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INNER LOOP STORM DRAIN SUMMARY REPORT

PHOENIX, ARIZONA

INTERSTATE
ARIZONA
10

INTERSTATE 10

PHOENIX-CASA GRANDE HIGHWAY

15TH AVENUE TO MARICOPA T.I.

HNTB

HOWARD NEEDLES TAMMEN & BERGENDOFF

I-10 MANAGEMENT CONSULTANT

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INNER LOOP STORM DRAIN
SUMMARY REPORT

Projects I-10-2(83)PE and I-10-3(140)PE
Contract 81-3

INTERSTATE 10
PHOENIX-CASA GRANDE HIGHWAY
15TH AVENUE TO MARICOPA T.I.

JULY, 1988

Prepared for
ARIZONA DEPARTMENT OF TRANSPORTATION

By
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REMARKS

HOWARD NEEDLES TAMMEN & BERGENDOFF

July 19, 1988

Mr. Thomas A. Parlante, P.E.
I-10 Papago Engineer
Urban Highway Section
Highways Division
Arizona Department of Transportation
205 South 17th Avenue, Room 216E
Phoenix, Arizona 85007

Re: I-10, Phoenix, Arizona
Inner Loop Storm Drain Summary Report

Dear Mr. Parlante:

HNTB is pleased to transmit 2 copies of the Inner Loop Storm Drain Summary Report. This report summarizes I-10 drainage design projects between 15th Avenue and the Maricopa Traffic Interchange. The report was prepared at the direction of the Federal Highway Administration.

We believe this report satisfies the intent and directions of the Federal Highway Administration. Should you have any questions or comments, please call at anytime.

Very truly yours,

HOWARD NEEDLES TAMMEN & BERGENDOFF

Richard M. Wells

Richard M. Wells, P.E.
Deputy Project Manager

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DOCUMENTS SUMMARIZED IN THIS REPORT

DOCUMENT	PREPARED BY
Final Report: Hydrology and Hydraulic Design Concepts for I-10 Inner Loop Storm Drain, October, 1982	Arthur Beard Engineers, Inc.
Evaluation of Value Engineering Alternatives (with appendices), December, 1983	HNTB
Mathematical Modeling of I-10 Inner Loop Drainage System, August, 1984 with subsequent additions	Dr. Charles C.S. Song
Air Release Requirement, I-10 Inner Loop Drain Tunnels, Phoenix Arizona, July, 1984	Dr. Charles C.S. Song
Geotechnical and Engineering Assessment of Tunnel Feasibility, Storm Water Disposal for the Inner Loop Freeway, I-10, Phoenix, Arizona, March, 1983	HNTB (In Value Engineering Appendices)
Final Drainage Concept Report, Phoenix I-10 Inner Loop, 15th Avenue to 21st Street, 21st Street to Maricopa Freeway, September, 1983	HNTB
21st Street Storm Drain, Phoenix I-10 Inner Loop: Alternative Cost Comparisons, December 1983	HNTB
Phoenix I-10 Inner Loop, North Storm Drain: Alternative Cost Comparisons, December, 1983	HNTB
Phoenix I-10 Inner Loop Freeway, Stormwater Disposal System April, 1985	Joseph Maestas, FHWA (Covers Tunnel Design)
I-10 Inner Loop, East Tunnel Outfall January, 1986	HNTB
I-10 Inner Loop, West Tunnel Outfall May, 1986	HNTB

DOCUMENT	PREPARED BY
Initial Design - Phase I Life Cycle Cost Analyses, Ehrenberg-Phoenix Highway, 51st - 27th Avenue Pump System, December, 1982	Harza Engineering Co.
Interstate 10 - West Papago/Inner Loop Pumping Stations, Final Design Report, Segment C 15th Avenue to 16th Street March, 1984	HNTB
Initial Design - Phase II, Phoenix-Casa Grande Highway, 15th Avenue to 16th Street Pumping Stations, August, 1986	Harza Engineering Co.
Final Drainage Design Report: Phoenix-Casa Grande Highway Inner Loop Storm Drain, December, 1986	PRC Engineering, Inc. (Covers SWI, drop structures, drop shafts and 2nd Street junction structure) (In January 1987 PRC Eng. became P & D Technologies)
The Helicoidal Ramp Dropshaft, May, 1987	Drs. John F. Kennedy & Subhash C. Jain Iowa Institute of Hydraulic Research
Model Tests of Phoenix, Arizona Central Avenue and I-10 Helicoidal Ramp Dropshaft, December, 1987	Drs. John F. Kennedy & Panayiotis Diplas Iowa Institute of Hydraulic Research
Preliminary Design Submittal, Central Avenue Drop Structure April, 1987	PRC Engineering, Inc.
Preliminary Drainage Design Report, I-10/Phoenix-Casa Grande Highway, 15th Avenue to Central Avenue, November, 1984	BRW, Inc.
Final Drainage Design Report: Phoenix-Casa Grande Highway, Central Avenue - 16th Street December, 1985	Greiner Engineering Sciences,Inc.
Final Drainage Design Report: The Deck at Central Avenue, 3rd Avenue - 3rd Street June, 1987	HNTB

DOCUMENT	PREPARED BY
Final Drainage Report: Phoenix-Casa Grande Highway Project, 16th Street - Roosevelt Street March, 1986	HNTB
Final Drainage Design Report: Phoenix-Casa Grande Highway, 16th Street - Roosevelt Street, Roadway August, 1983	Ruiz Engineering Corp.
Final Drainage Design Report: Phoenix-Casa Grande Highway, Roosevelt Street to Buchanan Street, April, 1986	Civil Engineering, P.C.
Final Drainage Design Report: Phoenix-Casa Grande Highway, 16th Street to Fillmore Street, November, 1987	HNTB
Hydrologic and Hydraulic Investigation for Phoenix-Casa Grande Highway, Harrison Street to Buckeye Road, April, 1986	Johnson-Brittain & Assoc., Inc.
Final Drainage Design Report: Phoenix-Casa Grande Highway, Buckeye Road to Maricopa Freeway, February, 1985	Stanley Consultants in association with Hess, Fogt, Rountree, Inc.
Feasibility Study: I-10 Inner Loop Storm Drain Access Control Equipment September, 1987	HNTB
Phoenix-Casa Grande Highway, Maricopa Interchange Pumping Station Initial Design: Phase I and Phase II July, 1984	John Carollo Engineers
Final Drainage Design Report: Inner Loop Drainage Tunnel Outfalls, June, 1987	Arthur Beard Engineers, Inc.
Pertinent Construction Documents	
Pertinent Correspondence	

DII162.13

CHAPTER 1 - INTRODUCTION

SYNOPSIS

This chapter provides a brief discussion of the steps involved in the design of the Interstate 10 - Inner Loop Drainage System. Included is a general description of the reports and studies leading up to the final design of the system and a summary of the system configuration.

I. PURPOSE

This report is intended as a reference manual covering the design, operation and maintenance of the I-10 Inner Loop drainage system in Phoenix, Arizona. The report summarizes the many documents prepared during the design of individual components of the total drainage system.

The Inner Loop drainage system intercepts storm runoff at the east-west section of Interstate 10 from 15th Avenue to 24th Street, and the north-south section from McDowell Road to the Maricopa Freeway. The project location is shown on Figure 1-1.

Although this report is primarily concerned with off-site drainage intercepted by the freeway project, on-site drainage systems and pump stations are also discussed. Drainage systems built in conjunction with the covered deck structure from 3rd Avenue to 3rd Street are included in appropriate sections. Topics range from preliminary concepts through design criteria and alternative analysis to hydraulic analysis and final component design.

II. REPORT ORGANIZATION

The discussion in this report is organized along subject lines, rather than merely presenting a project chronology. While the

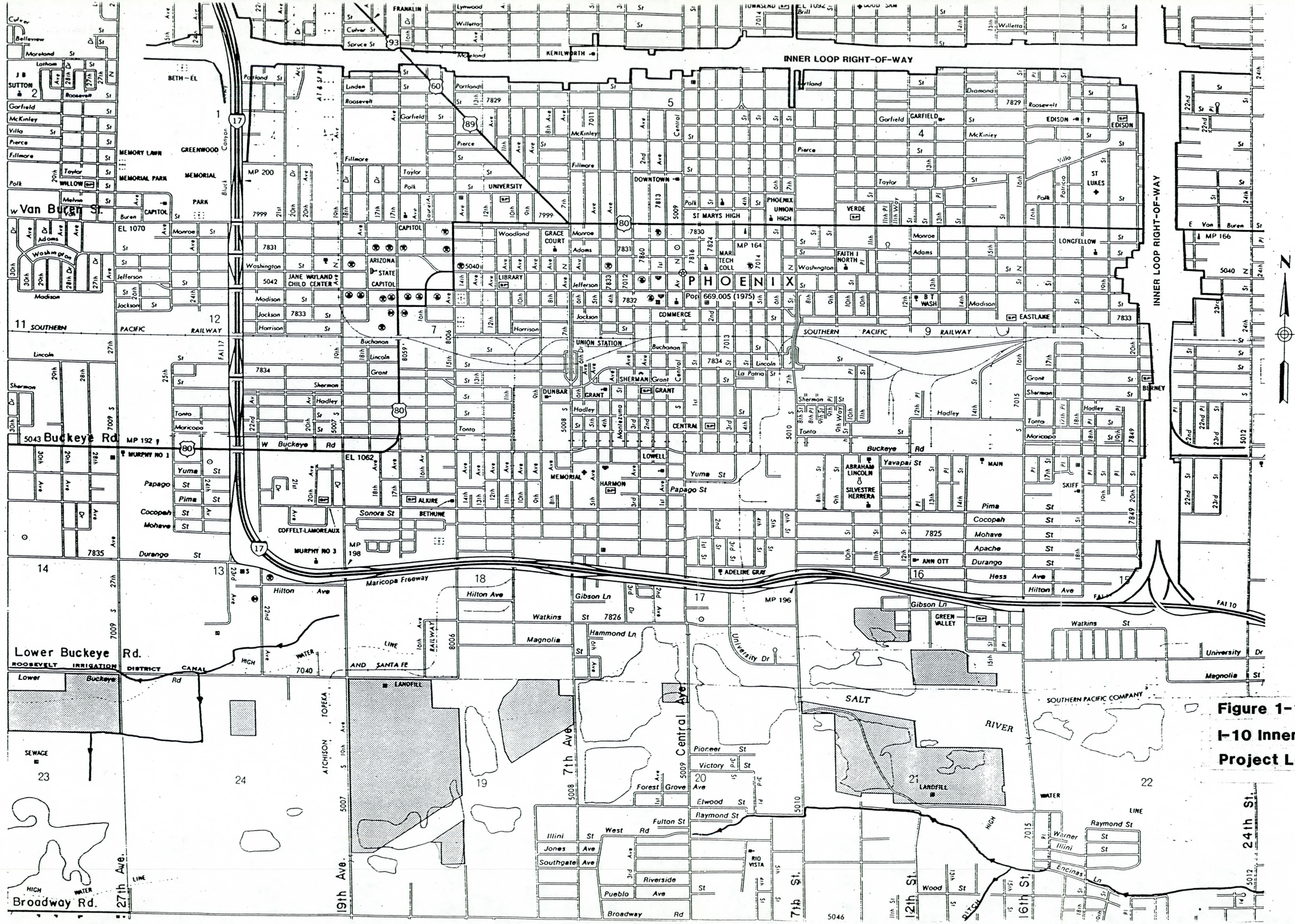


Figure 1-1
I-10 Inner Loop
Project Location Map

various subjects are by their nature closely related, they are treated more or less in order of increasing dependence.

