. Therefore, the total frequencies are obtained as follows:

Let T_s = Recurrence interval in years for a flood of magnitude, Q cfs, due to summer rain,

T_w = Recurrence interval in years for a flood of the same magnitude, Q cfs, due to winter rain,

then T = Recurrence interval in years

$$= \frac{T_s \times T_w}{T_s + T_w}$$

Expressing this in terms of probabilities, the total probability of a flood of magnitude Q is as follows:

Let P_s = Probability of a flood of magnitude Q, due to a summer rain.

P_w = Probability of a flood of the same magnitude Q, due to a winter rain.

then P = Total probability of a flood of magnitude Q.

$$= P_s + P_w$$

Design Rainfall. Brater et al (Ref. 17) have analyzed the daily rainfall for 1950 station years and have given separate design curves for 24-hour summer and winter storms. They have included the effect of snowmelt and excluded snowfall from their analysis in the development of winter storm frequencies. The point rainfall of a given frequency is more than the average rainfall on various size areas for the same frequency. The U.S. Weather Bureau (Ref. 18) has derived ratios between point rainfall and average rainfall of the same frequency for basins up to 400 square miles in area.

The prediction of floods of various frequencies from precipitation and/or snowmelt requires accurate time histories of inputs to the surface runoff system. Typical seasonal time-intensity rainfall patterns of various frequencies are also essential input ingredients. Brater et al (Ref. 17) have examined the chronological rainfall intensity patterns during rains in Southeastern Lower Michigan. The results of their analyses of numerous summer rains and winter rains are reproduced in Table B-1 and are also shown graphically in Figure B-3. The values are in percent of daily precipitation. It can be seen that the patterns are very different from summer and winter storms.

Design Infiltration Capacity, Retention and HSIA. The design values of the infiltration capacities of the pervious portion of the watershed for the Allen's Creek Drain and its tributaries used in this study were 0.40 inch/hour for the summer and 0.10 inch/hour for the winter. For the hydrologically impervious portion of the watershed, the infiltration capacity is zero.

For the impervious portion of the watershed, zero retention was used. For the pervious portion, the design values used for the summer retention were 0.15 inch for the Allen's Creek Drain and its tributaries. Winter retention was assumed to be 0.0-inch.

HSIA was computed from the following equation:

HSIA = 1.38 x population density in thousands of persons per square mile

This formula gave the porportion of the hydrologically significant impervious area as a percentage of the total drainage area.

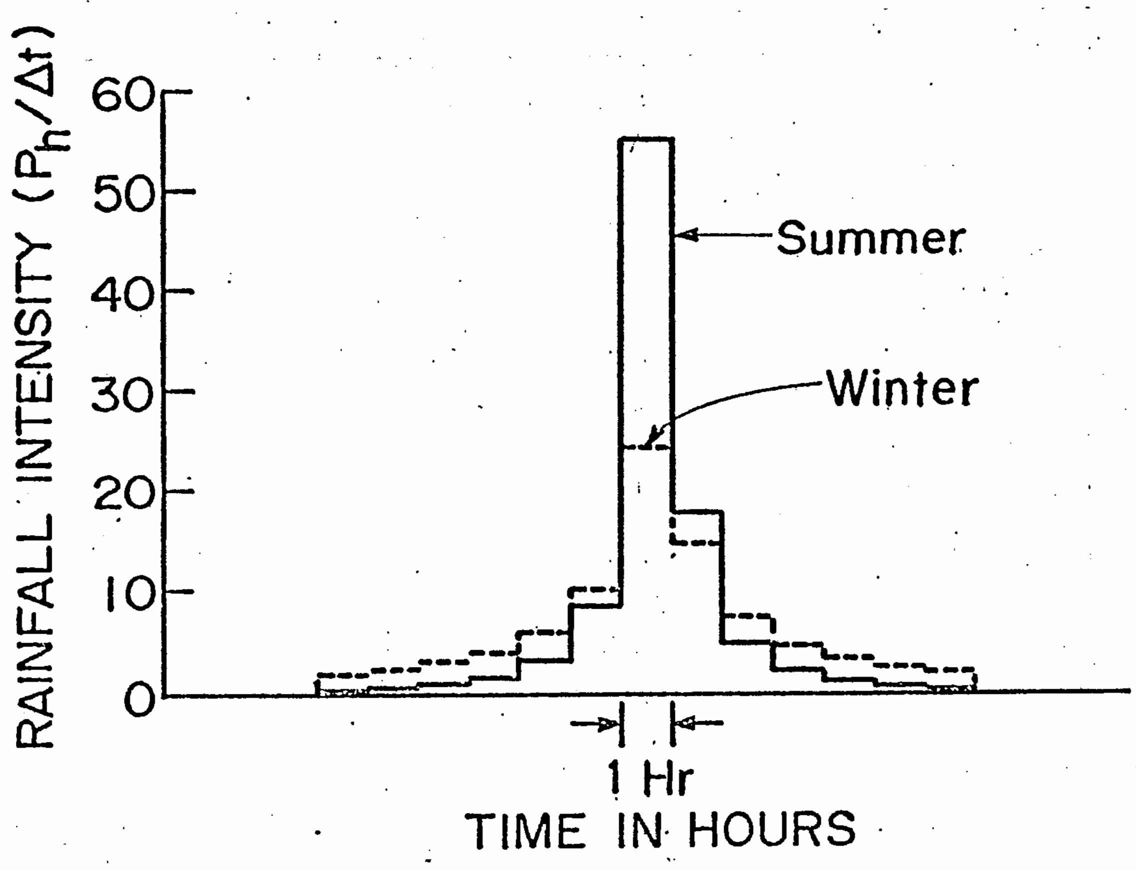
Table B-1

Average Time - Intensity Patterns

(Percentage of Daily Precipitation)

Hours Before Maximum	Summer Average Precipitation	Winter Average Precipitation
6	0.39	1.98
5	0.58	2.40
4	0.92	3.03
3	1.58	4.05
2	3.16	5.91
1	8.46	10.08
Maximum Hour	55.0	24.06
Hours After Maximum		
1	17.56	14.60
2	4.90	7.52
3	2.18	4.82
4	1.19	3.47
5	0.72	2.68
6	0.47	2.17

FIGURE B-3
TYPICAL SEASONAL
HYETOGRAPHS



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Plan for Allen's Creek Drainage System

Procedure Used for Predicting Flood Magnitudes and Frequencies. In this method, the computed flood peak is assigned the same frequency as the rainfall used to compute the flood peak. Also, at any given location one unit hydrograph was used both for summer and winter for all frequency rainfalls for a particular level of urbanization. The flood frequency results, therefore, show a significant increase in flood peaks due to urbanization, irrespective of frequency. The procedure for developing flood frequency relationshps is explained below by means of an example for the Clinton River at Mt. Clemens (Moravian Drive) for Year 2025 level of urbanization.

Example Computation for Clinton River at Moravian Drive for the Year 2025 Projected Level of Urbanization.

Drainage area of the basin = 747 square miles

Projected population density for the Year 2025 = 2425 persons/square mile

HSIA = 1.38 P_d = 1.38 x 2.425 = 3.3% = 0.033

The following procedure was used to compute the flood peak discharge for a 100-year summer rainfall. The computation of the volume of surface runoff is shown in Table B-2.

100-year summer point rainfall = 4.57 inch

Average rainfall/point rainfall = 0.81 inch

Average rainfall = 4.57 x 0.81 = 3.70 inch

The hourly rainfall excess plus retention values on the pervious area, shown in row 3, were obtained by subtracting infiltration capacity, f_s = 0.40-inch/hour, from the corresponding hourly rainfall values.

Table B-2

Example Surface Runoff Computations for a 100-Year Summer Storm

		2 Hours Before	1 Hour Before	Maximum Hour	1 Hour After	2 Hours After
1.	Ratio of hourly rainfall to 24 hour rainfall	0.037	0.100	0.551	0.120	0.032
2.	Hourly rainfall, inches	0.137	0.370	2.039	0.444	0.118
3.	Rainfall excess plus retention, inches	0.0	0.0	1.639	0.044	0.0

(continued)

Table B-2 (continued)

		2 Hour Before	1 Hour Before	Maximum Hour	1 Hour After	2 Hours After
4.	Rainfall excess, inches	0.0	0.0	1.389	0.044	0.0
5.	Surface runoff from pervious area, inches	0.0	0.0	1.343	0.043	0.0
6.	Surface runoff from impervious area, inches	0.005	0.012	0.067	0.015	0.004
7.	Total hourly surface runoff, inches	0.005	0.012	1.410	0.058	0.004
8.	Total volume of surface runoff, inches	1.489				

A summer retention value of 0.25-inch was deducted to give hourly rainfall excess values, shown in row 4. These values were converted to surface runoff values over the total drainage area by multiplying by (1 - 0.033), or 0.967. The resulting values are shown in row 5.

The hourly surface runoff values from the impervious area were computed by multplying row 2 by 0.033. The retention in the impervious area was assumed to have been filled by the antecedent rainfall (see row 6).

The total hourly surface runoff values are shown in row 7.

The total volume of surface runoff from the 100-year summer rain will be 1.489 inches. Assuming the duration of a unit storm for this watershed for the Year 2025 urban conditions is approximately 16 hours, the hourly surface runoff could be combined into one single input of 1.489 inches.

Unit hydrograph peak	=	22.1 cfs/sq.mi./in. (Figure B-3)
Peak rate of surface runoff	=	22.1 x 747 x 1.489
	=	24,600 cfs
Flood peak discharge for a 100-year summer rain	=	24,600 cfs

The above procedure was used for the 10 and 100 year rains for the Allen's Creek system to obtain the flood peak discharges corresponding to each of these frequencies. The winter flood peak discharge frequency relationship was obtained following the same procedure as the one used for the summer rain. The infiltration capacity for the summer was taken to be 0.40-inch/hour and the retention for the summer was taken to be 0.15-inch/hour. The infiltration capacity for the winter was taken to be 0.10-inch/hour and the retention for the winter was taken to be zero. Total frequency flows were then determined at selected locations on Allen's Creek, West Park-Miller Drain, Murray-Washington Drain and Eberwhite Drain. Peak flow values are shown in Table B-3. Drainage areas and population densities are presented in Table B-4. Population densities were determined from the 1980 United States Census Tracts for the City of Ann Arbor and from pertinent data supplied by the City of Ann Arbor Planning Department

Table B-3

Peak Discharges

Allen's Creek	Cross section Location	Drainage Area (sq.mi.)	Overland Flow/Total Flow (cfs) Frequency (yrs.) 10		100	
Huron River	9					
		5.50	1050	(2075)	2355	(3380)
Kingsley	50					
		5.03	985	(1825)	2100	(2940)
West Park-Miller Drain	61					
		3.85	610	(1450)	1470	(2310)
Murray-Washington Drain	69					
		2.60	510	(1030)	1240	(1760)
Eberwhite Drain	85					
		2.17	415	(775)	920	(1280)
Williams	90					
		1.81	200	(545)	690	(1035)
Hill Street	115					
		1.60	125	(460)	565	(900)
Hoover	130					
		0.95	85	(280)	335	(535)
Upstream Stadium Blvd. (d/s end of open channel)	155					
		0.74	260	(260)	480	(480)
Downstream of Main Street (u/s end of open channel)	170					
		0.50	0	(260)	135	(480)
Upstream South Main	200					
		0.24	0	(100)	0	(180)
Upstream End	—					
Eberwhite Drain						
Allen's Creek	545					
		0.43	115	(265)	320	(470)
Upstream End	—					

(continued)

Table B-3 (continued)

Allen's Creek	Cross section Location	Drainage Area (sq.mi.)	Overland Flow/Total Flow (cfs) Frequency (yrs.) 10		100	
Murray-Washington Drain						
Allen's Creek	695					
		1.09	175	(430)	325	(580)
Crest	—					
West-Park Miller Drain						
Allen's Creek	950					
		1.34	165	(425)	390	(650)
Junction of N/S Branches	965	1.29				
South Branch						
Junction with N. Branch	823					
		0.76	0	(155)	145	(305)
Ravena	865					
		0.68	0	(155)	295	(455)
Upstream End	—					
North Branch						
Junction with S. Branch	965					
		0.53	0	(260)	205	(465)
Upstream End	—					

Table B-4
Hydrologic Characteristics

Location	Drainage Area (sq.mi.)	Population Density (persons/sq.mi.)
Allen's Creek		
u/s South Main - north portion	0.25	4,500
u/s South Main - south portion	0.24	3,000
At Stadium Blvd.	0.25	2,500
At Hoover - east of RR	0.65	11,000
At Hill - west of RR	0.21	7,000
At Williams - west of RR	0.21	7,000
At Washington - east of RR	0.36	9,000
Eberwhite Drain		
At Allen's Creek Junction	0.43	6,000
Murray Washington Drain		
At Allen's Creek Junction	1.09	5,500
West Park-Miller Drain		
West Park Drain at Ravena St.	0.68	6,000
West Park Drain at junction with Miller Drain	0.08	6,000
Miller Drain at Maple Ridge - North Branch	0.25	3,000
Miller Drain at Maple Ridge - West Branch	0.28	6,000
Miller Drain at Allen's Creek	0.05	6,000
Allen's Creek		
At Huron River	0.47	7,000

Hydrograph Routing Model

A stream network model was utilized to combine flood hydrographs and channel route the hydrographs. In addition, the model was utilized to route selected hydrographs through large storage aras where significant storage volume was present.

The river routing component is used to represent flood wave movement in a river channel. The input to a particular river routing reach is an upstream hydrograph. The hydrograph is routed to the downstream point based on the characteristics of the channel's particular shape, slope and roughness coefficient.

The reservoir component is used to represent storage areas through which the flood wave must travel. This component is similar to the river routing component. The river routing component was used to represent the storage-outflow characteristics of natural detention areas created by valleys, closed off at the downstream end by natural or man-made barriers. The reservoir component functions by receiving upstream inflows and routing these inflows through a reservoir using a storage routing method.

The stream network model yields resultant flows at selected locations which would occur overland if the existing storm sewers were not in place. Therefore, storm sewer capacities were accounted for and net overland flows computed. Storm sewers were assumed to be surcharged to the point the hydraulic gradeline approximated the ground surface.

Overland flows for Allen's Creek, West Park-Miller Drain, Murray-Washington Drain and Eberwhite Drain are presented in Table B-3.