Chapter 1 discusses the preliminary studies which established the basic system configurations and design criteria. Also included is a tabulation of the system components, the consultants responsible for their design, and the construction project numbers associated with each component.

Chapter 2 discusses the formulation of the off-site hydrology model and subsequent refinements, which provided the basis for the design flows used for all components of the drainage system.

The actual design flows applied to the various drainage system components are described in Chapter 3.

Chapter 4 discusses the design criteria, flows, and procedures used in the design of the surface flow collection systems.

Chapter 5 deals with the detailed design of the freeway pump stations.

Chapter 6 discusses the design of the Storm Water Interceptor and the main drainage tunnels. Since these structures are designed to accommodate the surge pressures anticipated during the design storm, this chapter also includes a discussion of the Transient Wave Analysis of the drainage system.

Chapter 7 describes the design of structures appurtenant to the drainage tunnels, including drop structures, drop shafts and the 2nd Street junction structure.

Chapter 8 deals with the design of the tunnel outfall structures at the Salt River and the tunnel dewatering pump stations.

III. BACKGROUND

A number of reports and studies were prepared before initiating the design of the Inner Loop Drainage System. The following sections of this chapter describe the three significant reports which established the criteria for the design of the major system components. This discussion merely provides an overview of the scope and results of these reports. Later chapters of this report treat several of the subjects in more detail.

The following three reports are discussed here:

- Hydrology and Hydraulic Design Concepts, prepared in October 1983, by Arthur Beard Engineers, Inc. (ABE), referred to in this report as the ABE Report.

- Evaluation of Value Engineering Alternatives, prepared in December 1983, by Howard Needles Tammen & Bergendoff (HNTB), also called the VE Report.

- Mathematical Modeling of I-10 Inner Loop Drainage System, prepared in August 1984, with later supplements, by Dr. Charles C.S. Song of the University of Minnesota-St. Anthony Falls Hydraulic Laboratory (SAFHL), known also as the Song Report.

A. Hydrology and Hydraulic Design Concepts

In October 1982, in accordance with its contract with the Arizona Department of Transportation, the firm Arthur Beard Engineers, Inc., prepared and submitted its Final Report on Hydrology and Hydraulic Design Concepts for the I-10 Inner Loop Storm Drain.

The purpose of this report was to present the results of studies relative to storm drainage along the I-10 Inner Loop Corridor in Phoenix. The study consisted of two major elements:

- Compute the hydrology of the drainage area contributing storm water flow along the Inner Loop's east-west corridor, located approximately 0.25-mile south of McDowell Road, from its intersection with I-17 to the end of the corridor at approximately 21st Street.

- Develop and evaluate alternative concept designs which safely conduct storm water away from the proposed freeway for storms equal to and less than the 50-year frequency storm event, without inundation of the freeway.

The hydrology portion of the study resulted in the establishment of a detailed mathematical model of the watershed using the Soil Conservation Service TR-20 Program. This model produced hydrographs of storm sewer and surface flow at each of the major 1/2-mile streets along the corridor for the 50-year design storm. Details of this model are discussed in the next chapter.

The scope of work for the storm sewer study envisioned the primary study route to be along the East (Northbound) Frontage Road of the Black Canyon Freeway. However, the consultant was required by the contract to evaluate other routes for the storm sewer in the initial phase of his study. In the course of this evaluation, the consultant investigated the feasibility of:

- Use of existing and future storm sewers.
- Use of bridges as hydraulic structures.
- Peak discharge reduction by storm water retention.
- Interception of flows above freeway alignment.
- Outfall tunnels to the river.
- Open channels.
- Joint-use facilities.

Most of these concepts were eliminated from detailed consideration, resulting in four alternative system designs:

- Alternative I utilized box culvert outfalls installed under I-17 Northbound Frontage Road, 1st Avenue, and 11th Street.

- Alternative II also utilized box culvert outfalls at I-17 Northbound Frontage Road, 1st Avenue, 11th Street and 20th Street (north-south leg of I-10).

- Alternative III utilized a 30-foot diameter tunnel under 15th Avenue to conduct all flows to the Salt River from the I-10 alignment.

- Alternative IV utilized two 20-foot diameter tunnels, one under 15th Avenue and the other under Central Avenue, for discharges to the river.

The report recommended Alternative IV, with an estimated total construction cost of about $85.8 million.

B. Value Engineering Study

During the review of the ABE Report, it became apparent that the calculated storm drainage flows combined with the depressed freeway alignment created a very unusual and difficult design and construction problem. The magnitude of the problem was demonstrated in the ABE report by high estimated construction costs, large-scale drainage structures and major impacts on traffic and existing utilities. Accordingly, ADOT, with the concurrence of the Federal Highway Administration (FHWA), elected to have a Value Engineering (VE) study conducted to identify, develop and evaluate additional conventional and non-conventional conceptual approaches toward solution of the off-site drainage problem.

The first phase of the VE study consisted of an eight-hour "brainstorming" session to identify potential additional conceptual approaches, including some of those previously ruled out by the ABE Report, which were then to be further developed and evaluated in the second phase of the study. Based on the recommendations from the brainstorming sessions, the evaluation phase of the VE study included the following elements:

- Analysis and refinement of the hydrologic and hydraulic studies to identify means of reducing the peak flows. In particular, the review of the hydrology included a more detailed analysis of the spatial distribution of flows, a sensitivity analysis of the TR-20 hydrology model parameters, and an investigation of alternative hydrologic modeling methods (such as SWMM). The hydraulic studies included a risk analysis of the incremental costs associated with full freeway protection versus real and potential costs associated with freeway flooding for a range of storm frequencies up to and including 100 years.

- Development of systems which would convey storm sewer and surface drainage flows across the freeway by inverted siphons or surface hydraulic structures.

- Consideration of temporary storage of storm water as a means of reducing peak flows at the freeway.

- Further consideration and refinement of the ABE Report alternatives after re-evaluation of the hydrologic and hydraulic studies. This included a tunnel feasibility study and a preliminary soils exploration program.

- Evaluation of alternatives considering combinations of possible solutions, including groundwater recharge, storage, tunnel and open-cut outfalls, open channels, and conveyance of flows across the freeway right-of-way.

A report discussing the procedures and results of the evaluation phase of the Value Engineering Study was prepared by HNTB in December of 1983. The conclusions of the various portions of the study are summarized as follows:

- The refinements to the TR-20 hydrology model used by Arthur Beard Engineers resulted in significant reductions in the predicted peak off-site flows reaching the Inner Loop, and also resulted in a redistribution of the flows along the length of the corridor. These changes were caused primarily by the effects of storage and diversion of surface flows north of the Grand Canal.

- Investigation of the STORM and SWMM3 computer models confirmed that TR-20 was the most appropriate to the situation.

- The freeway flooding risk analysis was inconclusive in determining the most economical level of flood protection, although the 50-year design could not be supported. However, further analysis of flooding damage indicated that the 50-year level of protection would provide significant benefits to residents and businesses located within the I-10/I-17 Inner Loop. Such a system would also provide direct benefits to the City of Phoenix in reduced capital costs for proposed storm sewers in the Central Corridor area south of I-10.

- The final plan was redesigned for the runoff from the 25-year storm, resulting in a potential cost savings of only seven percent. Therefore, the 50-year design criterion was retained for the depressed portion of the freeway alignment.

- The analysis of alternative drainage system concepts concluded that, while feasible, conveyance of storm drainage across the freeway or temporary storage of storm water could not be justified from an economic or environmental standpoint.

- The Geotechnical and Tunnel Feasibility Studies concluded that construction of bored tunnel outfalls is feasible, provided the alignment remains within the sand-gravel-cobble (S-G-C) soil layer which underlies the surface soils throughout the project area.

- The evaluation of alternatives concluded that the most economical and practical drainage system consists of a network of pipe, box culverts, and bored tunnels, intercepting all storm sewer and surface drainage flow at the Inner Loop corridor and conveying it to the Salt River for discharge.

- Mathematical modeling of the drainage system indicated the need for special structures to accommodate surges generated within the system as the tunnels pressurize, with further modeling necessary to finalize the locations and sizes of these structures.

The recommendations in the VE Report describe an off-site drainage system designed to intercept all runoff from the 50-year storm which reaches the I-10 Inner Loop between 13th Avenue and the Maricopa Freeway interchange. All existing and proposed City of Phoenix storm sewers north of the east-west leg of the Inner Loop will be connected to the system, as will the proposed Papago East Freeway off-site drainage system and proposed City storm sewers within the Central Corridor area south of I-10.

The report recommended two separate conveyance systems, as follows:

- A system of concrete pipe and box culvert along the north side of the freeway from 19th Street to 2nd Street and from 13th Avenue to 7th Avenue; a 14-foot diameter tunnel from 7th Avenue to 2nd Street; and a 21-foot diameter tunnel under 2nd Street and Central Avenue, from Culver Street south to the Salt River.

- Concrete pipe and box culvert from 20th and Belleview Streets east to 21st Street, then south along 21st Street to Van Buren Street; and a 21-foot diameter tunnel along the east side of the freeway, from Van Buren Street to the Salt River.

C. Transient Wave Analysis

As part of the VE Study, the drainage system was modeled using the Extended Transport (EXTRAN) block of the U.S. Environmental Protection Agency's Storm Water Management Model (SWMM). The results of this computer modeling indicated that the configuration of the tunnels as inverted siphons created severe hydraulic instabilities during the tunnel filling process, resulting in extensive street flooding and backflow in the system. Further investigation revealed that the EXTRAN program could indicate the existence of this phenomenon, but it is not capable of accurately modeling it.

In the search for alternatives to the EXTRAN model, it was discovered that similar problems had been encountered on tunnel projects in Rochester, New York and Minneapolis, Minnesota. A recognized expert in the field of transient pressure wave analysis, Dr. Charles C.S. Song of the University of Minnesota-St. Anthony Falls Hydraulic Laboratory, had developed a computer model to analyze the tunnel pressurization process and the resulting transient pressure surges common to such systems. Dr. Song was retained to conduct detailed modeling of the I-10 drainage tunnels to verify the design of the system.

Dr. Song's modeling work encompassed a number of drainage system configurations and hydraulic conditions. In all cases, however, the objective was the same: to determine the most efficient combination of facilities to contain the volume of water associated with the surge front and to provide sufficient transient pressure relief to prevent destructive internal pressures from developing within the system. As a result of this work, many refinements have been made in the drainage system since the completion of the VE Report. The details of these changes are discussed later in this report.

IV. DRAINAGE SYSTEM DESIGN AND CONSTRUCTION

Various consultants have been responsible for the detailed design of I-10 drainage facilities.

The drainage system components, their design consultants, and the construction projects they were assigned to are listed on Table 1-1. A map of the major structures in the Inner Loop storm drain system is shown in Figure 1-2.

TABLE 1-1

SUMMARY OF DRAINAGE CONSTRUCTION PROJECTS

COMPONENT	DESIGNER	CONSTRUCTION PROJECT
Off-Site Collection Systems:		
- 15th Ave. to 9th Ave.	BRW, Inc.	I-10-3(120)
- 7th Ave.	BRW, Inc.	I-10-3(82)
- 5th Ave. to 3rd Ave.	BRW, Inc.	I-10-3(240)
- 3rd Ave. to Central Ave.	BRW, Inc.	I-10-3(223)
- Central Ave.	BRW, Inc.	I-10-3(307)
- Central Ave. to 3rd St.	Greiner Engineering	I-10-3(307)
- 3rd St. to 5th St.	Greiner Engineering	I-10-3(213)
- 5th St. to 7th St.	Greiner Engineering	I-10-3(208)
- 7th St. to 12th St.	Greiner Engineering	I-10-3(239)
- 11th, 12th and 13th St.	Greiner Engineering	I-10-3(214)
- 13th St. to 20th St.	Greiner Engineering	I-10-3(215)
- 20th St. to Fillmore St.	*PRC Engineering	I-10-3(224)
- Fillmore St. to Jackson St.	Centennial Engineering	I-10-3(239)
- Harrison St. to Buckeye Rd.	**Johnson-Collins & Assoc.	I-10-3(239)
- Buckeye Rd. to Maricopa Int.	Stanley Consultants	I-10-3(239)
Storm Water Interceptor:		
- 13th Ave. to 9th Ave.	PRC Engineering	I-10-3(120)
- 9th Ave. to 7th Ave.	PRC Engineering	I-10-3(82)
- 10th St. to 12th St.	PRC Engineering	I-10-3(239)
- 12th St. to 13th St.	PRC Engineering	I-10-3(214)
- 13th St. to 20th St.	PRC Engineering	I-10-3(215)
- 20th St. to Moreland St.		I-10-3(224)
Drop Structure	PRC Engineering	

* Currently P & D Technologies

** Currently Johnson-Brittain & Associates

TABLE 1-1 (CONTINUED)

COMPONENT	DESIGNER	CONSTRUCTION PROJECT
Drop Shafts:		
- West Tunnel (4)	PRC Engineering	I-10-3(223)
- 3rd Ave.	PRC Engineering	I-10-3(223)
- 3rd St. and 7th St.	PRC Engineering	I-10-3(239)
- East Tunnel (6)	PRC Engineering	I-10-3(224)
- Central Ave.	PRC Engineering	I-10-3(307)
Drop Structures:		
- 7th Ave.	PRC Engineering	I-10-3(82)
- 10th St.	PRC Engineering	I-10-3(239)
- Moreland St.	PRC Engineering	I-10-3(224)
Storm Drain Tunnels:		
- West Tunnel	HNTB	I-10-3(189)
- East Tunnel	HNTB	I-10-3(188)
- North Tunnel	HNTB	I-10-3(187)
West Tunnel Outfall:	Arthur Beard Engineers	I-10-3(223)
East Tunnel Outfall:	Arthur Beard Engineers	I-10-3(225)
2nd St. Junction Structure:	PRC Engineering	I-10-3(247)
Pump Stations:		
- 3rd Ave.	Harza Engineering	I-10-3(223)
- 3rd St. and 16th St.	Harza Engineering	I-10-3(239)
- Maricopa Freeway	John Carollo Engineers	I-10-3(204)
13th Ave. Surge Pond:	BRW, Inc.	I-10-3(120)

TABLE 1-1 (CONTINUED)

COMPONENT	DESIGNER	CONSTRUCTION PROJECT
On-Site Drainage:		
- 15th Ave. to 9th Ave.	BRW, Inc.	I-10-3(120)
- 9th Ave. to 3rd Ave.	BRW, Inc.	I-10-3(246)
- 3rd Ave. to Central Ave.	BRW, Inc.	I-10-3(245)
- Central Ave. to 2nd St.	BRW, Inc.	I-10-3(307)
- 2nd St. to 3rd St.	Greiner Engineering	I-10-3(307)
- 3rd St. to 16th St.	Greiner Engineering	I-10-3(239)
- 16th St. to Fillmore St.	Ruiz Engineering/HNTB	I-10-3(239)
- McDowell Ramps	HNTB	I-10-3(239)
- McDowell Ramps	HNTB	I-10-3(246)
- Fillmore St. to Harrison St.	Centennial Engineering	I-10-3(239)
- Harrison St. to Buckeye Rd.	Johnson-Collins & Assoc.	I-10-3(239)
- Buckeye Rd. to Maricopa Int.	Stanley Consultants	I-10-3(239)
Covered Deck Drainage:		
- 3rd Ave. to Central Ave.	BRW, Inc./HNTB	I-10-3(245)
- Central Ave. to 3rd St.	BRW, Inc./HNTB	I-10-3(307)

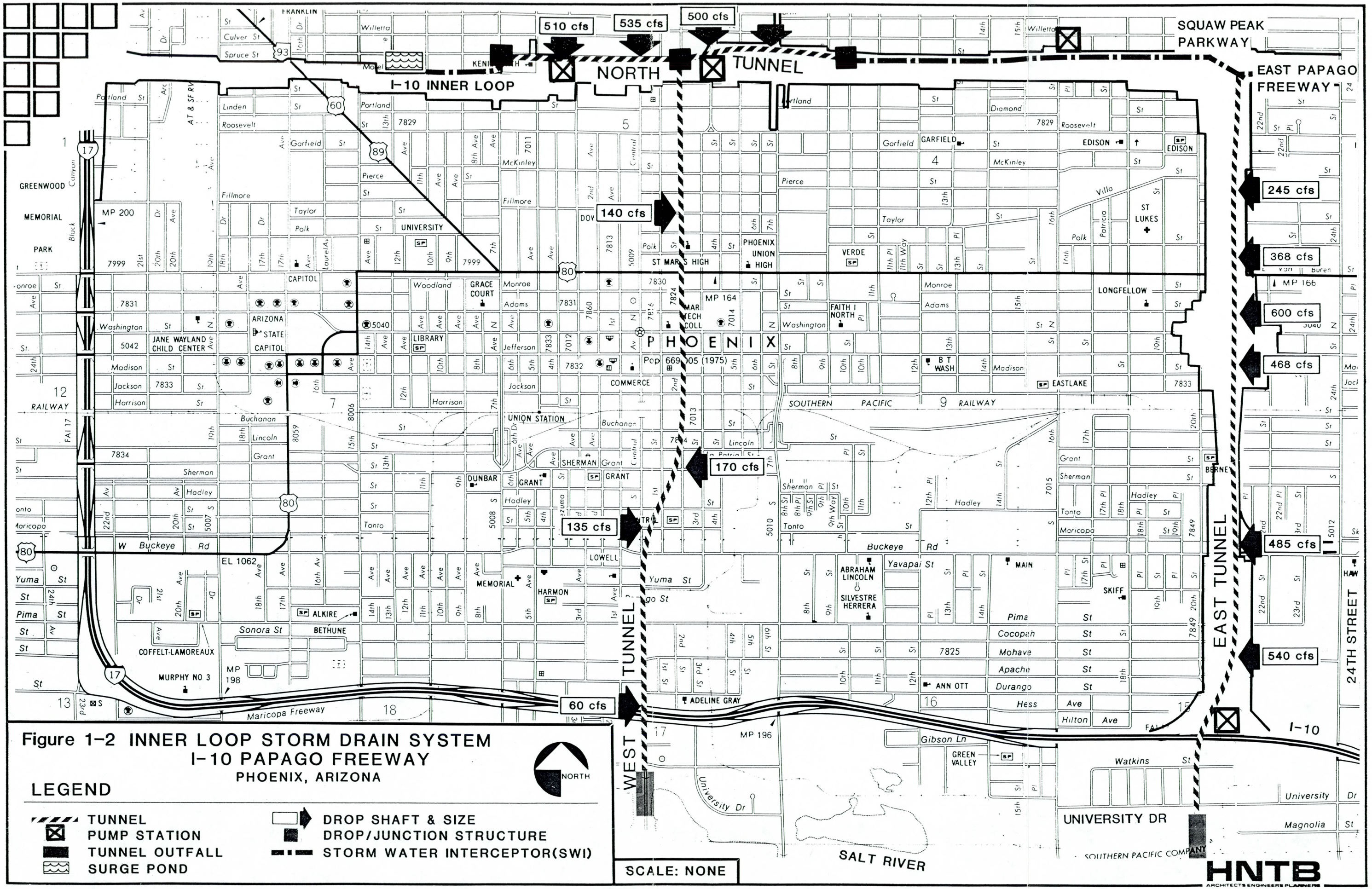


Figure 1-2 INNER LOOP STORM DRAIN SYSTEM
I-10 PAPAGO FREEWAY
PHOENIX, ARIZONA

LEGEND

- TUNNEL
- PUMP STATION
- TUNNEL OUTFALL
- SURGE POND
- DROP SHAFT & SIZE
- DROP/JUNCTION STRUCTURE
- STORM WATER INTERCEPTOR(SWI)

CHAPTER 2 - OFF-SITE HYDROLOGY

SYNOPSIS

This chapter describes the development of the original TR-20 off-site hydrology model and subsequent refinements and expansion of the model. The discussion is in two major parts. The first part deals with methodology behind the development of the original model by Arthur Beard Engineers, and the second part discusses each of the later refinements by HNTB.

I. ORIGINAL TR-20 MODEL

As stated in the previous chapter, part of the scope of work for the original study by Arthur Beard Engineers (ABE) included the computation of the hydrology affecting the east-west leg of the Inner Loop, from approximately 15th Avenue to 21st Street. ABE elected to employ the SCS program TR-20 to accomplish this purpose. The following is a brief description of the procedure followed in establishing the model.

A. Description of the Drainage Area

The drainage area affecting the east-west leg of the Inner Loop extends from the Arizona Canal on the north to the I-10 corridor on the south. The western boundary is defined by the Black Canyon Freeway (I-17), with the eastern boundary extending northeast from 21st Street and Moreland to the Arizona Canal at 44th Street. Figure 2-1 shows a map of the drainage area. The Arizona Canal will form a barrier to surface flow into the drainage area from the north and east. At 15th Avenue, the I-10 freeway changes from a depressed to an elevated highway. Cave Creek Wash surface flows reaching that portion of the