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APPENDIX C

Public Meetings

PLEASE SIGN IN

Ann Arbor Stormwater Management Plan
Public Meeting No. 1, August 25, 1982, Slausson Middle School

Name	Address	Representing
John S. Wise		McNamee, Porter and Seeley
Vyto Kaunelis		McNamee, Porter and Seeley
Jim Rogers		S.E.M.C.O.G.
Jim Murray		Washtenaw Co. Drain Commr.
Kathy Edgner	1600 Arborview	
Grace Shackman		Old West Side News
Leigh Chizek		City Engineer
Jim Chaffers		North Central Assoc. (NCPOA)
John Oyer		McNamee, Porter and Seeley
Helen M. West		League of Women Voters
Tom McKinnev		
Letty M. Wickliffe		NCPOA
Ethel K. Potts	1014 Elder	My family
Lou and Beth Velker		City Council 454 5th St.
Phil Weaver		Developer
Dick Force		McNamee, Porter and Seeley
Sabah Yousif		City of Ann Arbor (Eng)
Suresh Sangal		McNamee, Porter and Seeley
Peter Pollack		Self
Godfrey W. Collins		City of Ann Arbor
Gerry Clark		City Planning Dept.
Mrs. Russell J. Burns	406 Maple Ridge	
Paul M. Spurlin	505 N. Seventh St.	
Alix R. Spurlin	505 N. Seventh St.	
Kathleen Hubler	307 Mulholland	
Charles E. Hubler	306 Mulholland	
Joyce Chesbrough	3176 Lakewood	City Council
Susan Buchan	242 Murray Ave.	
Barry Johnson		Wash. Co. Health Dept.
Leslie Morris		Ann Arbor City Council
Tom Blessing		

Ann Arbor Stormwater Management Plan
Public Meeting #2 on Allen's Creek Drain
Sept. 8, 1982 at Slausson Middle School

Name	Address	Representing
John Oyer		McNamee, Porter & Seeley
Dick Force		McNamee, Porter & Seeley
Vyto Kaunelis		McNamee, Porter & Seeley
Wm. R. Wheeler		City Engineering Dept.
S. H. Yousif		City Engineering Dept.
Godfrey Collins		Acting City Administrator
Lou Velker	454 5th St.	City Council
Mariana Kopacz	209 S. Fourth	LWVAAA
Don East	628 Revena Pl.	
Peter Pollack	515 Detroit St.	
A. H. Wheeler	234 8th St.	Self
Doris Preston	1731 Fair St.	Virginia Park neighborhood
Susan Buchan	242 Murray Ave.	Self & Murray Mulholland
Michael Mouradian	110 W. Revena	
Grace Shackman	515 Soule	Old West Side News
Marcia Dorsey	415 W. Washington	Huron River Watershed Council
Kathy Edgren	1600 Arborview Blvd.	
Jim Murray	P.O. Box 8645, Ann Arbor	Drain Comm. Office
Jim Rogers	1249 Washington Blvd., Det.	SEMCOG
Ethel K. Potts	1014 Elder 48103	
Joyce Chesbrough	3176 Lakewood	City Council
Letty Wickliffe	305 Beakes	N.C.P.O.A. 769-6654
Mrs. Russell J. Burns	406 Maple Ridge	
Meredith Woods	115 Depot	First Martin
Tom McKinney	1102 Prospect	
Leigh A. Chizek		City Engineer
Tom Blessing		

ANN ARBOR STORMWATER MANAGEMENT PLAN
PUBLIC MEETING #3 ON ALLEN'S CREEK DRAIN

Public Meeting #1 on Selected Drains on the East Side
Public Meeting #1 on the Stormwater Utility at Slauson Middle School

January 26, 1983 at Slauson Middle School

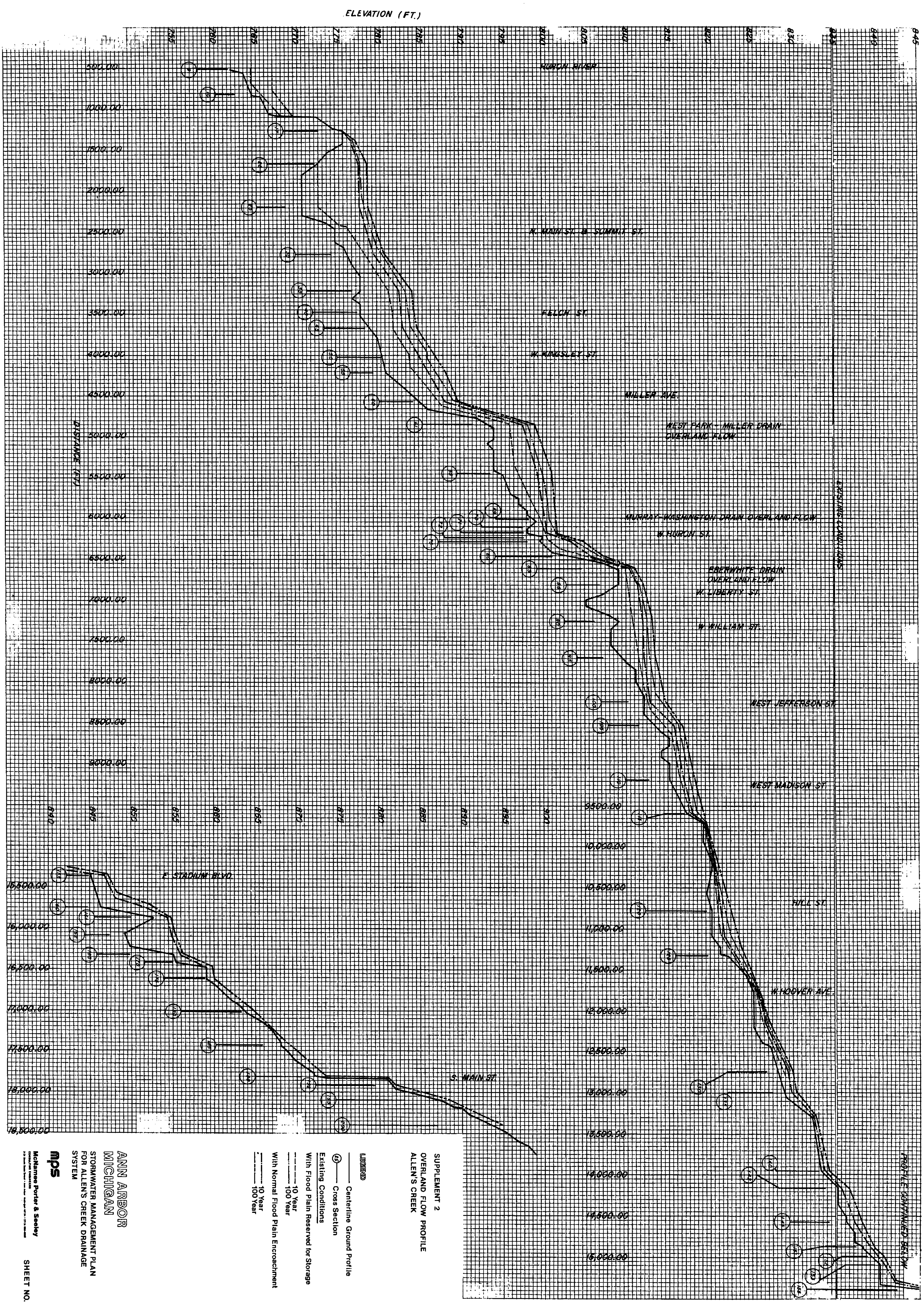
PUBLIC MEETING SIGN-UP SHEET

	Name	Address	Telephone
1.	John Oyer	McNamee, Porter & Seeley	665-6000
2.	Sabah Yousif	City of Ann Arbor	994-2744
3.	Leigh Chizek	City of Ann Arbor	994-2744
4.	John S. Wise	McNamee, Porter & Seeley	665-6000
5.	Vyto Kaunelis	McNamee, Porter & Seeley	665-6000
6.	Vic Cooperwasser	McNamee, Porter & Seeley	665-6000
7.	Thomas E. Bletcher	Harmon Culhane	663-6772/8005
8.	L. A. Manpun	3215 Ramsey Dr.	663-0238
9.	Phillip E. Omala	2418 Pittsfield Blvd.	973-6498
10.	Mrs. Russell J. Burns	406 Maple Ridge	668-8409
11.	Lou Velker	454 5th St.	995-2141
12.	Cindy Armstrong	1109 E. University	769-4252
13.	Gary Skrel	1203 Prescott	995-2466
14.	Marcia Dorsey	Huron River Watershed Council	769-5123
15.	Ethel Potts	1014 Elder	662-3833
16.	Grace Shackman	515 Soule	662-2187
17.	Eugene Katz	Argo Development Corp.	994-6698
18.	Barbara Hall	342 Mulholland	668-7558
19.	Tom Blessing	625 Fountain St.	665-7067
20.	Suresh Sangal	McNamee, Porter & Seeley	665-6000

SUPPLEMENT 2

FLOOD PLAIN DELINEATION MAPS

AND FLOOD PROFILES



SUPPLEMENT 2
OVERLAND FLOW PROFILE
ALLEN'S CREEK

LEGEND

——— Centerline Ground Profile
(45)—— Cross Section
Existing Conditions
With Flood Plain Reserved for Storage
– – – – 10 Year
—·—·— 100 Year
With Normal Flood Plain Encroachment
—··—··— 10 Year
——— 100 Year

ANN ARBOR
MICHIGAN
STORMWATER MANAGEMENT PLAN
FOR ALLEN'S CREEK DRAINAGE
SYSTEM

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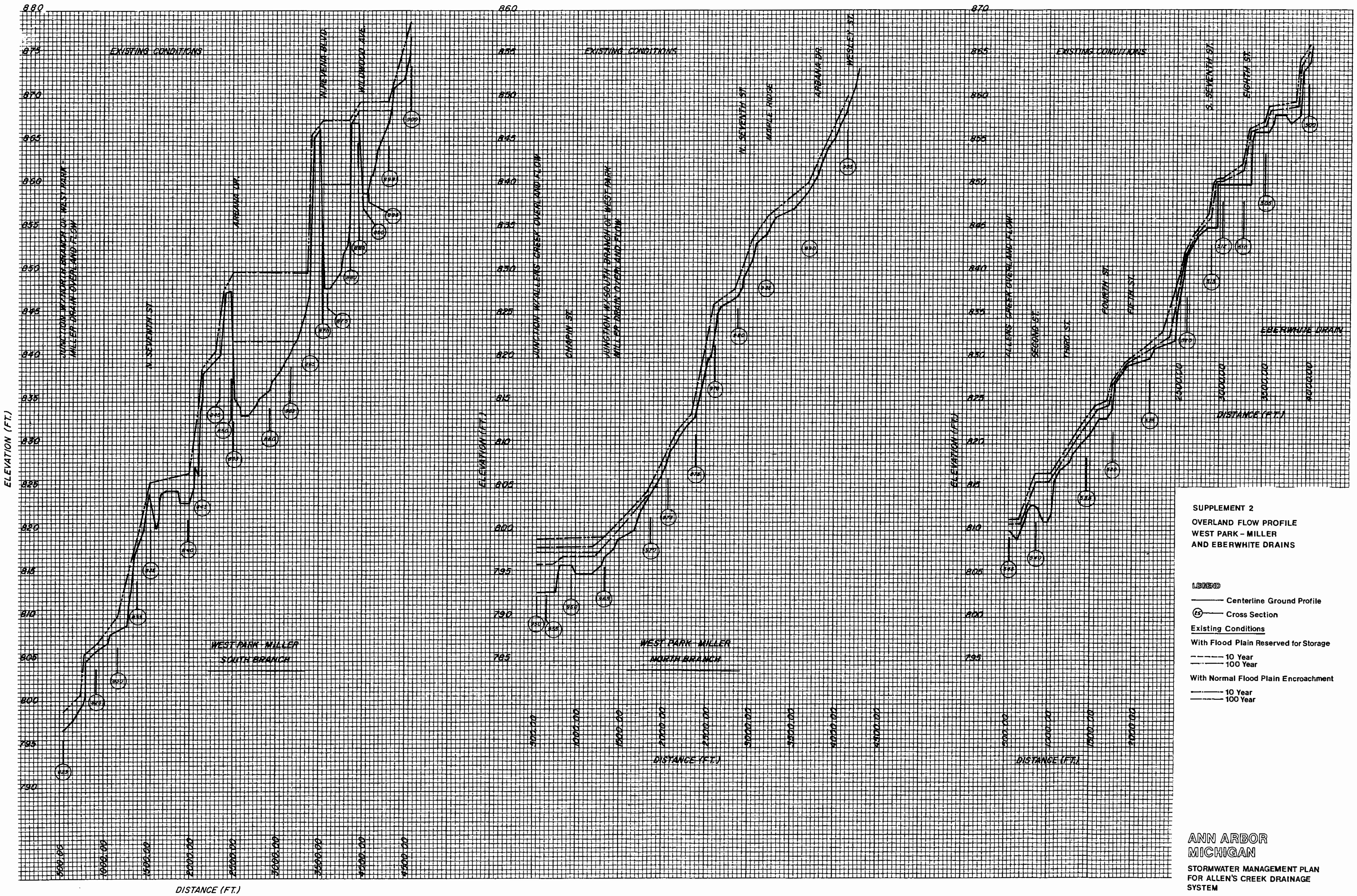
SHEET NO. 1



SUPPLEMENT 2

OVERLAND FLOW PROFILE
WEST PARK – MILLER
AND EBERWHITE DRAINS

LEGEND

——— Centerline Ground Profile

(85)——— Cross Section

Existing Conditions

With Flood Plain Reserved for Storage

- - - - - 10 Year
—·—·— 100 Year

With Normal Flood Plain Encroachment

—·—·— 10 Year
— — — 100 Year

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FOR ALLEN'S CREEK DRAINAGE
SYSTEM