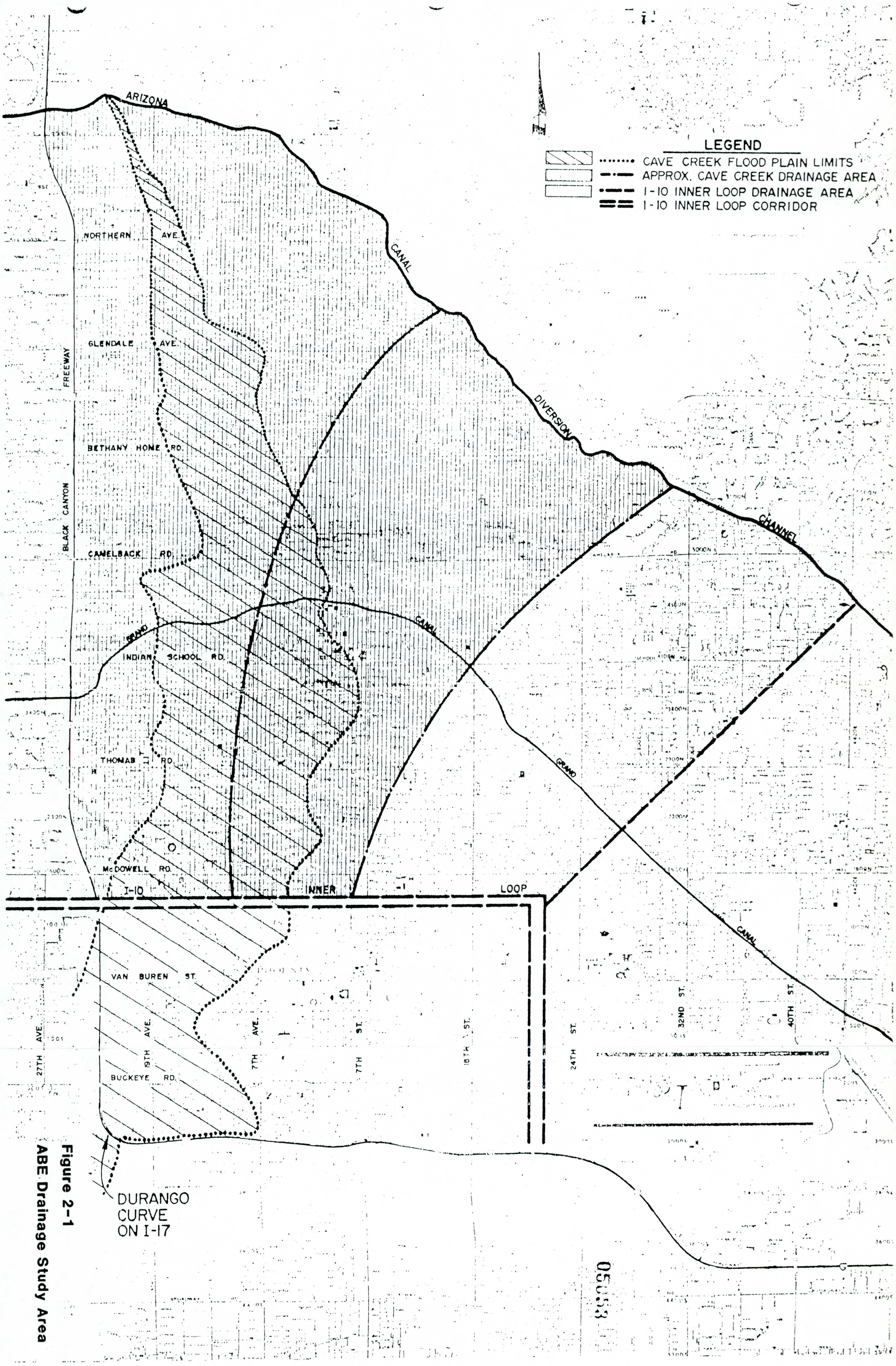


Figure 2-1
ABE Drainage Study Area

I-10 right-of-way from 15th Avenue to the Black Canyon Freeway will continue to flow overland, under the elevated freeway, and will not be intercepted by the Inner Loop drainage system. 15th Avenue, although not a drainage area boundary, represents an effective study area boundary on the west. The eastern boundary represents the approximate limits of westbound surface flow reaching the east-west leg of the I-10 Inner Loop. The gross drainage area amounts to approximately 44 square miles.

The general topography of the study area is nearly flat, with slopes generally less than one percent, trending toward the southwest. Street widths range from 20 to 77 feet, and all major streets in the study area run in a north-south, east-west grid pattern. No major diagonal streets cross the freeway corridor within the study area.

Development in the study area ranges from urban single family residential to high rise commercial. Many of the property owners in the areas zoned for single-family residential development irrigate their yards by the flooding method, in which berms around each lot confine irrigation water and rainfall, allowing the water to percolate into the soil. In addition, certain areas zoned for multi-family residential, commercial and office development are required by zoning regulation to retain their storm water on-site for the 10-year frequency storm.

Existing storm water drainage within the study area is provided by City of Phoenix storm drains and by surface runoff along existing city streets. The major existing City of Phoenix storm drains are located in 15th Avenue, 9th Avenue, 7th Avenue, 7th Street, 16th Street, 24th Street and 32nd Street. All of these existing systems conduct storm water from the study area south to the Salt River, and are generally sized for the 2-year frequency storm or less.

Surface runoff (overland flow) occurs in the areas not served by storm sewers for all storm frequencies, and in all areas when a storm of greater intensity than a 2-year storm occurs. The surface runoff generally follows the major streets in the area, since these routes offer the least resistance to flow. The grid pattern followed by the major streets causes the surface runoff to split at each major intersection, with the amounts of flows to the south and west from each intersection related to the street configurations and the downstream slopes of the streets at that intersection.

B. Development of the Hydrology Model

The TR-20 program can compute surface runoff for any synthetic or natural rainfall pattern, and takes into account conditions affecting runoff in the drainage area through the use of input data furnished by the engineer. From this data, hydrographs are developed for all drainage areas at collection points, or points of concentration. These hydrographs are then routed through the drainage system and combined with other hydrographs to develop composite hydrographs. The TR-20 program is capable of diverting part or all of a hydrograph through a storm drain, and can account for on-site retention within the drainage area. At street intersections where runoff flows away in two or more directions, the program can split the hydrograph and route flows downstream in several directions.

To estimate the hydrology of a watershed, the following input data is required:

- Rainfall intensity and duration.
- Drainage area size and time of concentration.
- Drainage area runoff characteristics, expressed as a composite curve number.
- Distance and velocity of routing between drainage areas.

1. Rainfall Data

The TR-20 program calculates runoff hydrographs based on a 24-hour rainfall distribution which contains within it peak 1-hour, 2-hour, 3-hour, 6-hour, and 12-hour storms. This is accomplished by expressing rainfall amounts for each half-hour as a percentage of the total 24-hour rainfall. To do this, the rainfall amounts for the shorter storms are determined as a percentage of the 24-hour rainfall for return periods from 2 years up to 100 years. The half-hourly rainfall percentages are then determined so that the shorter storms are all centered around hour 12.

Using rainfall frequency data for the Phoenix area, the relationships between storm durations are as follows, averaged among six return periods:

1-hour rainfall = 66% of 24-hour rainfall
2-hour rainfall = 72% of 24-hour rainfall
3-hour rainfall = 76% of 24-hour rainfall
6-hour rainfall = 84% of 24-hour rainfall

The 24-hour rainfall distribution was developed so that 66% of the total rainfall occurs between hours 11.5 and 12.5, 72% falls between hours 11 and 13, 76% between hours 10.5 and 13.5, and 84% between hours 9 and 15 (See Table 2-1).

The resulting distribution was found to show higher rainfall intensities for the short duration storms than those obtained from standard SCS distributions. However, it was determined that this was appropriate for rainfall patterns in the area. Furthermore, this distribution was found to be very close to that already being used by the City of Phoenix for local drainage projects.

TABLE 2-1

24-HOUR RAINFALL DISTRIBUTION

Hour	Percent Total Rainfall	Hour	Percent Total Rainfall
0.0	.000	12.5	.83
0.5	.004	13.0	.86
1.0	.008	13.5	.88
1.5	.013	14.0	.893
2.0	.018	14.5	.907
2.5	.022	15.0	.92
3.0	.026	15.5	.924
3.5	.031	16.0	.928
4.0	.035	16.5	.933
4.5	.040	17.0	.937
5.0	.044	17.5	.942
5.5	.048	18.0	.947
6.0	.053	18.5	.951
6.5	.057	19.0	.956
7.0	.062	19.5	.96
7.5	.066	20.0	.964
8.0	.071	20.5	.969
8.5	.075	21.0	.973
9.0	.08	21.5	.978
9.5	.093	22.0	.982
10.0	.107	22.5	.987
10.5	.12	23.0	.991
11.0	.14	23.5	.995
11.5	.17	24.0	1.00
12.0	.50		

SOURCE: "Hydrology and Hydraulic Design Concepts for I-10 Inner Loop Storm Drain," Arthur Beard Engineers, Inc., October 1982.

Total 24-hour rainfall amounts were established as follows, using rainfall frequency atlases and standard procedures developed by the National Weather Service:

Return Period, years	Rainfall, inches
2	1.4
5	2.0
10	2.4
25	2.95
50	3.4
100	3.85

Areal reduction of these rainfall amounts over the 44-square mile area was considered, but not recommended, based on discussions with local climatologists and FHWA.

2. **Drainage Sub-Area Breakdowns**

For input to the TR-20 hydrology procedure, the Inner Loop drainage area was divided into 45 sub-areas. The following criteria were considered in the breakdown:

- Minimum time of concentration within each sub-area of ten minutes.
- Locations of existing and proposed major storm sewers by the City of Phoenix.
- Locations of major streets.
- Location of the Grand Canal.

The sub-area boundaries in the original ABE model generally follow the north-south streets at 1/2 mile intervals, and the

east-west streets at 1-mile intervals, with additional diversions along the Grand Canal. Larger sub-areas were used east of 24th Street.

3. Sub-Area Characteristics

For each drainage sub-area, the following input parameters were determined:

- Net contributing area (Acres)
- Composite curve number
- Retention volume (Acre-feet)
- Time of concentration (Hours)
- Modified curve number

The following sections explain the procedures followed in the determinations.

a. Net Contributing Area

The net contributing area for each drainage sub-area was calculated from the overall dimensions of each sub-area, subtracting the total quantity of residential area which is irrigated by flooding and does not contribute to runoff. It was assumed that the berm heights in these areas are sufficient to retain all precipitation up to and including the 100-year storm.

b. Composite Curve Number

SCS hydrologic soil cover complexes (runoff curve numbers, CN's) have been determined by the City of Phoenix for fully developed conditions, covering all city zoning designations and soil classifications. Each sub-area in the hydrology model was further divided by zoning type and soil classification. The runoff curve number for each division

was then weighted by the percentage of the sub-area the division covers and the results averaged to obtain the sub-area composite curve number. Undeveloped parcels were treated as fully developed in accordance with their particular zoning designation, except in the Central Avenue Corridor, where land use changes projected to occur through re-development were considered.

Flood irrigated areas were excluded from the curve-number weighting calculations. Also, zoning classifications having a total area of less than 0.002 square miles were ignored, and the area added to the largest zoning category in that sub-area.

c. **Retention Volume**

New commercial and industrial developments are required by ordinance to provide retention volume on-site for runoff from a 10-year storm. Retention basin design must be approved by the City of Phoenix prior to the beginning of construction. Developments on less than one acre and areas currently without retention undergoing redevelopment generally do not have to meet the retention requirement.

Existing commercial and industrial developments with retention were identified and recorded during a lot by lot drainage area examination. It was assumed for each commercial and industrial zoning classification that the required retention would affect a specific percentage of the total lot area, with street right-of-ways and lot fringe areas draining away from the retained areas. These percentages vary by zoning classification. Where the presence of 10-year retention was verified, the gross lot area was multiplied by the percentage affected by retention,

then multiplied by the 10-year runoff depth, producing a volume of retained runoff that could be subtracted from sub-area runoff volume totals for storms of greater than 10-year return periods.

d. **Time of Concentration**

The time of concentration for a sub-area is the time required for runoff from the most remote part of the sub-area to reach the concentration point. This was generally determined for the model by finding the maximum distance of flow along city streets within the sub-area, and dividing that distance by the average full gutter flow velocity. The average full gutter flow velocity was determined by using the standard City of Phoenix Gutter Flow Chart in conjunction with the average slope along the maximum distance of flow. Average slopes were determined by using manhole rim elevation data from the City of Phoenix Quarter-Section Sanitary Sewer Maps.

e. **Modified Curve Number**

For sub-areas with multi-family, commercial, or industrial 10-year runoff retention, the total volume of runoff from the 50-year storm was calculated using the sub-area composite curve number, the 50-year rainfall depth, and the net contributing area within the sub-area. The retention volume (described above) was then subtracted from the 50-year runoff volume, to produce the total net volume of 50-year runoff actually contributed by the sub-area. This net volume of 50-year runoff was applied over the sub-area net contributing area to compute a Modified Curve Number (MCN_{50}) for the 50-year storm to be used in the TR-20 hydrology model.

Peak flows generated through use of the MCN_{50} are equal to or lower than peak flows generated by the original sub-area composite curve number, with any reduction of peak reflecting the effects of the 10-year runoff retention. This process was repeated for the 100-year storm, producing a different set of modified curve numbers for the affected sub-areas.

C. Flow Routing Procedures and Results

The ABE hydrology model made use of three of the subroutines available within the TR-20 program:

- The REACH subroutine uses the convex routing method to lag and attenuate a hydrograph, based upon a routing reach length and a coefficient (C) representing average velocity of flow. The C value is either determined by the computer or directly input by the programmer. The computer calculates C from an equation, using the average velocity of all hydrograph ordinates greater than half the peak ordinate value. The velocities of the ordinates are determined through use of the input cross section rating table for the routing reach. The REACH subroutine was used for routing both surface and pipe flows. In the latter case, a constant routing coefficient was input for each pipe segment.