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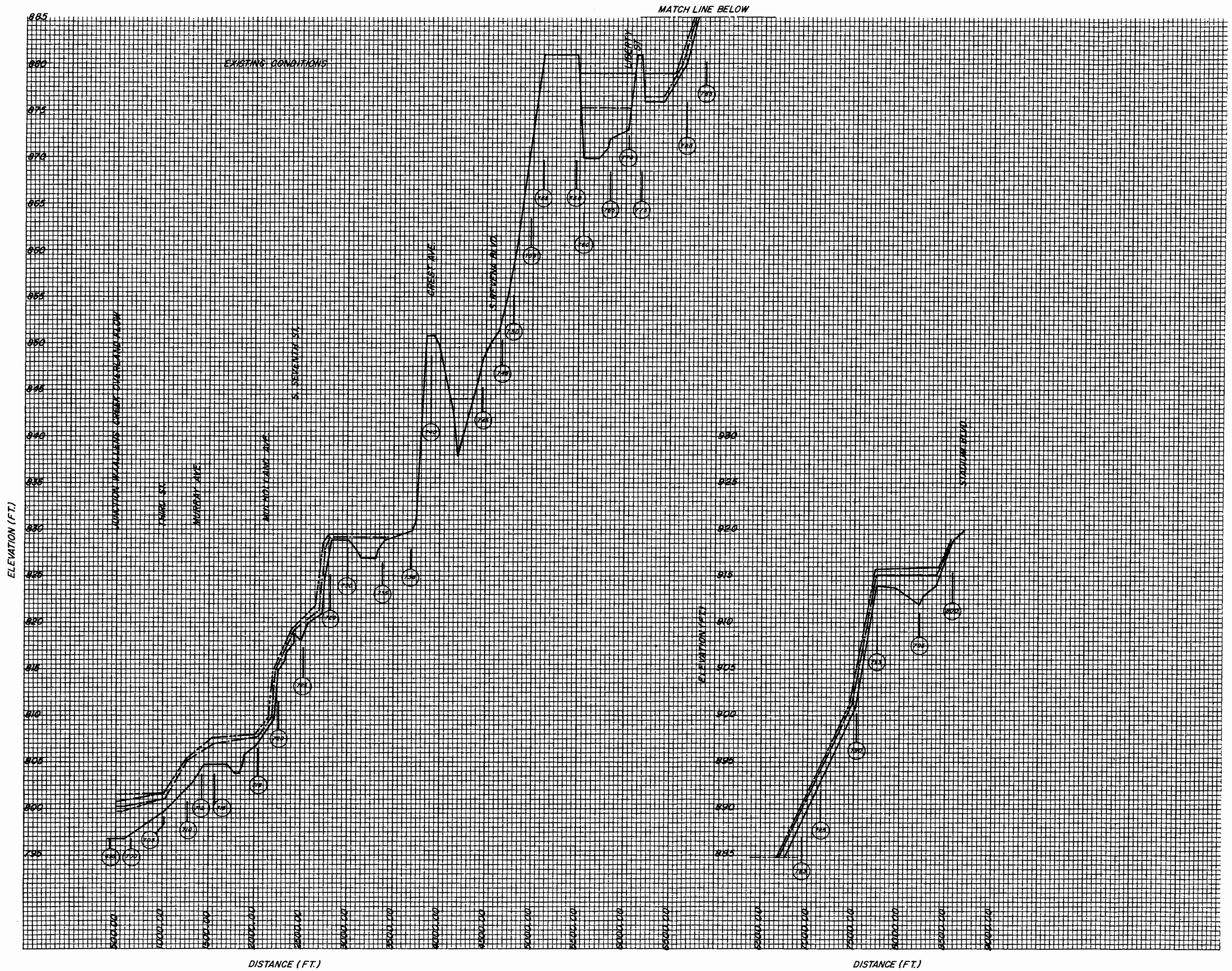
SHEET NO. 2



SUPPLEMENT 2

OVERLAND FLOW PROFILE
MURRAY-WASHINGTON DRAIN

LEGEND

——— Centerline Ground Profile
(#)—— Cross Section

Existing Conditions

With Flood Plain Reserved for Storage
- - - - 10 Year
—·—· 100 Year

With Normal Flood Plain Encroachment
——— 10 Year
——— 100 Year

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STORMWATER MANAGEMENT PLAN
FOR ALLEN'S CREEK DRAINAGE
SYSTEM

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SHEET NO. 3

SUPPLEMENT 2
FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

(2) Cross Section

Existing Conditions
With Flood Plain Reserved For Storage
- - - - 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
——100 Year

0 100 200
SCALE: FEET

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STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

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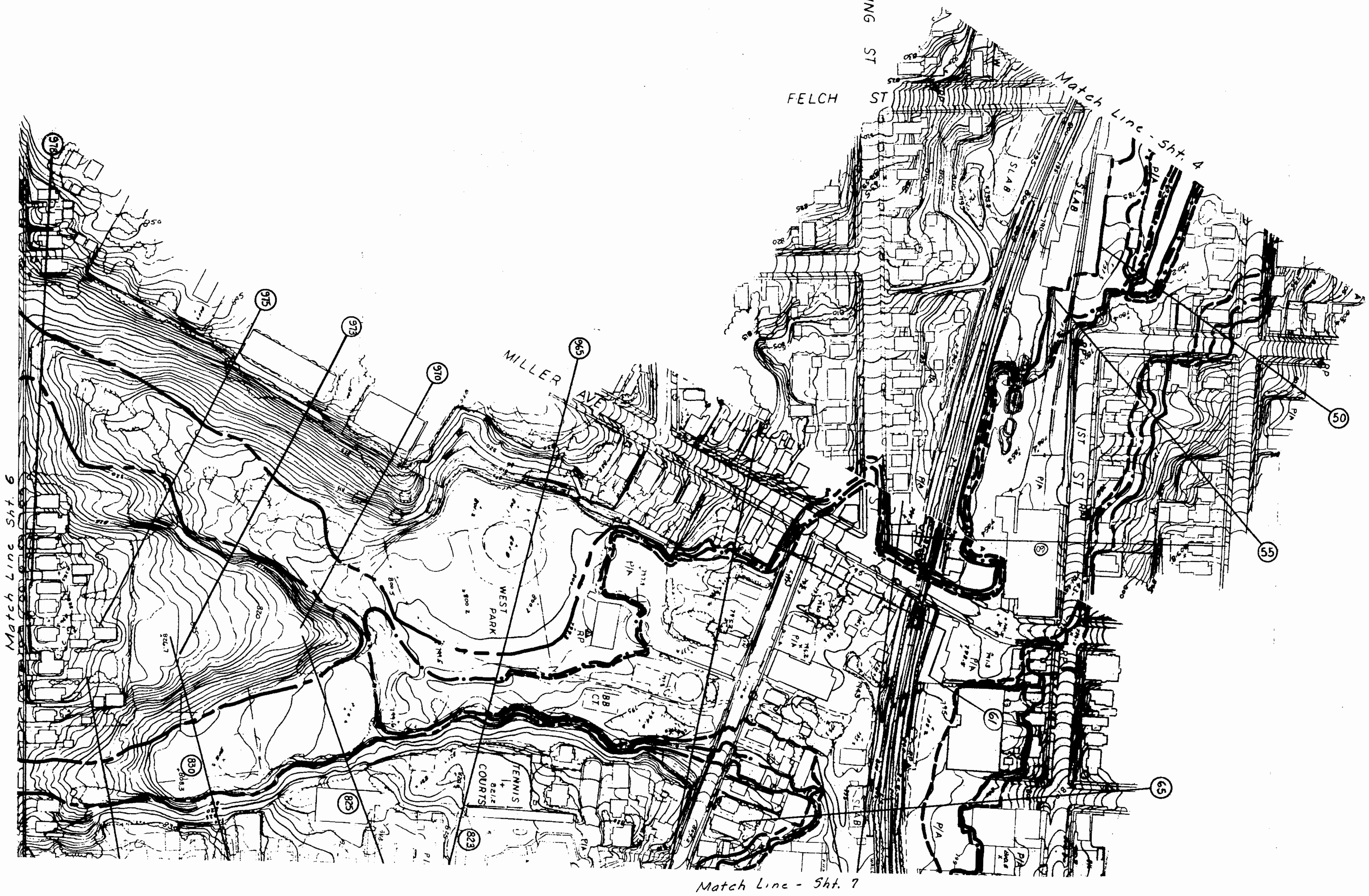
SHEET NO. 4



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—②— Cross Section

Existing Conditions
With Flood Plain Reserved For Storage
– – – – 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
—·–·— 100 Year

0 100 200
SCALE: FEET

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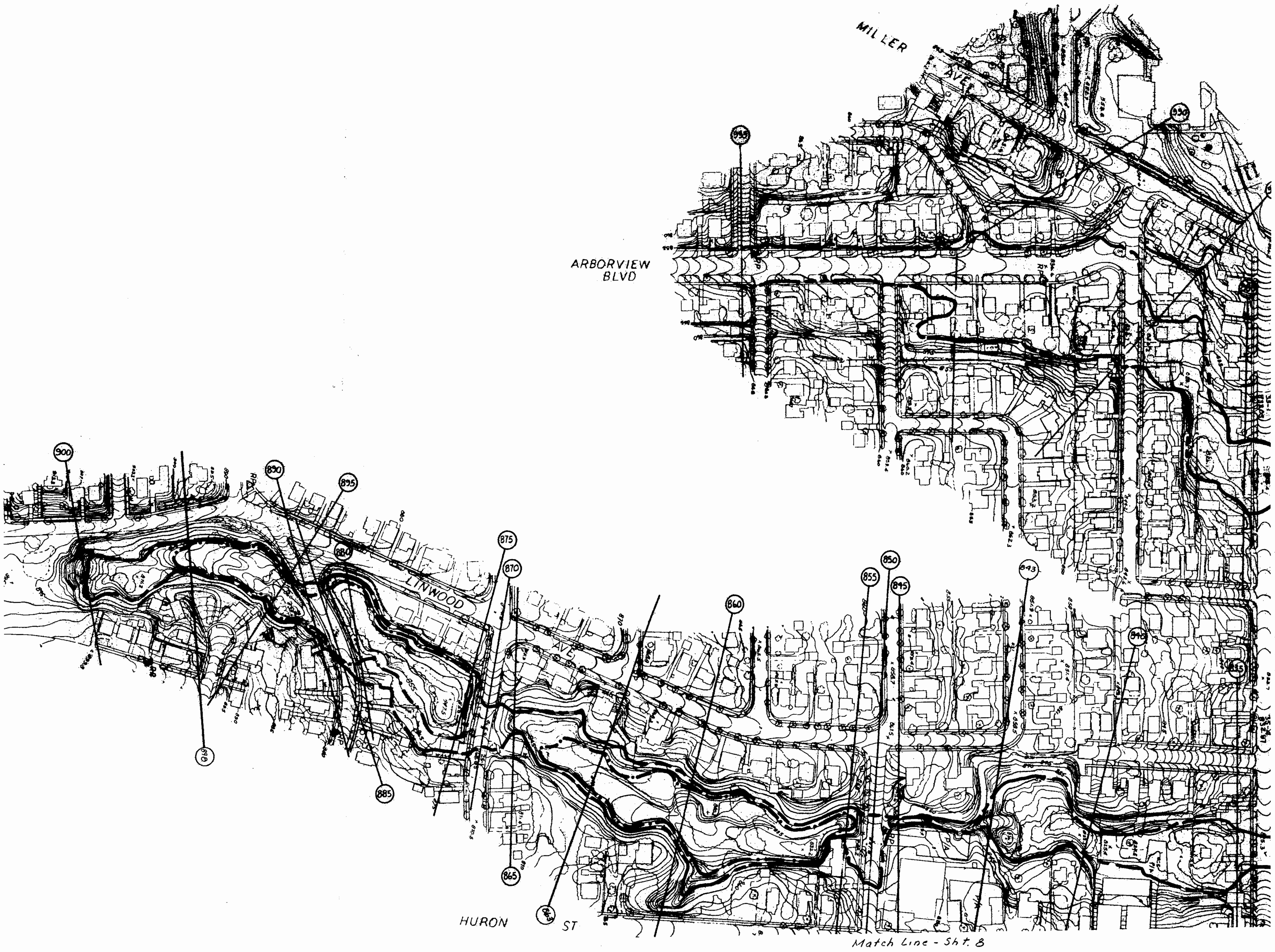
STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps
McNamee Porter & Seeley
CONSULTING ENGINEERS

SHEET NO. 5



SUPPLEMENT 2
FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—②— Cross Section

Existing Conditions
With Flood Plain Reserved For Storage
- - - - 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
-----100 Year

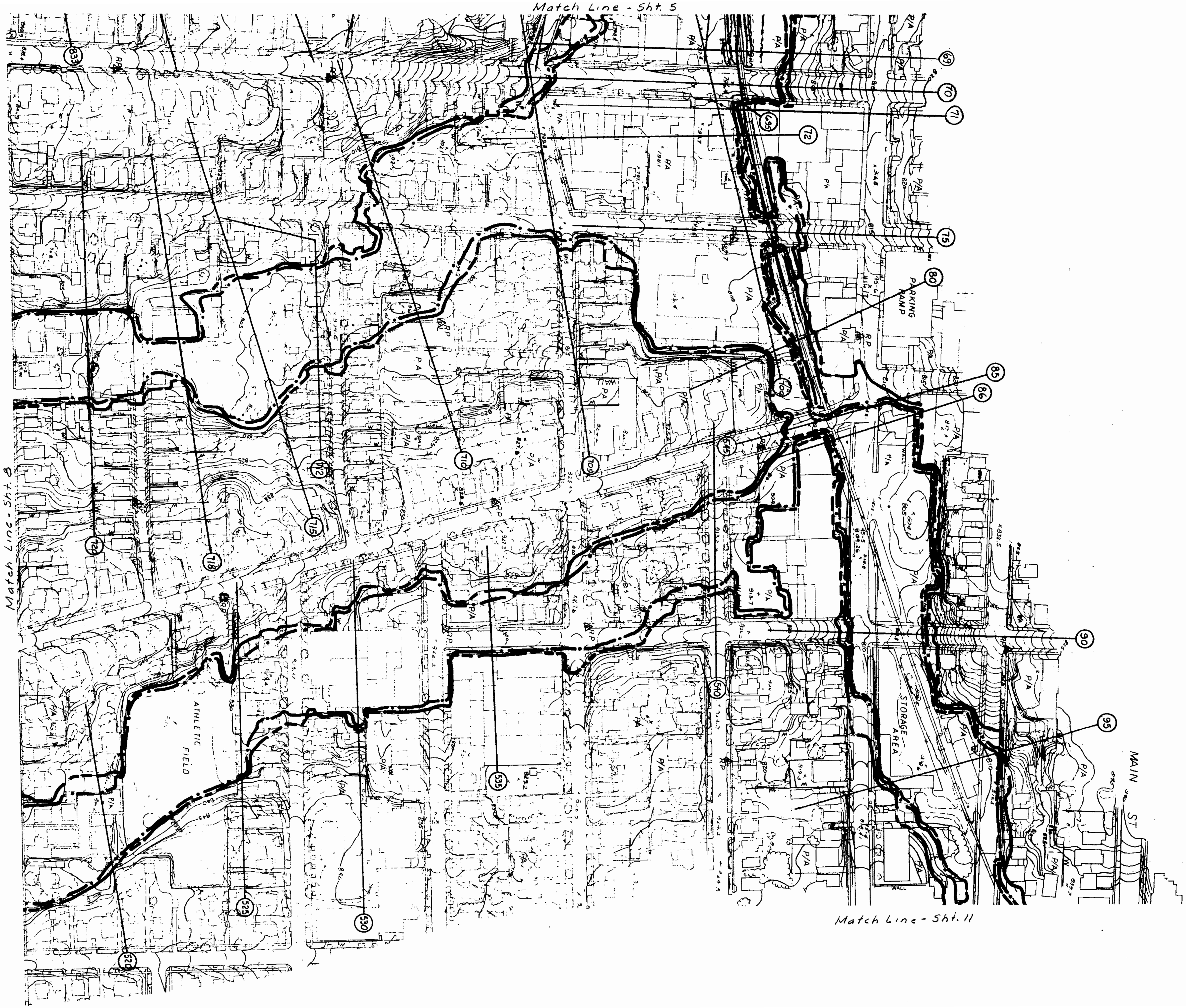
0 100 200
SCALE: FEET

ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps
McNamee Porter & Seeley
CONSULTING ENGINEERS