- The ADDHYD subroutine creates a composite hydrograph of total flow at a node by adding the local runoff hydrograph for the point in question and any pipe and surface flow hydrographs routed from upstream points.

- The DIVERT subroutine facilitates modeling the network type runoff pattern which results when major city streets carry most of the runoff, and is used to split hydrographs in two ways. First, at points where part of the runoff is carried in storm drains, the DIVERT subroutine is used to put all

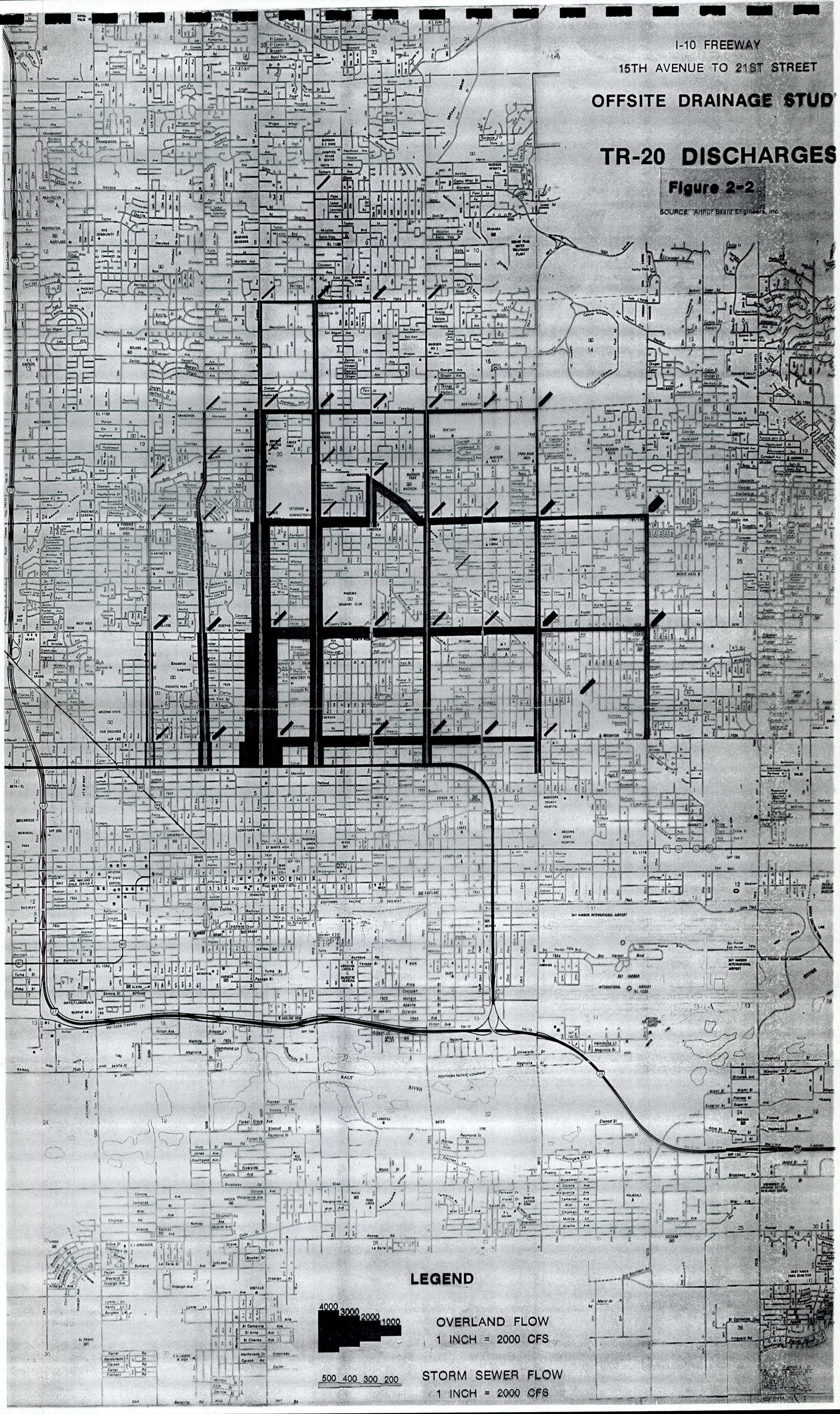
flow less than an input value (the storm drain capacity) into one hydrograph and the excess flow into another hydrograph representing surface flow. The second application of the DIVERT subroutine is to split surface flows by using two cross-section rating tables to balance the split flows, based on the relative conveyance capacities of the downstream street sections for the same depth of flow at the node point.

1. Routing Between Sub-Areas

a. Surface Flows

For modeling purposes, surface flows were schematically routed down the half-mile grid streets. The half-mile (generally arterial) streets tend to be the wider streets, with relatively constant gutter grades. Side street gutter grades are rarely carried through intersections with the half-mile streets, and frequently side street grades are slightly "humped" on the downstream side of the half-mile streets. Because of these conditions, it can be expected that the first flows exceeding the capacities of the city storm sewers will be conveyed primarily by the half-mile grid streets. As the flows in excess of the storm sewer capacities increase in depth and magnitude, the streets begin to convey more significant quantities of flow. Figure 2-2 shows the routed surface and storm sewer flows.

Because it was well beyond the scope of the computer model to precisely analyze the effect of these additional types of surface flow, it was assumed that all flows follow half-mile grid streets for routing purposes. Cross-section rating curves were developed for each reach of street between concentration points based upon Manning's equation, using a Manning's "n" of 0.015 and the average slope of the street.


I-10 FREEWAY
15TH AVENUE TO 21ST STREET
OFFSITE DRAINAGE STUD
TR-20 DISCHARGES
Figure 2-2
SOURCE: Arthur Beard Engineers, Inc.
LEGEND
4000 3000 2000 1000
OVERLAND FLOW
1 INCH = 2000 CFS
500 400 300 200
STORM SEWER FLOW
1 INCH = 2000 CFS

A six-inch vertical curb and a crown elevation equal to the top of curb elevation were assumed. The remainder of the right-of-way was assumed to be paved at the same elevation as the top of curb, and able to convey flows.

To allow for the spread of higher flows to side streets, alleys, and other interior areas capable of flow conveyance, the maximum velocity for any street cross section rating curve was limited to the velocity for a one-foot depth of flow in the street gutter. This was done to represent a reasonable average velocity for a flow situation including flows greater than one foot deep in the major streets and shallow flows in side streets and alleys.

b. Existing Storm Drains

City of Phoenix practice is to design storm drains for the runoff from a two-year frequency storm. Therefore it was assumed that the sewers would be surcharged during the major storms being analyzed. Storm drain capacities north of McDowell Road used in the TR-20 model were based on the installed pipe diameter and a hydraulic grade line slope equal to the average ground slope over the reach. South of McDowell Road, the ABE model used hydraulic slopes based on lowering the downstream end of the hydraulic grade line to a point three feet below existing ground.

Existing storm drains within the drainage area studied by ABE are located along 15th Avenue, 7th Avenue, 9th Avenue, 7th Street, 16th Street, 24th Street and 32nd Street.

c. Proposed Storm Drains

In keeping with their practice of providing major storm drains at 1/2-mile intervals, the City of Phoenix is planning

new storm drains along Central Avenue, 12th Street and 20th Street. At the time the ABE hydrology report was prepared, the City had not yet established design flows for these sewers. Capacities for use in the model were based on preliminary studies of the 2-year hydrology for the drainage area. The capacities used in the model for the storm drain segments between McDowell Road and the I-10 Corridor were 532 cfs at Central Avenue and 420 cfs at 12th Street. What was referred to as the 20th Street storm drain became the Squaw Peak Storm Drain. Its capacity was modeled at approximately 500 cfs for the segment between McDowell Road and the I-10 corridor.

2. Effects of Existing Canals

a. Grand Canal

In formulating the ABE model, the Grand Canal was assumed to be flowing full during the 50-year storm with runoff from outside the project study area. Therefore, the model showed no influence by the canal on the conveyance of surface flows. A more detailed analysis of storage and conveyance along the canal's banks was performed in the VE Report combined model, which is described later in this chapter.

b. Arizona Canal

The Arizona Canal was defined as the northern boundary of the drainage study area for the ABE model. It was assumed no drainage from areas north of the canal entered the Inner Loop drainage area.

3. Modeling Results

Table 2-2 summarizes various modeling results that are discussed throughout the remainder of this chapter.

TABLE 2-2

OFF-SITE HYDROLOGY

TR-20 DISCHARGES

	A		B		C		D		E		F		G		H		I	
	Peak 50-Year Discharge (cfs)																	
	ABE Report		HNTB Refined Model		Combined Model With Pap. E.		Combined Model Without Pap. E.		Proposed Storm Sewers Revised		Squaw Peak Hydrology		20th St. Sewer Removed		Corrected Model		Interim Conditons	
Point of Concentration	Sewer	Surface	Sewer	Surface	Sewer	Surface	Sewer	Surface	Sewer	Surface	Sewer	Surface	Sewer	Surface	Sewer	Surface	Sewer	Surface
East-West Corridor																		
15th Avenue	600	389	370	599	370	597	370	597	370	564	370	563	370	575	370	609	370	686
7th Avenue	300	1,021	155	978	155	954	155	954	155	1,018	155	1,017	155	1,061	155	1,095	155	1,212
Central Avenue	320	3,878	320	560	320	549	320	549	383	595	383	595	383	648	383	597	0	662
7th Avenue	310	1,270	195	601	195	637	195	637	195	788	195	786	195	947	195	793	195	826
12th Street	420	0	420	287	420	277	420	277	296	342	296	339	296	420	296	340	0	338
16th Street	185	753	130	258	130	257	130	257	130	251	130	243	130	339	130	244	130	244
20th Street	265	0	265	231	265	232	265	232	532	184	532	205	0	293	532	206	532	206
Subtotals	2,400	7,311	1,855	3,514	1,855	3,503	1,855	3,503	2,061	3,742	2,061	3,748	1,529	4,283	2,061	3,884	1,382	4,174
Totals		9,711		5,369		5,358		5,358		5,803		5,809		5,812		5,945		5,556
North-South Corridor																		
Belleview					1,957			84										
Roosevelt						119		269										
Van Buren						715		1,051										
Washington						800		935										
Buckeye						314		314										
Mohave						588		588										
Subtotals					1,957	2,536	0	3,241										
Totals						4,493		3,241										

NOTE: Revisions in Columns E through I did not affect flows along the north-south corridor.

Column A of Table 2-2 lists the peak flows at the Inner Loop, as predicted by the original ABE hydrology computer model.

II. REFINEMENTS TO THE MODEL

Part of the scope of the Value Engineering (VE) study prepared by HNTB included an analysis and refinement of the original ABE hydrology model to identify means of reducing peak flows. This section describes these refinements and their effects on predicted storm flows at the Inner Loop Corridor.

Throughout the evaluation and design phases which followed the VE study, numerous additional modifications were made to the hydrology model. These changes and their effects are also described below.

A. VE Report Refined Model

Initial review of the original ABE hydrology model identified three major conceptual refinements to be made to the model. The following paragraphs describe these modifications and their cumulative effect on the hydrology.

The area studied for this refinement was the same as that studied in the original hydrology report. The area is bounded on the west by I-17 (Black Canyon Freeway), on the north by the Arizona Canal, on the east generally by a line from 44th Street and Campbell Avenue to 32nd Street and McDowell Road, and on the south by the I-10 corridor.

1. Conceptual Refinements

The first conceptual refinement was the reduction in size of the individual drainage areas. In the ABE Report, areas one-half square mile or larger were used. In this refinement, sub-areas of one quarter square mile (one-half mile square) were generally used, providing more uniform sub-area size and shape. This

tended to reduce bias which could have been introduced through the application of uniform input parameters, such as flow velocity, to non-uniform areas. This also eliminated bias which was introduced in the ABE Report for the area west of 24th Street, where surface flows were diverted to the west at one mile intervals and to the south at one half mile intervals.

Procedures for determining drainage area characteristics and routing parameters between sub-areas were consistent with those used in formulating the original ABE model previously described.

The second conceptual refinement was to incorporate the effect of storage and conveyance along the north overbank of the Grand Canal. The ABE Report assumed no storage or conveyance impact on flows by the Grand Canal. However, further analysis was prompted by review of the City of Phoenix Flood Insurance Study, which indicated storage north of the canal, and a Corps of Engineers report on the 1972 flood, which indicated flows to the west along the north overbank. The Corps of Engineers step-backwater computer program HEC-2 was used to analyze storage and conveyance capacities for the north overbank of the Grand Canal. The HEC-2 output data were converted to rating curves for use in the TR-20 model.

The third conceptual refinement involved the capacities of the existing City of Phoenix major storm sewers between McDowell Road and the I-10 right-of-way. In the ABE model, the capacities of the sewers north of McDowell Road were calculated for the 50-year flow using a hydraulic gradient equal to the existing ground slope, assuming surcharged conditions in the pipes. The original model then increased the hydraulic gradient south of McDowell to reflect an assumed three foot drop in the hydraulic grade line at the east-west freeway collection system. This assumed drop in the hydraulic grade line was not considered valid in the refined model, since the only way to physically achieve the drop would be to re-lay the entire section of the storm sewer system between McDowell Road and the Inner Loop east-west collector. Storm

sewer capacities were therefore calculated using ground slope for hydraulic gradient for this section of sewer in the refined model, resulting in a shift of approximately 540 cfs of peak flow from the storm drainage system to surface flow. The capacities of the major storm sewers to the north of McDowell Road were not changed in the refined model.

2. Results

Column B of Table 2-2 lists the peak discharges at concentration points along McDowell Road from the ABE model and from the refined model. It can be seen that the peak discharges and flow distributions along McDowell Road were significantly changed due to the effects of flow along the Grand Canal, the revised storm drain capacities and the introduction of additional points of diversion.

B. VE Report Combined Model

One element of the hydrologic investigations in the VE study included the determination of the quantity and location of long-term return period runoff from the off-site drainage area affecting the north-south corridor of the I-10 Inner Loop. It was intended to be an extension of the hydrology modeling conducted by ABE in their report.

Although the north-south leg hydrology was originally developed as a separate TR-20 model, early results showed an inter-relationship with the previous modeling work for the east-west corridor. Therefore, the models were combined to encompass the entire Inner Loop Corridor drainage area.

1. Drainage Area

The drainage area for the combined model was expanded to the east and south, to include the southern slope of Camelback Mountain

and most of the area north of the Salt River between 20th Street and the New Cross Cut Canal. Figure 2-3 shows the expanded drainage area and the sub-area boundaries.

West of 40th Street, the northern boundary still follows the Arizona Canal Diversion Channel. From 40th Street, the boundary follows the Camelback Mountain ridge to 64th Street, then extends down to 68th Street and the Arizona Canal. The eastern boundary follows the New Cross Cut Canal from the Arizona Canal to approximately Thomas Road, then runs southwest along a ridge to the Old Cross Cut Canal, near the northeast corner of Sky Harbor International Airport. The southern drainage area boundary cuts across the east end of the airport to 32nd and Watkins Streets, then extends west to the I-10/I-17 interchange (the Maricopa Interchange).

2. Sub-Area Breakdowns

For the most part, drainage patterns in the expanded drainage area are controlled by the major street patterns and the canals, similar to the original drainage area. For the area north of the Arizona Canal, drainage is controlled by the steep ground slopes of Camelback Mountain, resulting in less uniform drainage sub-areas. Sub-areas east of the Old Cross Cut Canal are also somewhat irregular, as drainage patterns are controlled by the topography in the higher areas to the east, but street patterns control drainage closer to the canal.

The proposed Papago East Freeway was also considered in establishing the sub-area breakdowns. The highway corridor was used as a sub-area boundary, on the assumption that the road would be built on an embankment forming a barrier to overland flow.

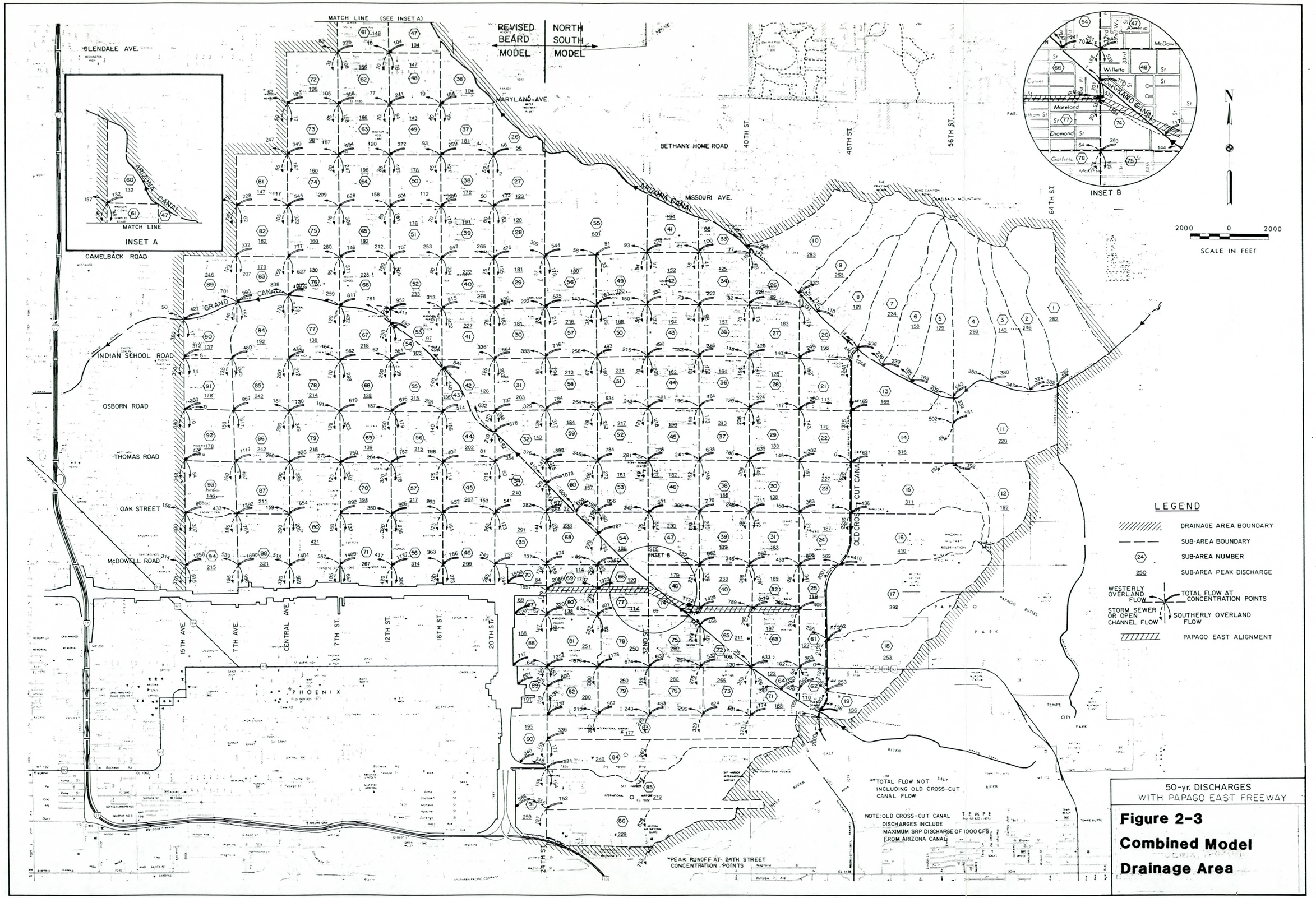


Figure 2-3
Combined Model Drainage Area

3. Sub-Area Characteristics

For the most part, sub-area characteristics for use in the combined model were determined by the procedures described above for the ABE model. The principal exception to these procedures came in the determination of times of concentration. For the area north of the Grand Canal and parts of the area east of the Old Cross Cut Canal, sub-area times of concentration were based upon the SCS modified curve number method, where:

$T_C = 1.67 \times LAG$ and

$$LAG = \frac{L^{.8}(S+1)^{.7}}{1{,}900Y^{.5}}$$

With:

L = hydraulic length of sub-area in feet

$$S = \frac{1{,}000}{CN} - 10$$

CN = runoff curve number

Y = average watershed land slope in percent.

T_C = Time of Concentration

The T_C from these equations was then modified by a lag factor (LF) for percent of impervious area in the sub-area.

$$LF = 1-(PRCT \times [-(6.789 \times 10^{-3}) + (3.35 \times 10^{-4} \times CN) - (4.298 \times 10^{-7} \times CN^2) - (2.185 \times 10^{-8} \times CN^3)])$$

With:

PRCT = percent of impervious area.

T_c's for some sub-areas east of the Old Cross Cut Canal were based upon the modified curve number method for the higher, less developed portion of the longest flow path, and the gutter flow velocity method for the lower, completely urbanized portion, using the sum of the T_c's from the two methods.

4. Routing Procedures

Routing of surface flows between sub-areas followed the procedures described in the ABE report, with the exception that the effect of storage behind the canals was modeled by the TR-20 RESVOR subroutine. Flows were routed through storage areas by the storage indication method, using data from a depth-storage-outflow table. Routing statements were also added to the model to route the southbound surface flow hydrographs from McDowell Road to the I-10 corridor.

Existing and proposed City of Phoenix storm drains were incorporated into the combined hydrology model following the same procedures used in the original model.

5. Effects Of Existing Canals

The Arizona Canal, the Grand Canal, and the Old Cross Cut Canal all serve as barriers to runoff as well as surface flow conveyance features. Therefore, each canal was analyzed to determine its effect upon the upstream sub-area runoff hydrographs along the canal.

Data were acquired for all three canals within the Inner Loop drainage area. These data were analyzed and input to the Corps of Engineers step-backwater computer program HEC-2. The resulting computer models were used to determine potential conveyance and storage above each canal's banks.

For the Grand Canal, the area behind the upstream bank was analyzed. For the Arizona Canal, the area east of 40th Street and north of the downstream bank was analyzed, because the downstream bank is generally 2-3 feet higher than the upstream bank. The Arizona Canal and the Grand Canal have limited capacities, and were assumed to be full to the lower bank elevation with runoff from outside the drainage area. Figure 2-4 shows typical sections of the Arizona and Grand Canals.

The Old Cross Cut Canal is the sole major surface drainage feature lying entirely within the drainage area. Therefore, in addition to the area behind its banks, the canal itself was modeled, including the culverts which carry canal flow across streets and a series of drop structures. Figure 2-5 shows cross-sections of the Old Cross Cut Canal.

The City of Phoenix and the Salt River Project have an arrangement whereby the Salt River Project is allowed to discharge 1000 cfs from the Arizona Canal into the Old Cross Cut Canal during major storms, with the Old Cross Cut Canal's remaining capacity handling runoff from the area directly to the east of the canal. The canal discharges to the Salt River, so the analysis of the canal was based upon a 10-year tailwater in the river and a base flow of 1000 cfs in the canal.