SHEET NO. 6



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—② Cross Section

Existing Conditions

With Flood Plain Reserved For Storage
- - - - 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
—··—100 Year

0 100 200

SCALE: FEET

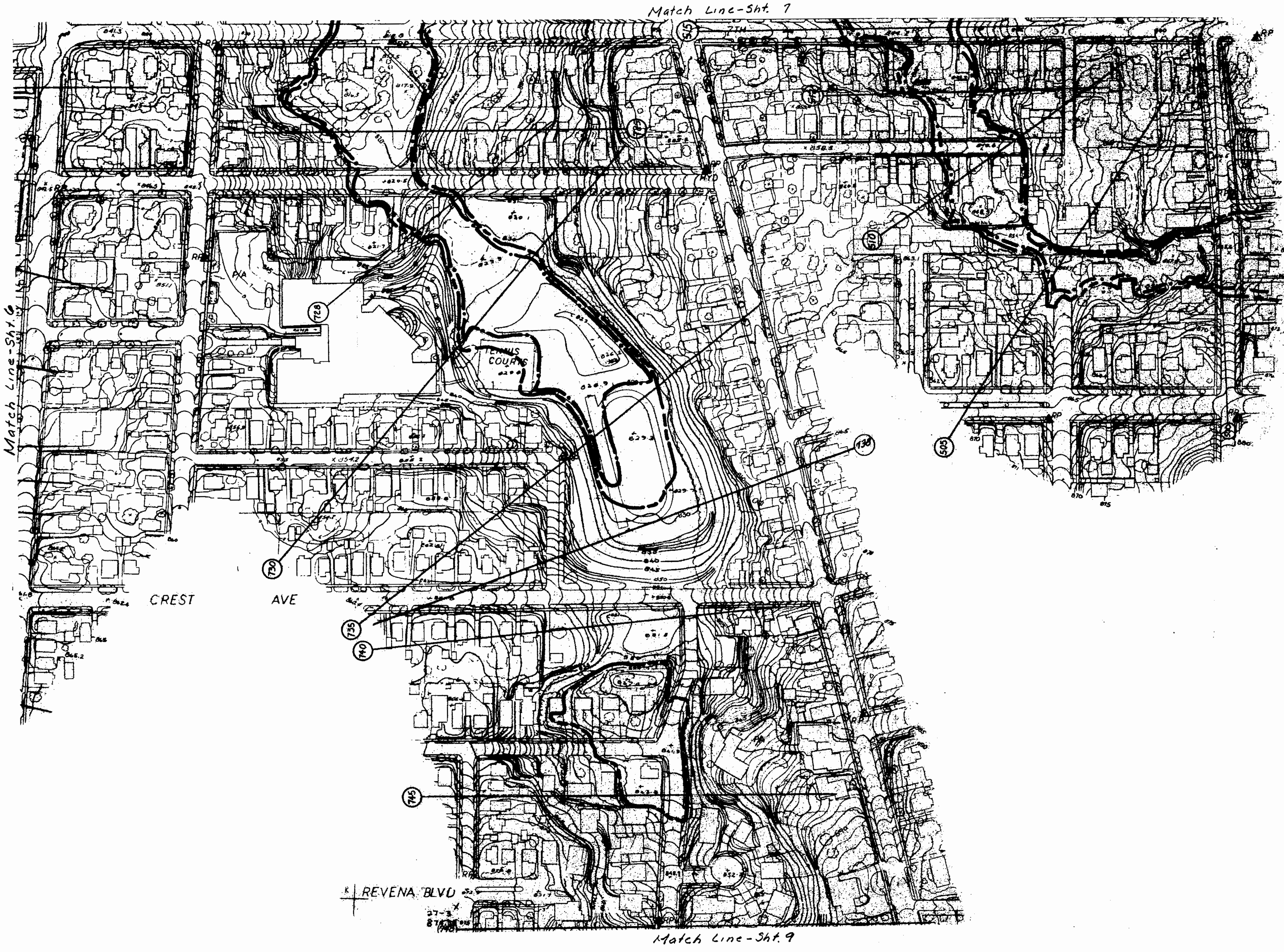
ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

McNamee Porter & Seeley
CONSULTING ENGINEERS

SHEET NO. 7



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—②— Cross Section

Existing Conditions
With Flood Plain Reserved For Storage
– – – – 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
—·–·— 100 Year

0 100 200
SCALE: FEET

ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

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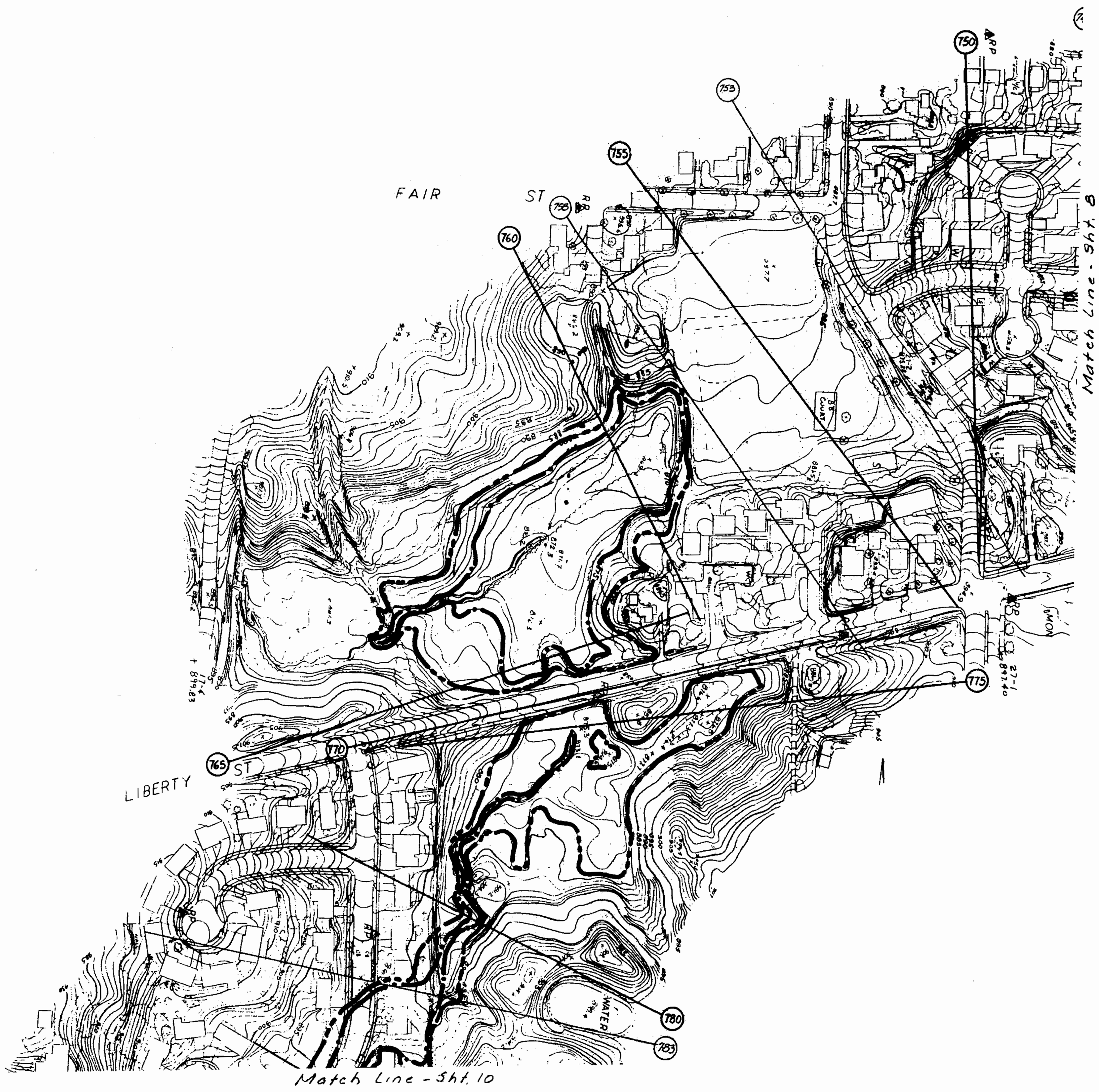
CONSULTING ENGINEERS

SHEET NO. 8



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

(2) Cross Section

Existing Conditions

With Flood Plain Reserved For Storage
- - - - 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
—·-·— 100 Year

0 100 200

SCALE: FEET

ANN ARBOR
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STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

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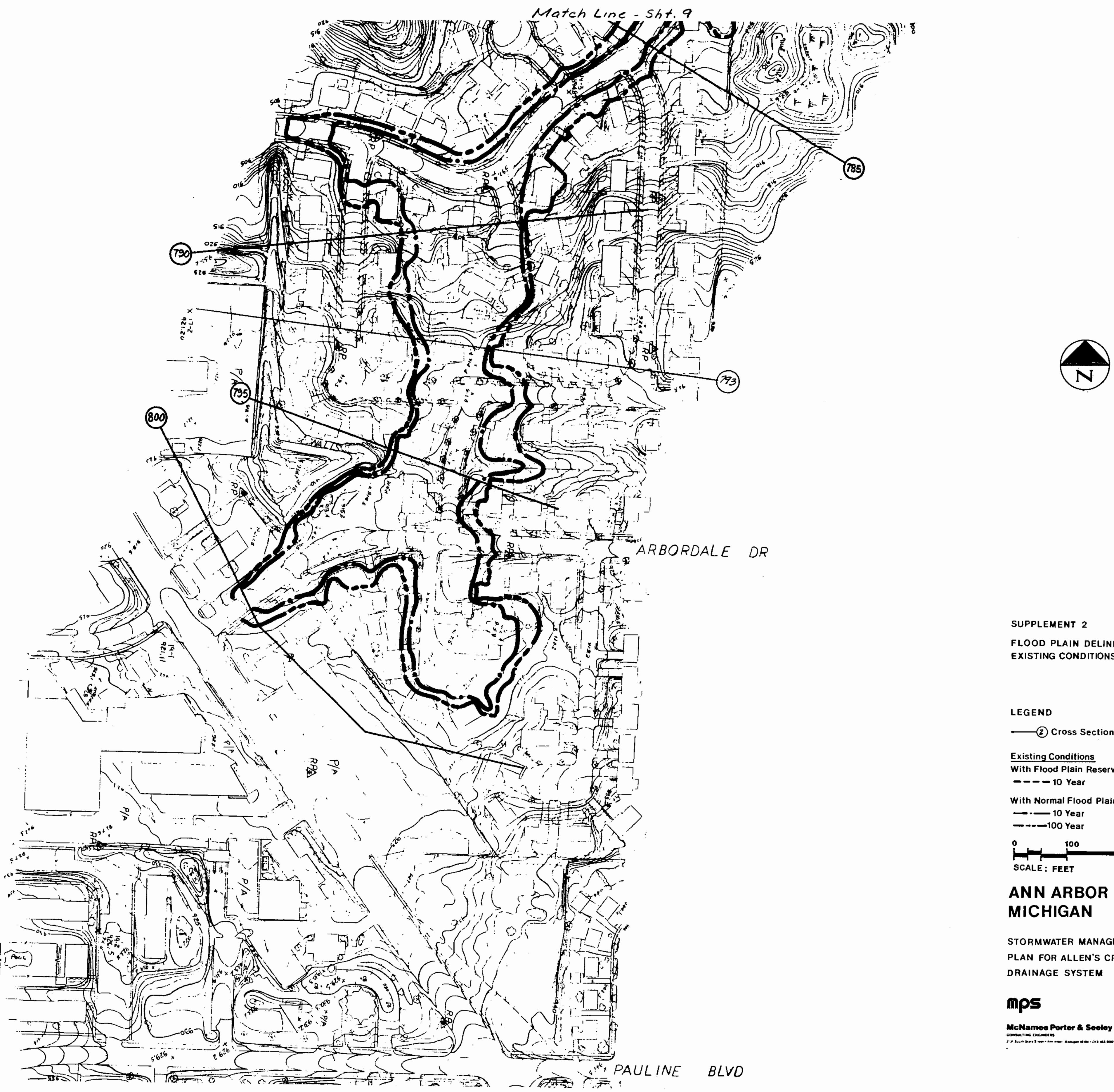
SHEET NO. 9



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—② Cross Section

Existing Conditions

With Flood Plain Reserved For Storage
- - - - 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
—··—100 Year

0 100 200
SCALE: FEET

ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

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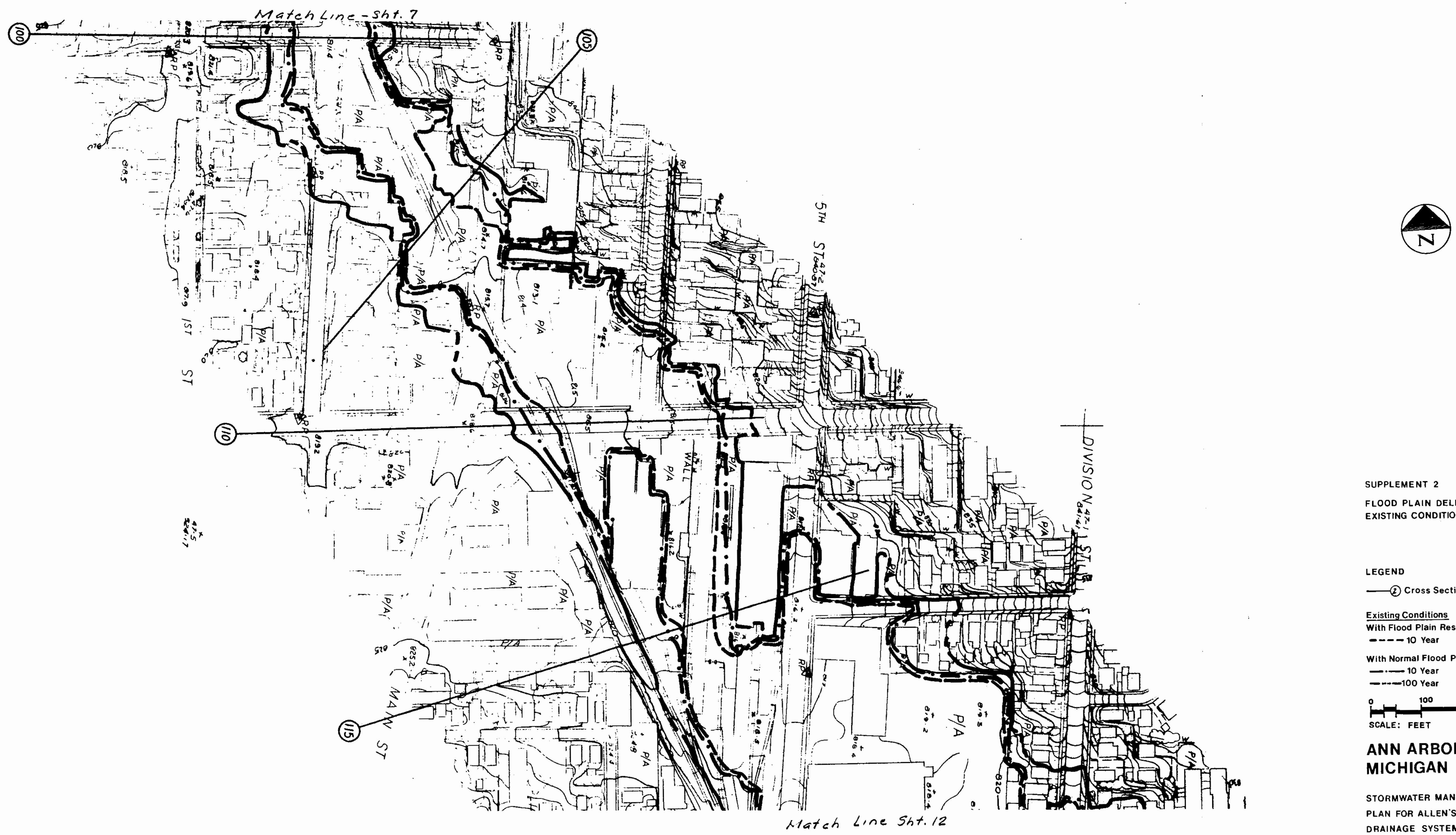
SHEET NO. 10