While all three canals present an effective general barrier to runoff, there are major breakout areas where flow trapped behind the canal banks escapes into the lower basins. Along the Old Cross Cut Canal, where the capacity of the canal is controlled by the capacity of the culverts conveying flow under the crossing roadways, flows in excess of the culvert capacities escape the canal and move westward. An example of this would be the area just north of McDowell Road and the Old Cross Cut Canal, where flow in excess of the capacity of the culvert under McDowell will overflow the canal banks and escape westward. Surveys of canal banks and crossing streets, along with contour maps and

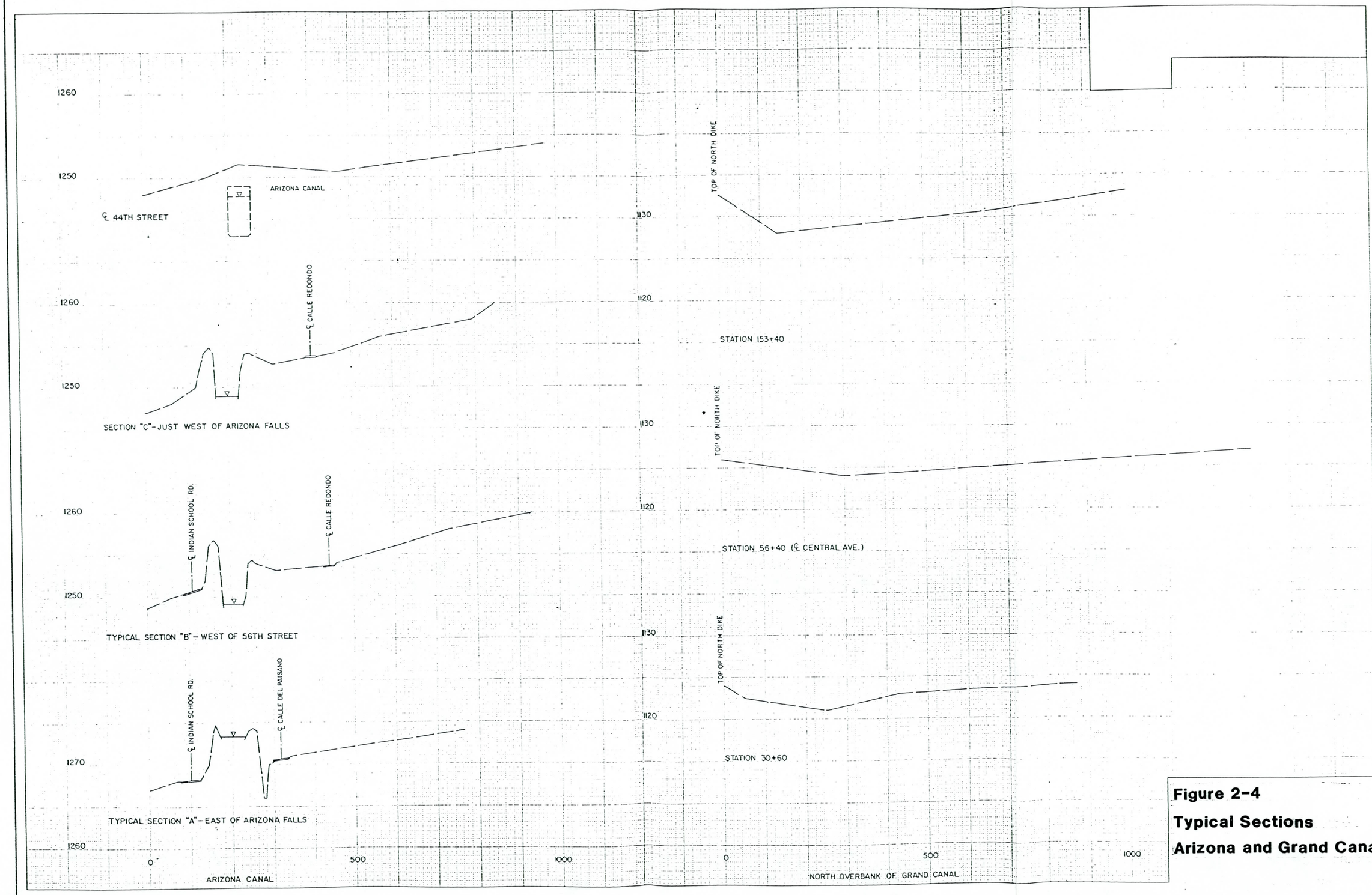


Figure 2-4
Typical Sections
Arizona and Grand Canals

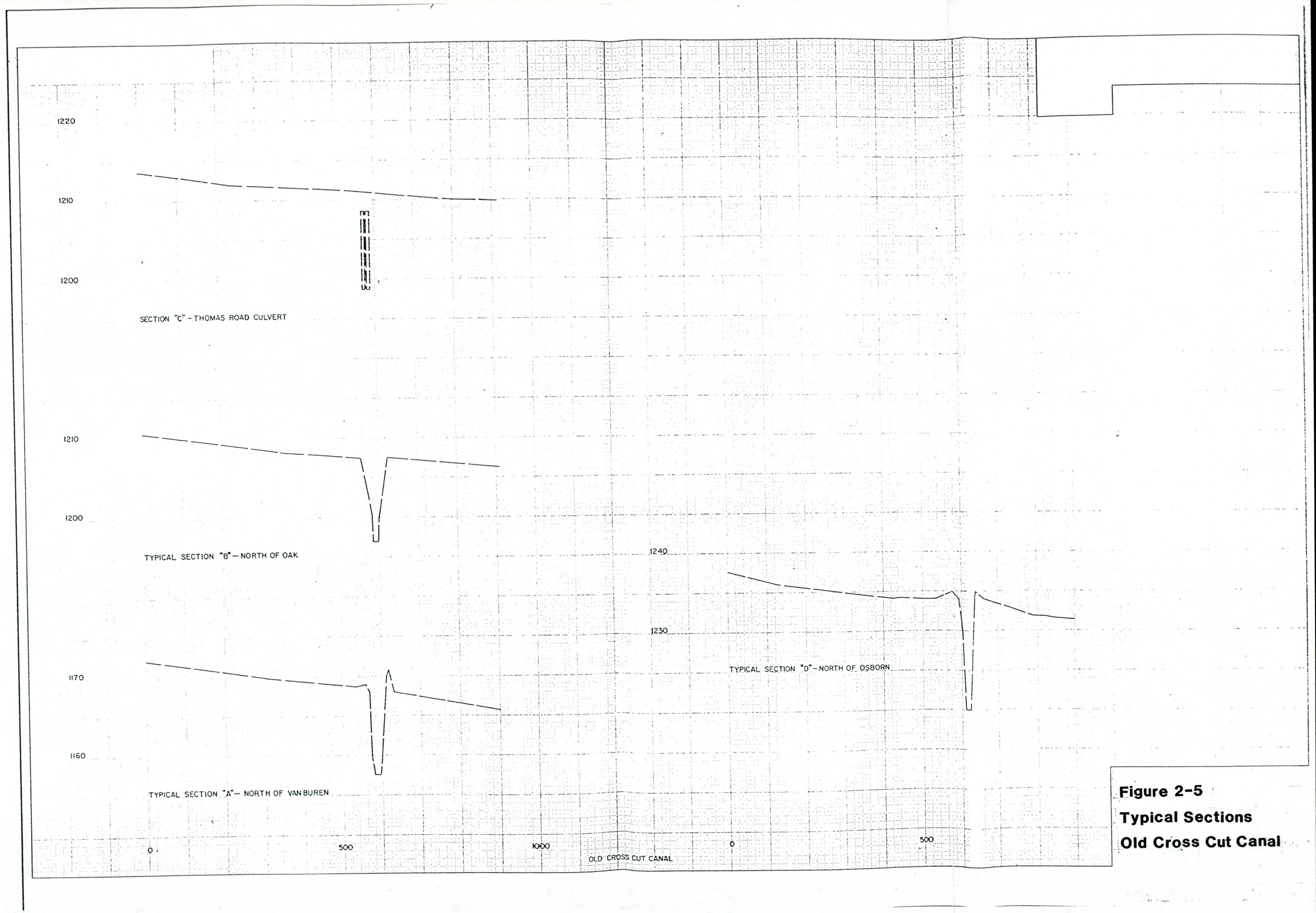


Figure 2-5
Typical Sections
Old Cross Cut Canal

construction plans, were used to determine the breakout points. Rating curves for the breakouts were generally developed by using the weir equation.

In situations where some flow would still be contained and conveyed behind the canal banks past the breakout point, the breakout point rating curve was input to the TR-20 computer model, as well as a rating curve for conveyance behind the canal at that point. A divert routine in the model determined the flow split using the pair of rating curves, splitting the hydrograph of the approaching flow from behind the banks into a breakout hydrograph and a hydrograph for continuation of flow behind the canal banks.

Depending upon the configuration of the area behind the canal banks, either reservoir routing or channel routing was used in the TR-20 model to analyze the attenuating effects upon the hydrographs of the conveyance behind the canal banks.

Where flow had escaped the canal banks, the hydrographs for the breakout points were routed through city streets to the next downstream concentration point.

6. **Squaw Peak Parkway**

At the time the combined model was formulated, the Squaw Peak Parkway was in the conceptual planning stages, and was envisioned to be an at-grade roadway with two-year capacity storm drains, per standard City of Phoenix practice.

A prime hydrologic/hydraulic effect of this improvement would be the restriction of conveyance north of the Grand Canal due to an approach fill. The model was formulated with fill in place which would divert all conveyance along the canal's northern bank to the south at 20th Street. Due to the distribution of this flow through routing via city streets, the effects were negligible at McDowell Road.

7. Papago East Freeway

Drainage patterns along the Pagago East Corridor were modified in the TR-20 model to account for the effects of the freeway, which will be built on a low embankment with underpasses at major city streets. The routing logic was based on the assumption that flow in city storm drains intersecting the corridor would continue southward, but all surface flow in excess of the storm drain capacity would be intercepted at the Papago East embankment and conveyed westward to the Inner Loop drainage system.

This increases the peak flow rate at the Inner Loop by about 1,250 cfs. The increase in peak flow is entirely a result of the path followed by the runoff, and the total volume of runoff reaching the Inner Loop is unchanged. The area affected by this redistribution of flows lies between McDowell Road and the Southern Pacific Railroad.

Because of the possibility that the Inner Loop drainage system would be in operation prior to construction of Papago East, the hydrology model was run both with and without the Papago East Freeway in place.

8. Modeling Results

Columns C and D of Table 2-2 list the TR-20 peak 50-year discharges resulting from the combined hydrology model, both with and without the Papago East Freeway in place. Comparison with earlier results shows that the combined effects of the revisions described in this section were relatively minor.

C. Proposed Storm Drain Capacities

As mentioned in earlier discussions, capacities entered into the TR-20 model for the proposed City of Phoenix storm drains were

based on preliminary investigations of the two-year hydrology, performed during the formulation of the ABE model. When the model was refined and expanded during the VE study, the city had not yet completed its detailed investigations for these sewers, and the original capacities were retained.

In early 1984, following publication of the VE Report, the city advised that the hydrologic investigations had been completed, and that design capacities and diameters for the proposed sewers had been determined. The capacities provided by the city for the segments from McDowell Road to the I-10 corridor were 535 cfs at Central Avenue, 500 cfs at 12th Street, and 445 or 495 cfs at 20th Street, depending on the alignment selected for the Squaw Peak Parkway storm drain. The city also indicated its intention of installing 96-inch diameter pipe for the lower portions of all three storm drains. Following the procedure described earlier for modeling of storm drains, the revised capacities entered into the model were based on 96-inch diameter pipe at a hydraulic gradient equal to the average ground slope over the pipe reach. For the segments between McDowell Road and I-10, these figures were 383 cfs at Central Avenue, 296 cfs at 12th Street and 532 cfs at 20th Street. Capacities for upstream segments were determined in similar fashion, using pipe diameters provided by the City. The results of these revisions are shown in Column E of Table 2-2.