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—(2) Cross Section

Existing Conditions
With Flood Plain Reserved For Storage
– – – – 10 Year

With Normal Flood Plain Encroachment
—·—·— 10 Year
—·–·–·100 Year

0 100 200
SCALE: FEET

ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

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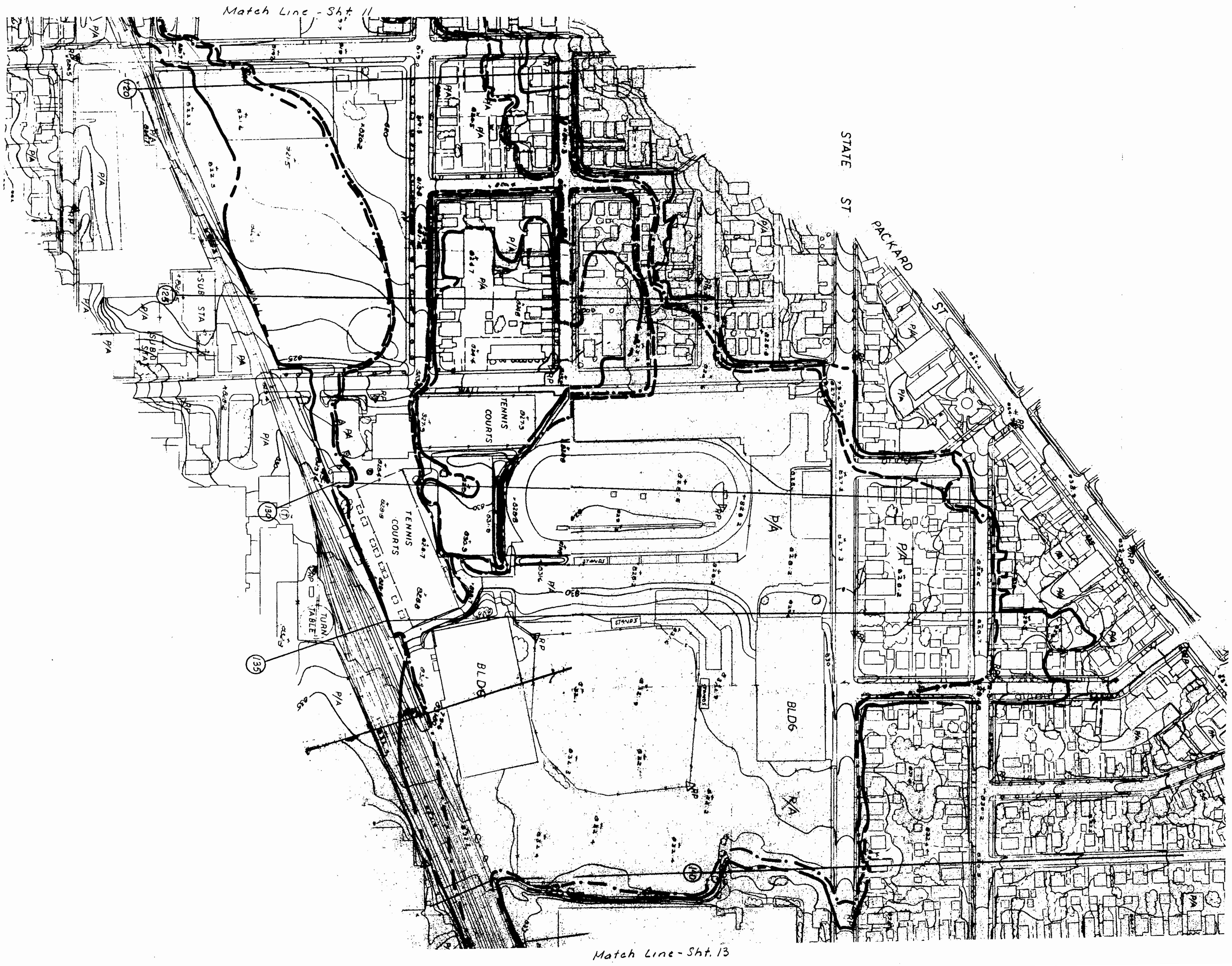
SHEET NO. 11



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—②— Cross Section

Existing Conditions

With Flood Plain Reserved For Storage
– – – – 10 Year

With Normal Flood Plain Encroachment
—·—·— 10 Year
–·–·–·100 Year

0 100 200
SCALE: FEET

ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

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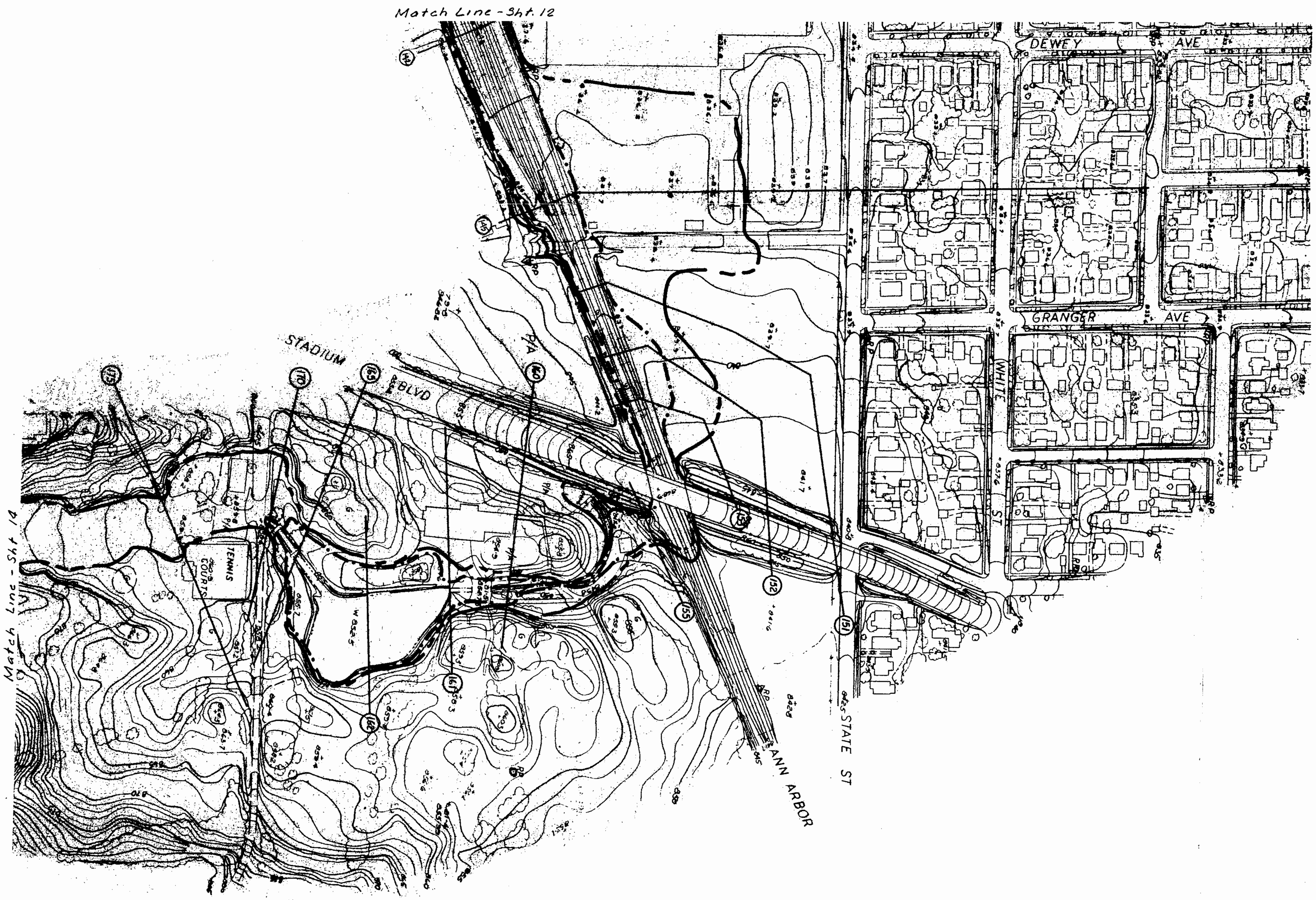
CONSULTING ENGINEERS

SHEET NO. 12



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

Cross Section

Existing Conditions
With Flood Plain Reserved For Storage
- - - - 10 Year

With Normal Flood Plain Encroachment
—·—·— 10 Year
- - - —100 Year

0 100 200
SCALE: FEET

ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

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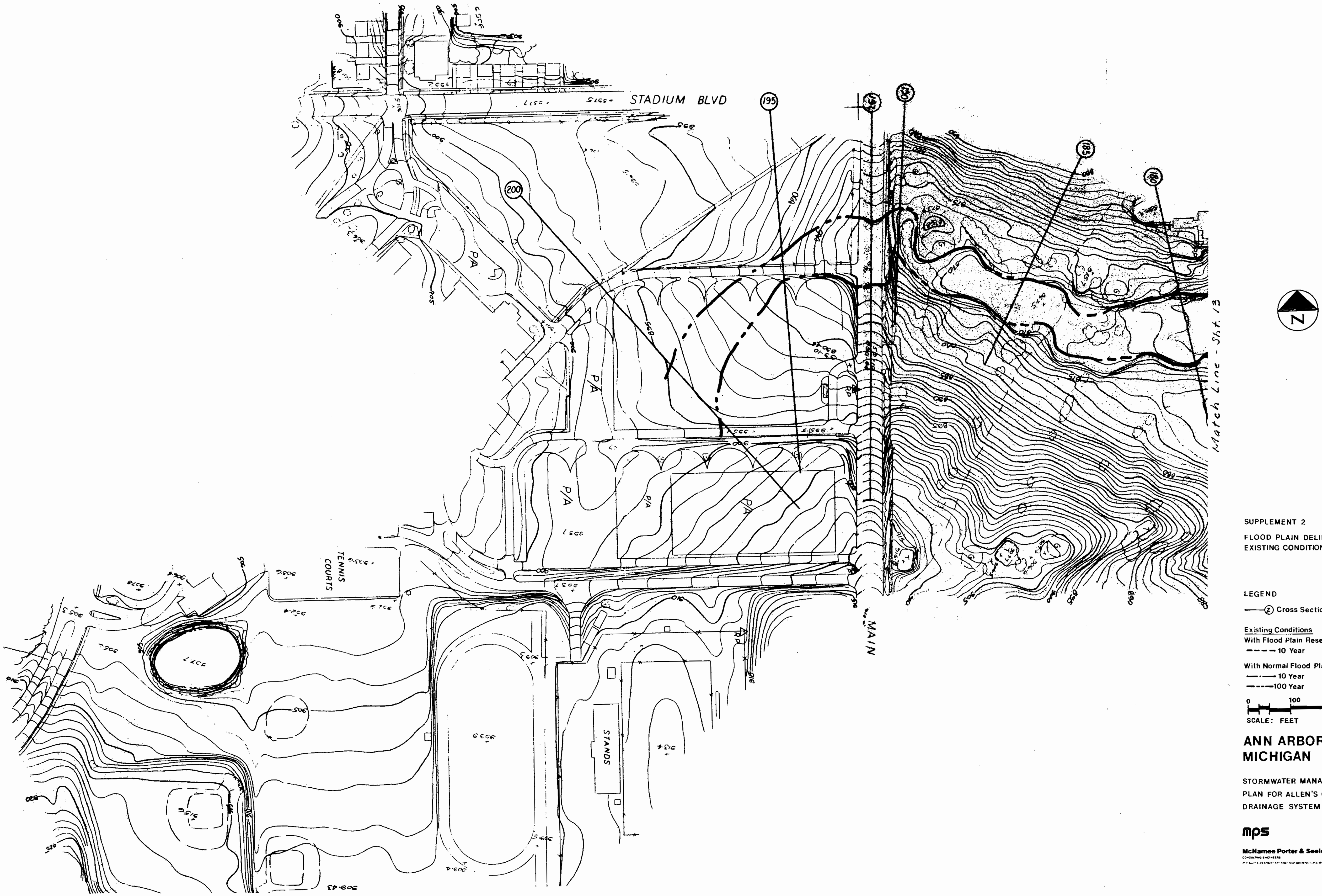
SHEET NO. 13



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
EXISTING CONDITIONS

LEGEND

—② Cross Section

Existing Conditions
With Flood Plain Reserved For Storage
- - - - 10 Year

With Normal Flood Plain Encroachment
—·— 10 Year
—·—·—100 Year

0 100 200
SCALE: FEET

ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

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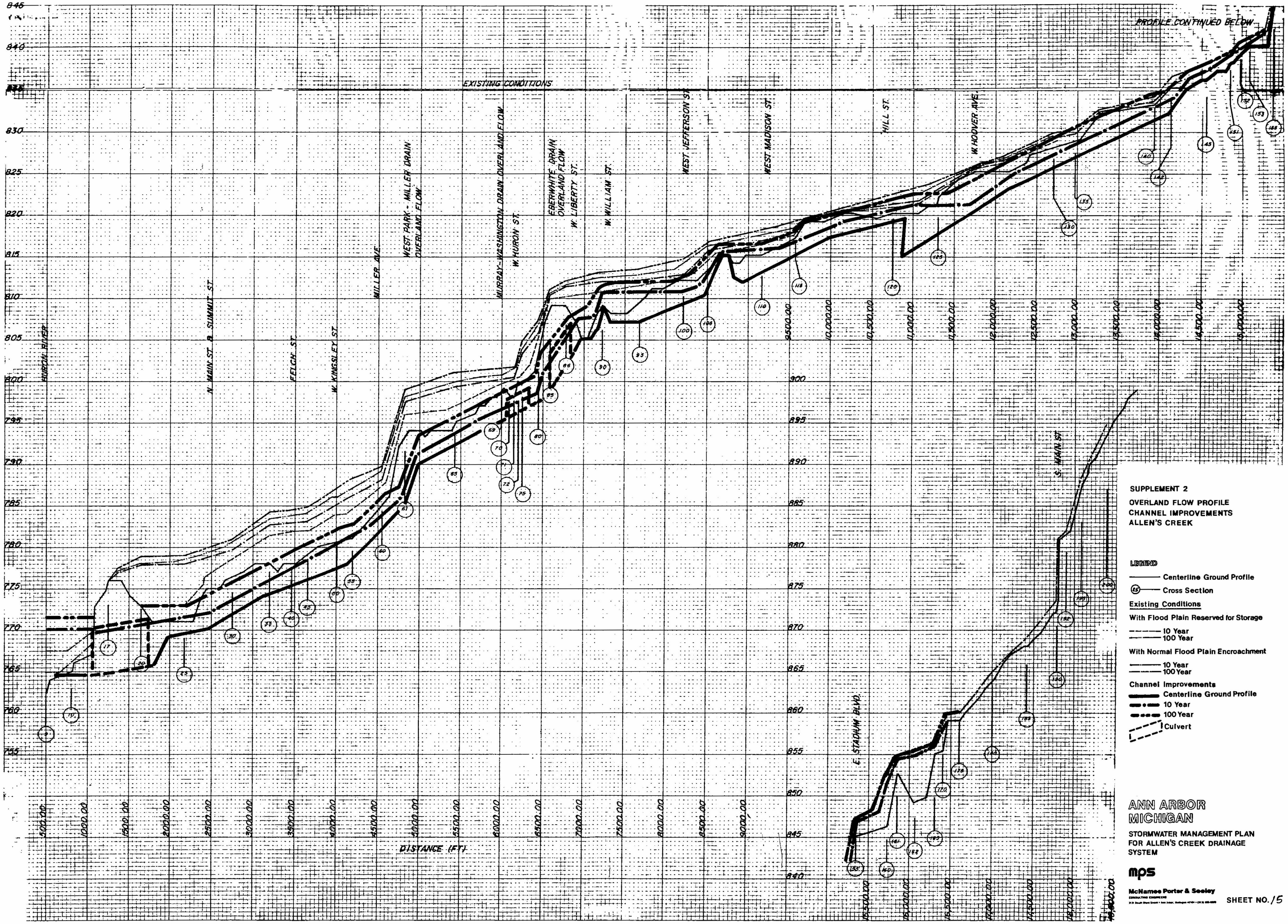
SHEET NO. 14



SUPPLEMENT 2

OVERLAND FLOW PROFILE
CHANNEL IMPROVEMENTS
ALLEN'S CREEK

LEGEND

Centerline Ground Profile
Cross Section

Existing Conditions

With Flood Plain Reserved for Storage
- 10 Year
- 100 Year

With Normal Flood Plain Encroachment
- 10 Year
- 100 Year

Channel Improvements
- Centerline Ground Profile
- 10 Year
- 100 Year
- Culvert

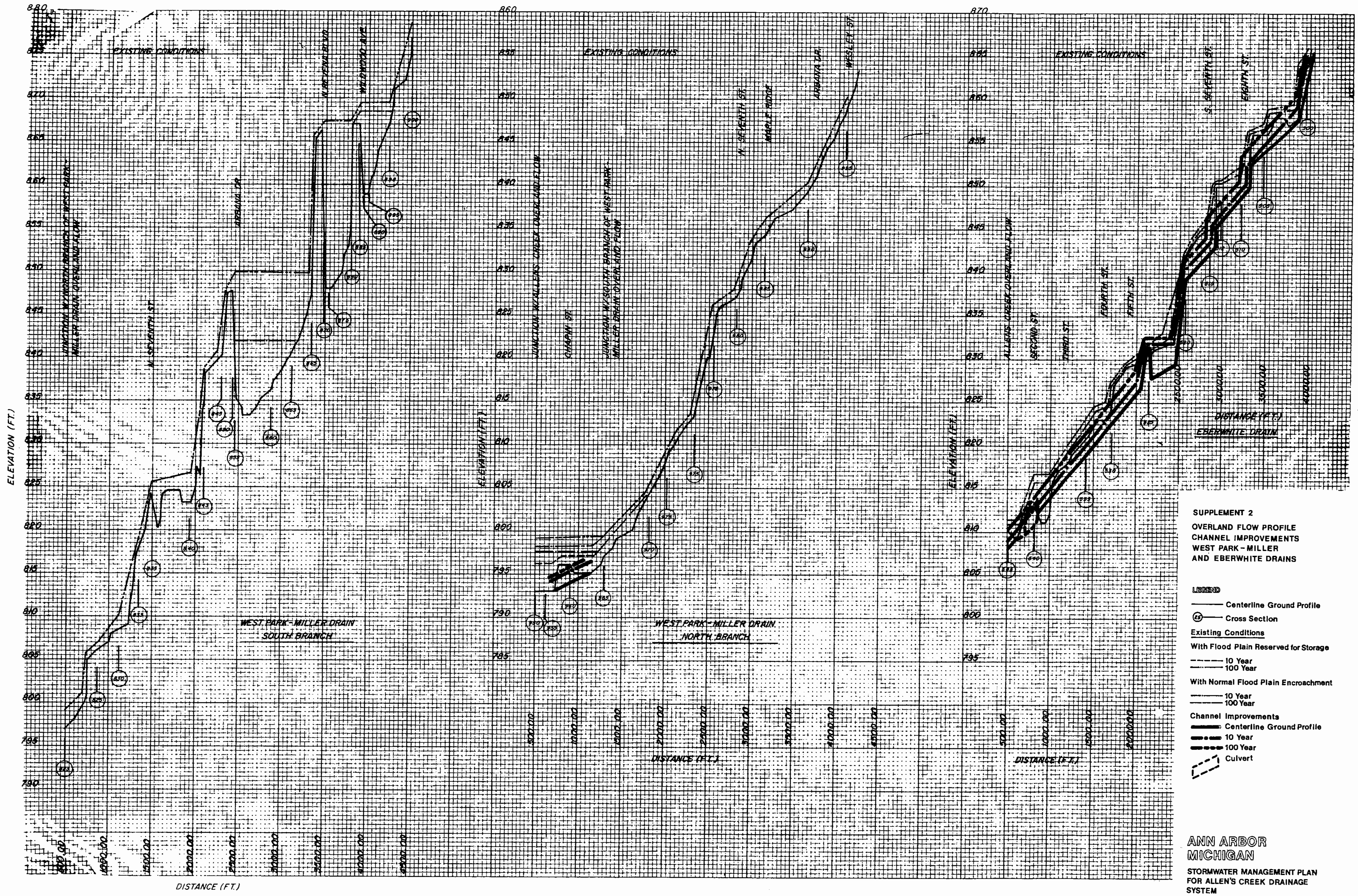
ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT PLAN
FOR ALLEN'S CREEK DRAINAGE
SYSTEM

mps

McNamee Porter & Seeley
CONSULTING ENGINEERS

SHEET NO. 15



SUPPLEMENT 2

OVERLAND FLOW PROFILE
CHANNEL IMPROVEMENTS
WEST PARK-MILLER
AND EBERWHITE DRAINS

LEGEND

Centerline Ground Profile
Cross Section

Existing Conditions

With Flood Plain Reserved for Storage
- 10 Year
- 100 Year

With Normal Flood Plain Encroachment
- 10 Year
- 100 Year

Channel Improvements
- Centerline Ground Profile
- 10 Year
- 100 Year
- Culvert

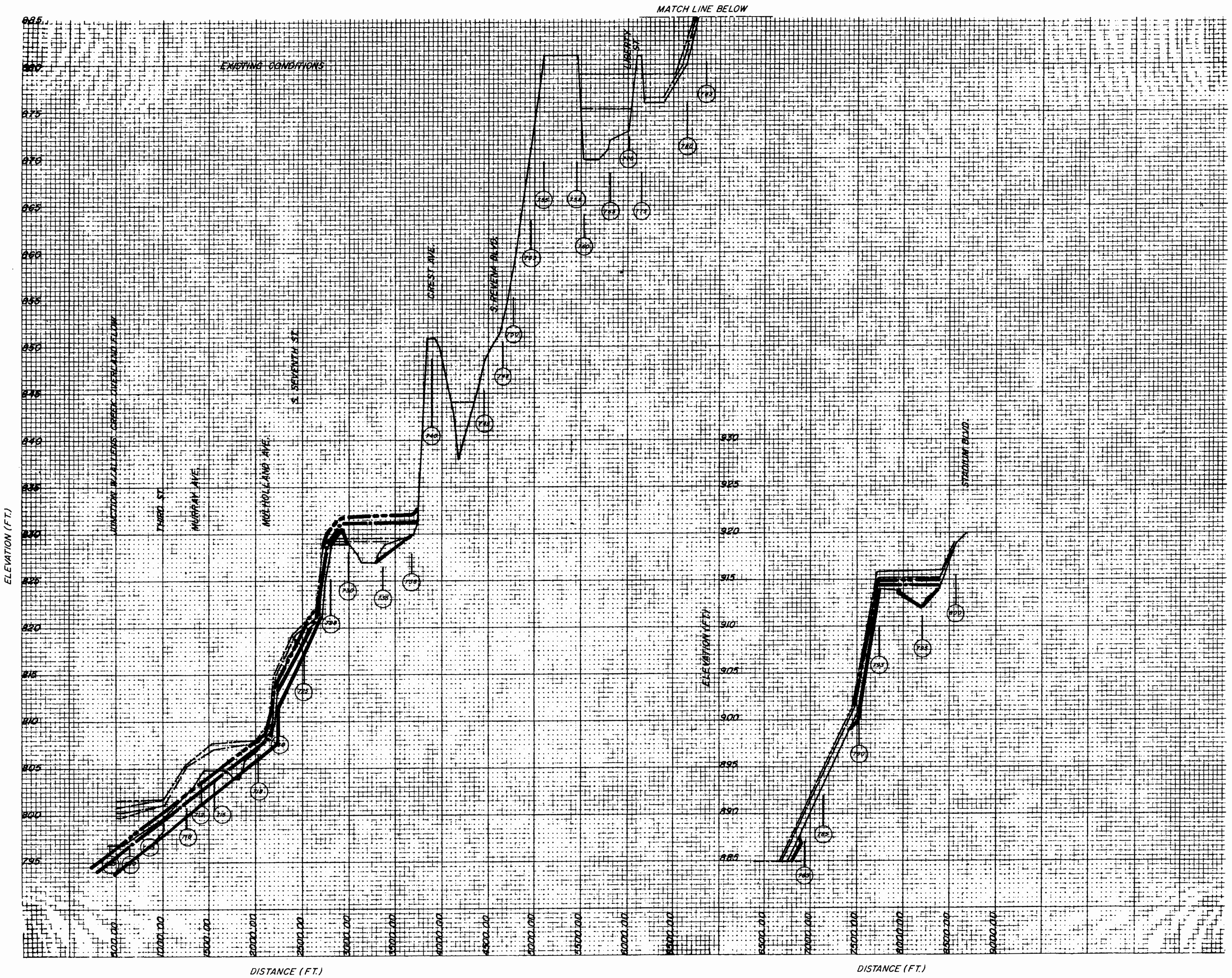
ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT PLAN
FOR ALLEN'S CREEK DRAINAGE
SYSTEM

mps

McNamee Porter & Seeley

SHEET NO. 16



SUPPLEMENT 2

OVERLAND FLOW PROFILE
CHANNEL IMPROVEMENTS
MURRAY-WASHINGTON DRAIN

LEGEND

——— Centerline Ground Profile

Ⓔ——— Cross Section

Existing Conditions

With Flood Plain Reserved for Storage

- - - - 10 Year
—·—·— 100 Year

With Normal Flood Plain Encroachment

——— 10 Year
——— 100 Year

Channel Improvements

▬▬▬ Centerline Ground Profile
▬•▬ 10 Year
▬●▬ 100 Year

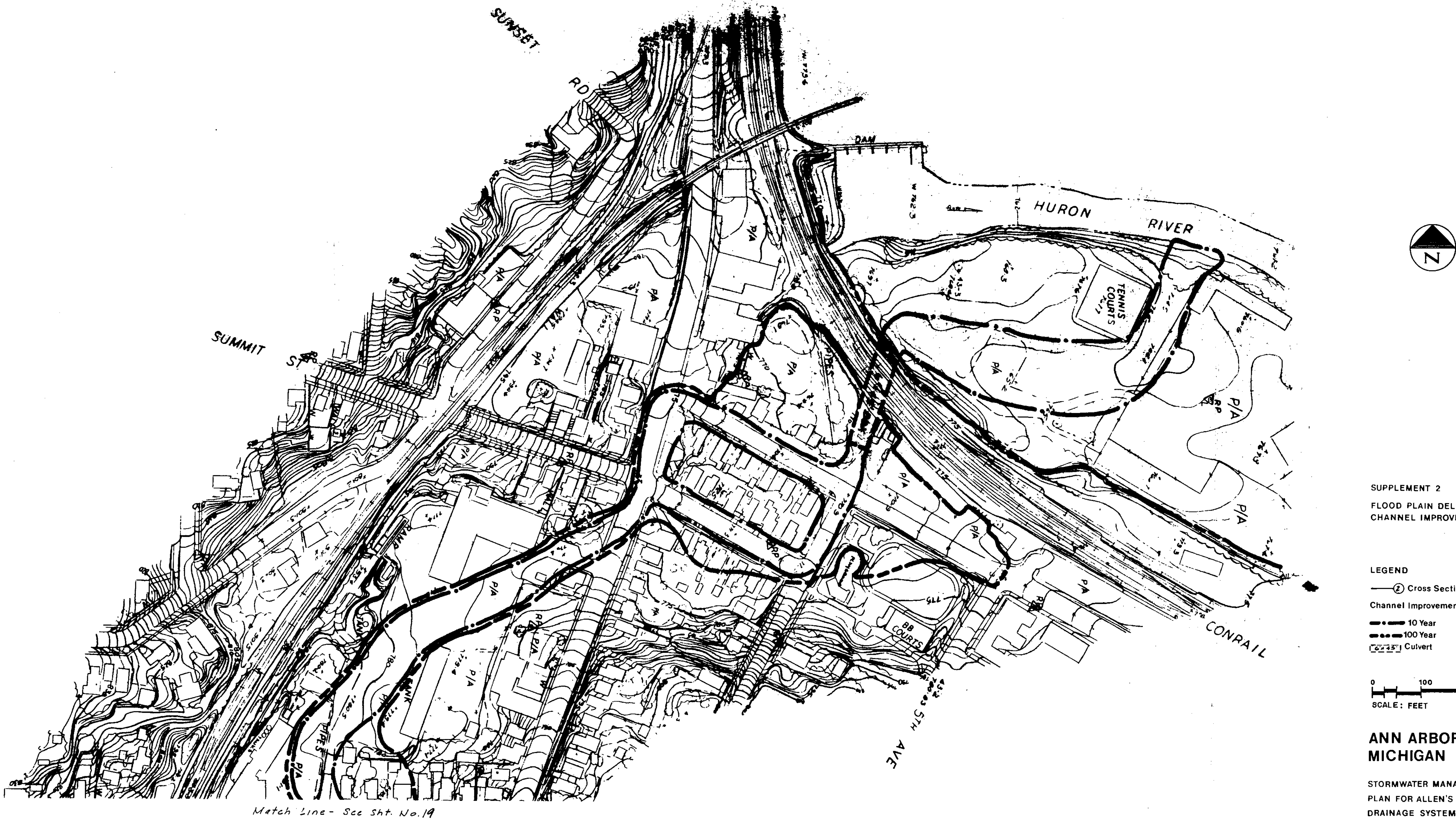
ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT PLAN
FOR ALLEN'S CREEK DRAINAGE
SYSTEM

mps

McNamee Porter & Seeley

SHEET NO 17



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

(2) Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200

SCALE: FEET

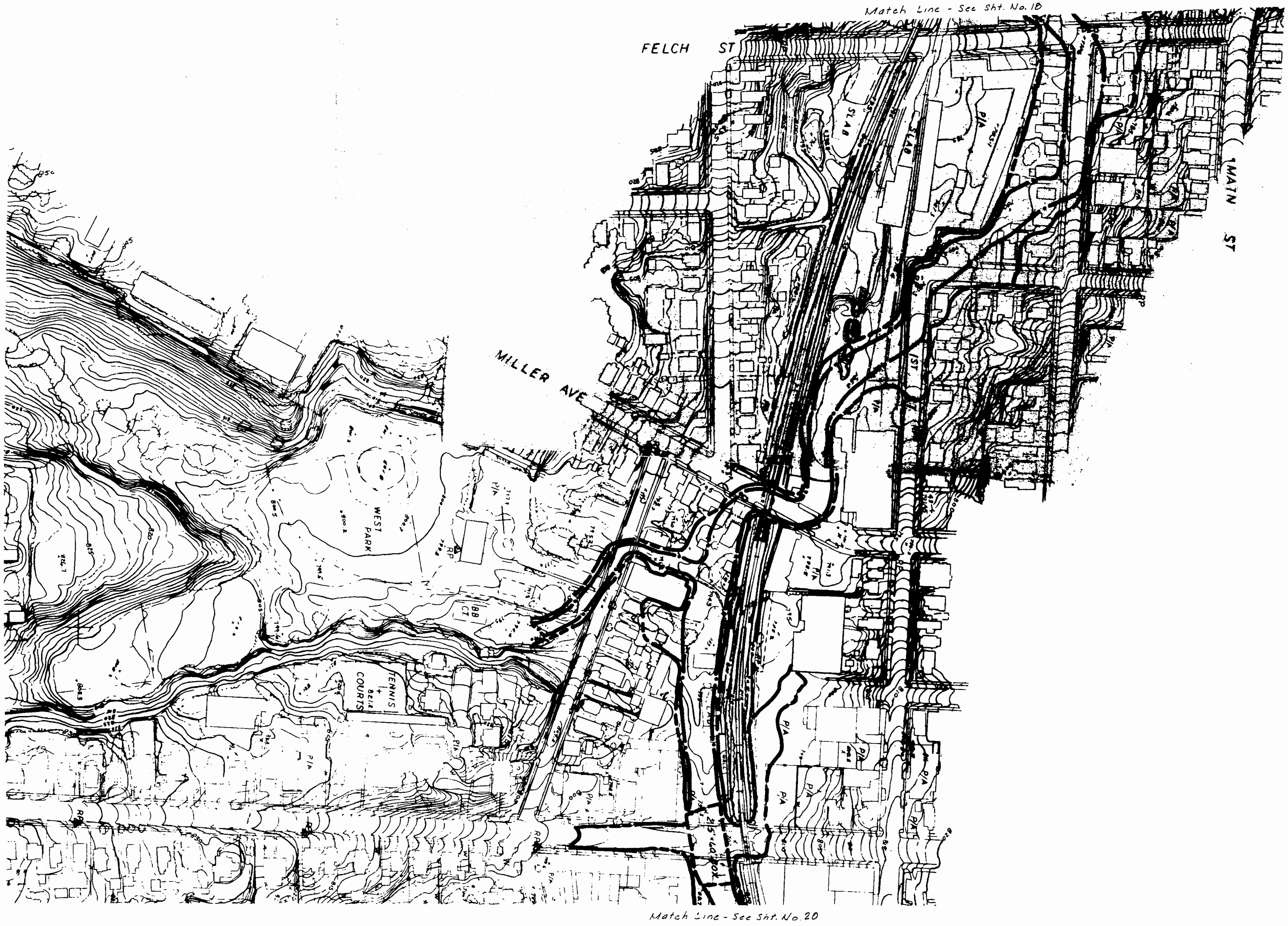
ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

McNamee Porter & Seeley

SHEET NO. 18



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200

SCALE: FEET

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STORMWATER MANAGEMENT PLAN FOR ALLEN'S CREEK DRAINAGE SYSTEM

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McNamee Porter & Seeley

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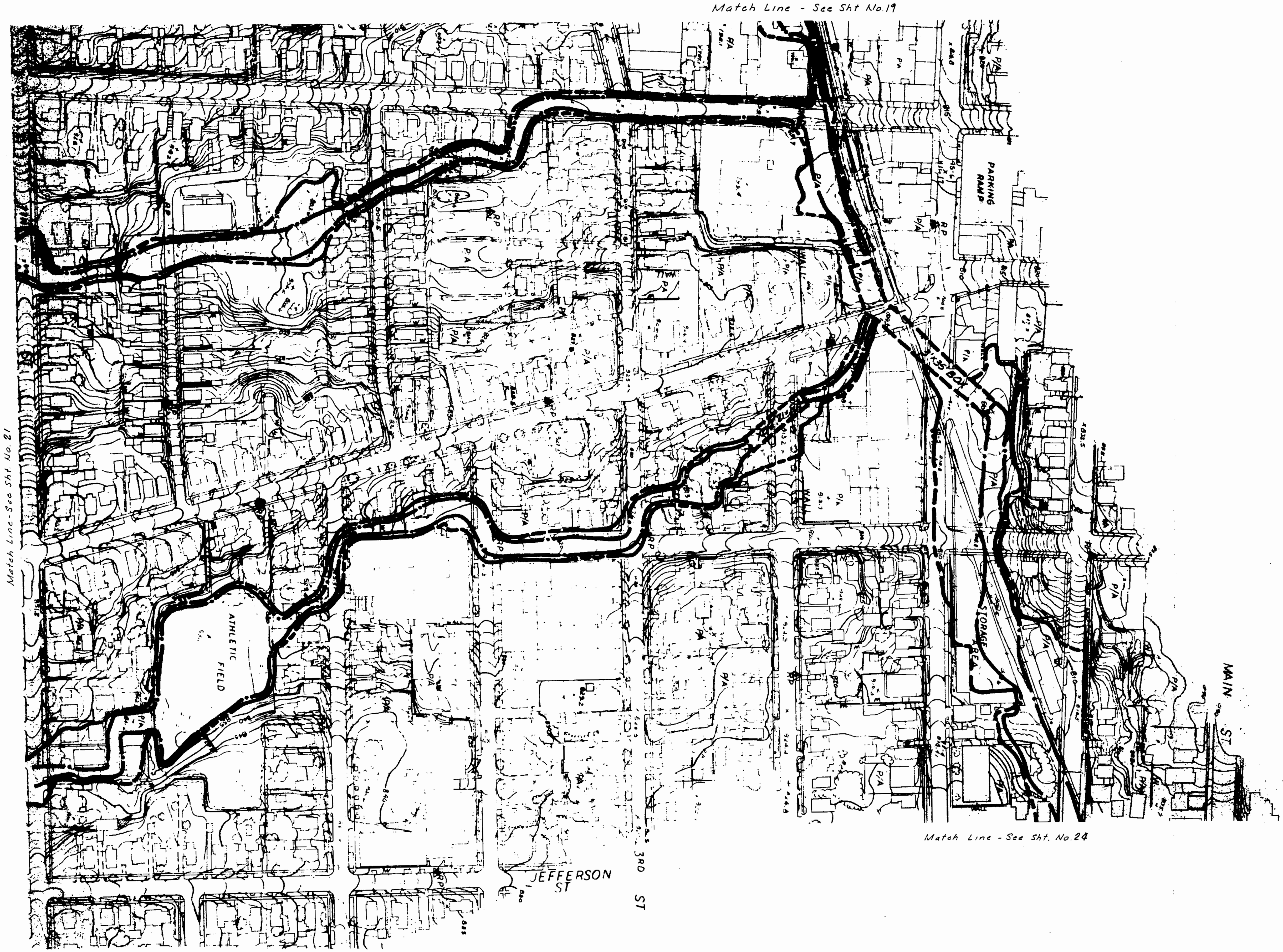
SHEET NO. 19



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

(2) Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200

SCALE: FEET

ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

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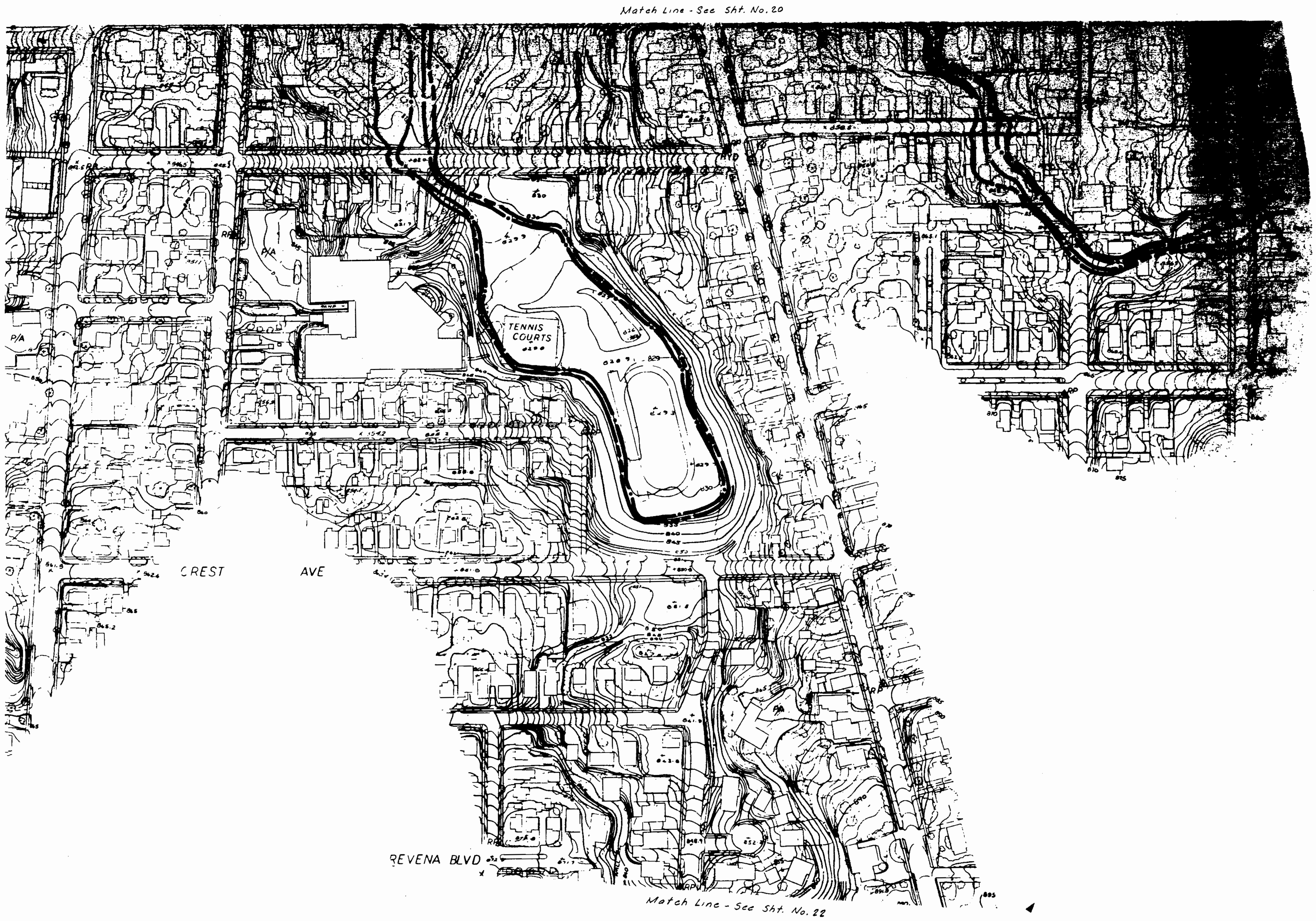
SHEET NO. 20



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

(2) Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200

SCALE: FEET

ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

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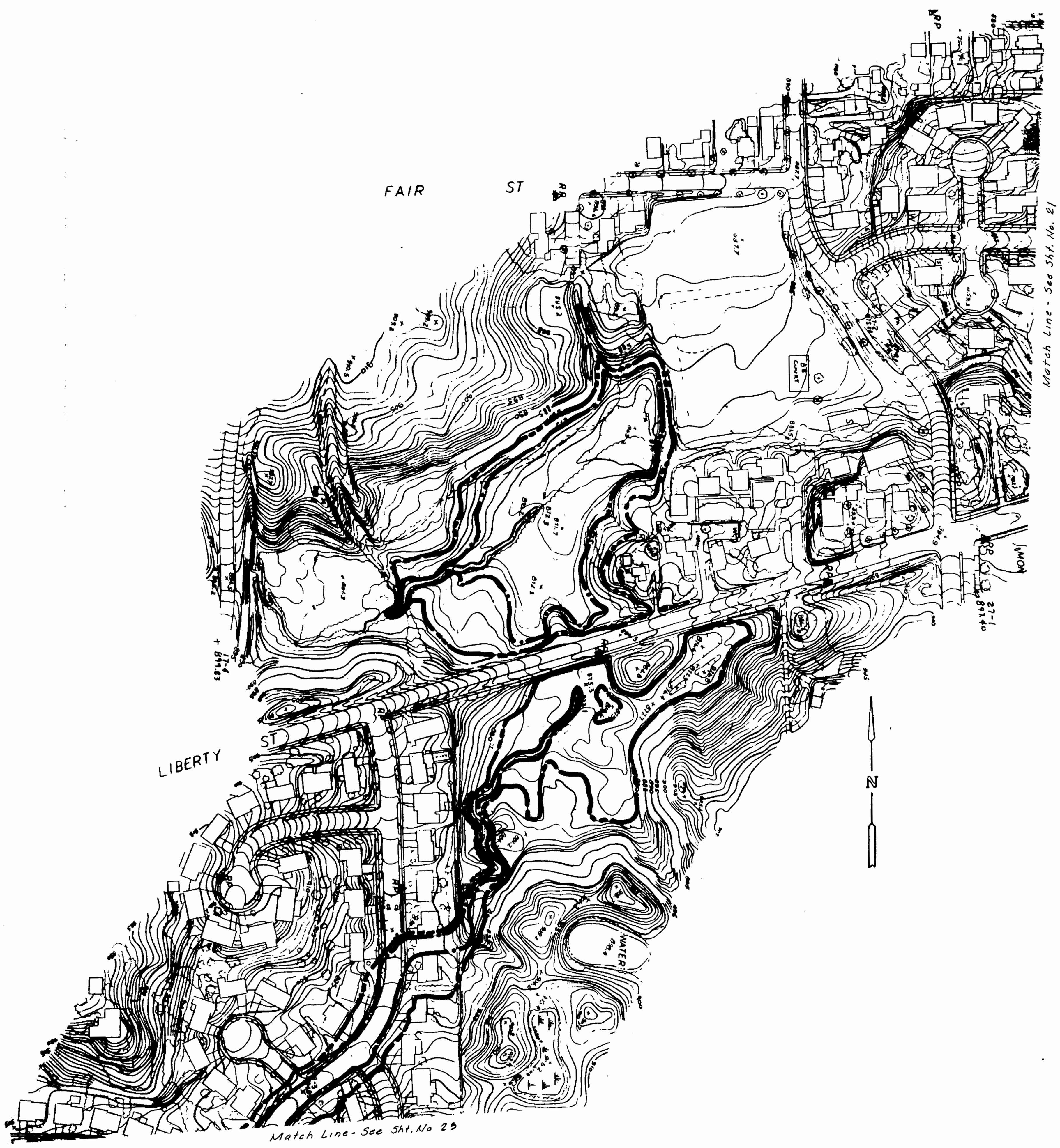
SHEET NO. 21



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

(2) Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200
SCALE: FEET

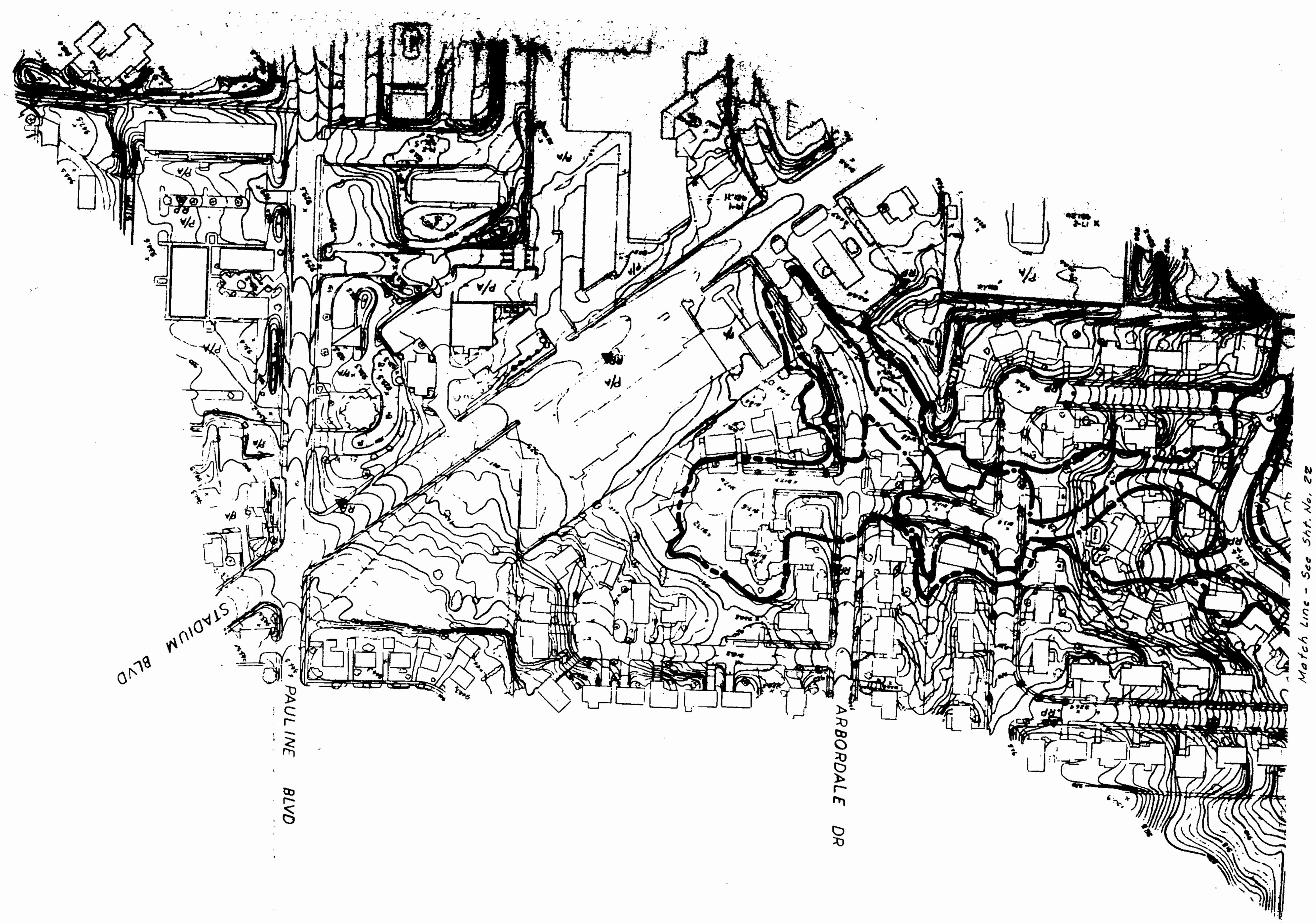
ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT PLAN FOR ALLEN'S CREEK DRAINAGE SYSTEM

mps

McNamee Porter & Seeley
CONSULTING ENGINEERS

SHEET NO. 22



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200

SCALE: FEET

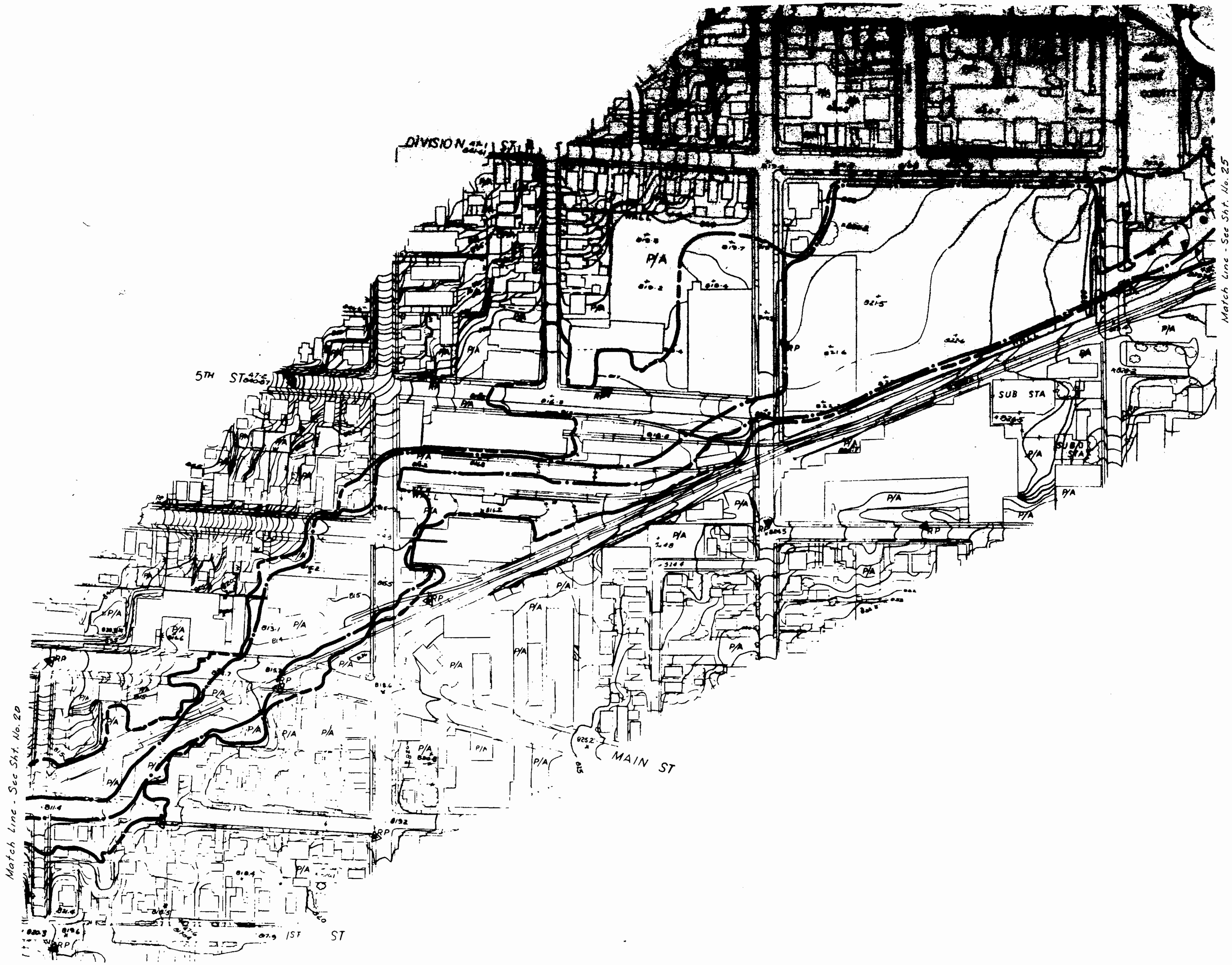
ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

McNamee Porter & Seeley

SHEET NO. 23



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

(2) Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200

SCALE: FEET

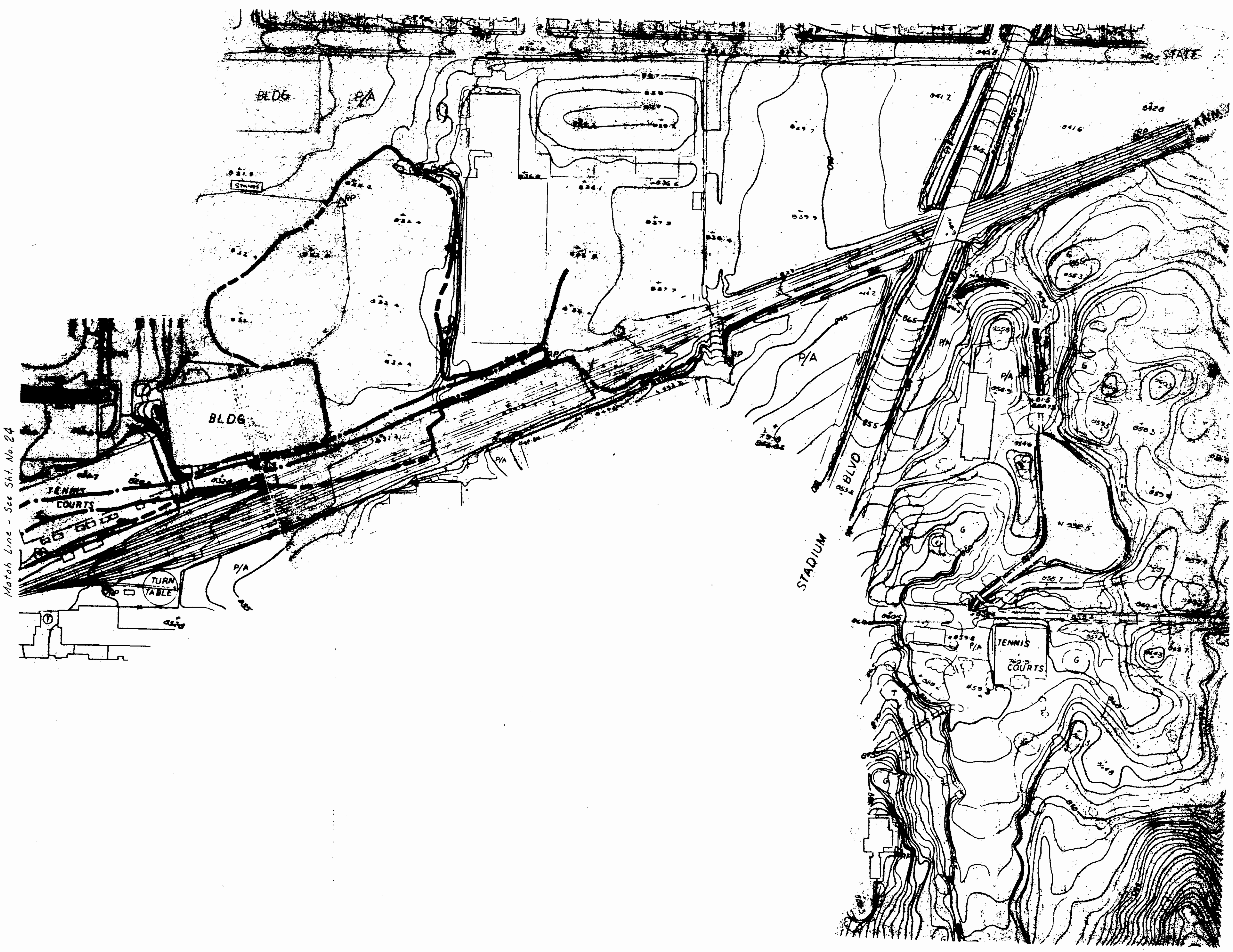
ANN ARBOR
MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

McNamee Porter & Seeley

SHEET NO. 24



SUPPLEMENT 2

FLOOD PLAIN DELINEATION
CHANNEL IMPROVEMENTS

LEGEND

② Cross Section

Channel Improvements

10 Year

100 Year

Culvert

0 100 200

SCALE: FEET

ANN ARBOR MICHIGAN

STORMWATER MANAGEMENT
PLAN FOR ALLEN'S CREEK
DRAINAGE SYSTEM

mps

McNamee Porter & Seeley

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SHEET NO. 25