D. Squaw Peak Parkway Hydrology

1. Effects on Surface Flows

In late 1984, as the planning for the Squaw Peak Parkway progressed, the preliminary roadway profile was reviewed to determine its effects on surface flows in the area and hence the flows reaching the Inner Loop corridor. The capacities previously determined for the Squaw Peak/20th Street storm drain had arisen from studies related to the highway planning, and were

not changed. The modifications were limited to the diversions of overland flow along 20th Street as follows:

Bethany Home Road - No change
Missouri Avenue - Westbound surface flow removed
Camelback Road - Westbound surface flow removed
Campbell Avenue - No change
Indian School Road - Westbound surface flow reduced by 20%
Osborn Road - No change
Grand Canal - Westbound surface flow reduced by 30%
Thomas Road - Westbound surface flow reduced by 20%
Oak Street - No change
McDowell Road - Westbound surface flow reduced by 20%

The effect of these changes on southbound 50-year surface flows along the Inner Loop corridor was minor, as shown in Column F of Table 2-2.

2. Effects of Phased Construction

Given the uncertainties of the schedule for construction of the Squaw Peak Parkway with respect to the completion of the Inner Loop drainage system, the TR-20 model was modified to determine the effects on the hydrology if the Inner Loop system were in operation prior to completion of Squaw Peak. This was done by removing the storm sewer diversions along 20th Street, so that surface drainage would follow existing paths through city streets. The surface flow diversion modifications discussed in the previous paragraphs also did not apply, since this run assumed that the Squaw Peak Parkway did not exist.

As shown in Column G of Table 2-2, the removal of the 20th Street storm drain increased the surface flows at each major street along the east-west corridor, although the effects are relatively

minor west of Central Avenue. The additional surface flow is approximately equal to the 530 cfs surcharged capacity of the Squaw Peak/20th Street storm drain. North-south flows were not affected by this change.

E. Corrected Model

During review of the hydrology modeling, two keypunch omissions were discovered which had persisted since the original modeling in the VE Study. These omissions had resulted in the loss of two hydrographs in upstream portions of the drainage area. After the model had been re-entered the Squaw Peak hydrology was rerun for the 50-year storm, with the results shown in Column H of Table 2-2.

F. Interim Conditions

An additional interim condition was modeled, based on the assumption that the proposed storm drains at Central Avenue and 12th Street would not be built until after the Inner Loop drainage system and the Squaw Peak/20th Street storm drain were in place. The Squaw Peak surface hydrology revisions were applied to this model, as were the corrections discussed above.

The increase in east-west surface flow for this condition ranged from 4 percent at 7th Street to 13 percent at 15th Avenue, as shown in Column I of Table 2-2.

G. Central Corridor Hydrology

The City of Phoenix is considering a plan to increase the capacity of the storm drainage system in the Central Corridor. The area under consideration is bounded by 12th Avenue and 12th Street on the west and east, the I-10 Inner Loop on the north, and the Salt River on the south.

Because the existing storm drainage system in this area is inadequate to collect and transport the 2-year storm runoff, the report "Phoenix Central Avenue Corridor Storm Sewer Study - Project No. ST-818930 Preliminary Report", prepared by Benson and Gerdin, recommended the following improvements:

- Construct a trunk storm drain along 3rd Avenue from Roosevelt Street to the Salt River.

- Construct a relief storm drain along Grant Street from 3rd Street to the 3rd Avenue trunk line.

- Construct a relief storm drain along Van Buren Street from 5th Street to the 3rd Avenue trunk line. This line can be extended east to 9th Street as a lateral.

- Construct a relief storm drain along Taylor Street from 7th Avenue to 9th Avenue.

The feasibility of connecting the proposed Central Corridor system to the West Tunnel rather than constructing an additional outfall to the Salt River was also investigated. The TR-20 model used for the Benson and Gerdin report was modified for this study by substituting the rainfall distribution used in the offsite drainage model. This modification increased some local sub-area runoff peaks, but the overall effect was negligible due to routing effects.

The model was further modified to provide inlets to the West Tunnel at Van Buren, Grant, and Watkins Street. This resulted in the following system:

- Grant Street (East) relief storm drain from 3rd Street to West Tunnel.

- Van Buren Street (East) relief storm drain from 5th Street to West Tunnel extended east as lateral to 9th Street.

- Storm drain along 3rd Avenue from Roosevelt Street south to Van Buren Street, then east to West Tunnel.

- Storm drain along 3rd Avenue from south of Van Buren Street south to Grant, then east to the West Tunnel.

- Storm drain along 3rd Avenue from south of Grant Street, south to Watkins Street, then east to the West Tunnel.

This model was run for 2-, 5-, 10-, and 25-year discharges.

In December 1983, the City of Phoenix requested the following four drop shaft locations and capacities, presumably as a result of further study by Benson and Gerdin, assuming a tunnel outlet:

Location	Discharge
Fillmore	290 cfs
Grant	240 cfs
Tonto	100 cfs
North of Maricopa	50 cfs

Rather than modify and rerun the HNTB hydrology model to reproduce these flows, it was decided to use hydrographs truncated at these values to represent the City of Phoenix flow to the tunnel, because only storm sewer flows with no additional overland flow were to be allowed into the tunnel system from the areas in question.

In a letter dated April 24, 1984, the City of Phoenix amended their earlier request by asking for the provision of the following:

STATION	LOCATION	CONNECTION SIZE	DIRECTION
24+20	Maricopa Freeway	48" (60 cfs)	West
57+70	Tonto	72" (135 cfs)	West*
71+60	Grant	78" (170 cfs)	East
120+25	Fillmore	72" (140 cfs)	West*

Subsequent to the receipt of the amended City of Phoenix request, the truncated hydrographs representing city flows were reduced to the above values.

* Later modified with City of Phoenix approval to connect from the East side. All tunnel connecting conduit sizes were revised to 42 inches.

III. ASSUMPTIONS

The development of the hydrology model for the Inner Loop off-site drainage area involved many assumptions in addition to those inherent in the basic computer modeling procedure. Some of these assumptions relate to specific features of the drainage area, while others are more general in nature.

The more significant general assumptions are summarized as follows:

- The 50-year design storm occurs simultaneously over the entire Inner Loop drainage area.
- Runoff curve numbers are based on zoning classification.
- Flood-irrigated residential areas with bermed yards will not contribute runoff up to and including the 100-year storm.
- Drainage sub-area times of concentration are based on the average full-flow gutter velocity.

- Storm drain capacities for the 50-year storm are based on a hydraulic grade line parallel to the average ground slope (uniform flood depth).

- Storm drain routing is based on a uniform velocity of seven feet per second.

- "Minor" City of Phoenix storm sewers with diameters of 24 inches or less are ignored.

- The City of Phoenix will continue to build major storm drains at 1/2-mile intervals.

- Except as previously discussed, all proposed storm drains exist for modeling purposes.

- Capacities for proposed storm drains are based on pipe sizes provided by the City of Phoenix where known.

- Capacities for remaining proposed storm drains are based on 2-year hydrology.

- Overland flow is restricted to the major 1/2-mile streets.

- Overland flow splits at intersections are based on the relative slopes of the downstream street sections.

- Overland flow routing is based on a Manning's "n" factor of 0.015.

- Street crowns and curbs are at the same elevation. Street rights-of-way outside the curbs are paved and level.

- Runoff south of McDowell Road and west of 24th Street is included by adding those areas to the drainage areas which have concentration points along McDowell Road.

Assumptions regarding specific portions or features of the drainage area include the following:

- The Squaw Peak Parkway will affect surface flow diversions as described earlier in this chapter.

- The Papago East Freeway storm drain will not intercept any existing or proposed storm drains from the north.

- The Papago East Freeway will block overland flow except at underpasses at 24th, 32nd, 40th, and 44th Streets and at the Grand Canal. Overland flow at these locations will be intercepted by the Papago East storm drain.

- The Arizona, Grand and Old Cross Cut Canals, their banks and the area behind their banks will remain essentially unchanged.

- The Grand Canal and the Arizona Canal will flow full (to top of lower bank) during major flooding events, conveying storm water from outside the offsite drainage area.

- The U.S. Corps of Engineers will construct the Indian Bend Wash Side Channels System, in addition to the Arizona Canal Diversion Channel.

- The Arizona State Hospital Farm and adjacent developed area will at the least undergo residential development. To allow somewhat for the possibility of a zoning change due to the proximity to the proposed Papago East roadway, no retention was assumed.

- The Phoenix Military Reservation and Papago Park will not undergo extensive development.

- The area west of Sky Harbor International Airport will undergo complete redevelopment, eventually becoming fully developed with no retention.

- The two major airport storm sewers (60" and 84") discharging to the Salt River adjacent to the 32nd Street storm sewer, plus the increase in capacity of the 32nd Street storm sewer through Sky Harbor International Airport, will intercept all runoff generated to the east of their alignment, in the southern three sub-areas of the airport.

- The western major storm sewers in the airport which tie into the 24th Street storm sewer will not function at the storm peak due to lack of head and lack of additional capacity in the 24th Street storm sewer.

CHAPTER 3 – DESIGN FLOWS

SYNOPSIS

This chapter presents a summary of the peak flows used in the design of the various components of the Inner Loop storm drainage system. Design flows for drop shafts, drop structures, pump stations, collector systems and outfall structures are tabulated. The derivation of the design flows is covered elsewhere in this report.

I. ON-SITE COLLECTION SYSTEM

Collection and disposal of runoff generated within the I-10 right-of-way is provided by a series of collection and conveyance systems, all of which eventually discharge to the main drainage tunnels. The on-site drainage system connects directly to tunnel drop shafts in areas of the freeway which are at-grade or elevated. In depressed areas, runoff is pumped into either the tunnel system or the Storm Water Interceptor.

Location and interception capacities of individual inlets for the on-site drainage system were determined using procedures outlined in Chapter 4. Pump stations were designed to handle the 50-year storm using procedures outlined in Chapter 5. Table 3-1 summarizes the design inflows for the pump stations.

TABLE 3-1

Pump Station Inflows

STATION	INFLOW
3rd Avenue	166 cfs
3rd Street	185 cfs
16th Street	182 cfs
Maricopa Interchange	21 cfs

II. OFF-SITE COLLECTION SYSTEM

Storm runoff originating outside the Inner Loop right-of-way is intercepted along the northern and eastern sides of the freeway. The primary computer modeling done to determine the off-site flows is described in Chapter 2.

The TR-20 computer model generated surface flow values at one-half mile intervals along the upstream side of the Inner Loop corridor. The surface flow into the corridor at these points represented all overland flow in the same direction as the major street flow, whether it occurred as street flow, alley flow or sheet flow between streets.

In order to design the off-site flow collection system, the surface flow peaks at the nodes had to be distributed to all streets, alleys, and open areas between streets. The surface flows at each node were first allocated among the side streets using a parabolic flow distribution. This procedure assumes that the flow spreads out among the side streets for 1/2-mile on either side of the major street, with the flow at each side street decreasing with the square of its distance from the major street. The sum of the allocated peak flow rates over the 1-mile distance is equal to the concentrated peak flow at the nodal point on the major street. The total peak flow at any side street is the sum of the allocated flows from the next major street in either direction.

Table 3-2 shows the design street flows used for the north-south leg of the collector system. These flows are a result of the hydrologic modeling done with the assumption that the Papago East drainage system is in place. A safety factor of 1.25 was applied to the model results to compensate for uncertainties in the model.

In addition to the allocation procedure described above, a separate analysis was performed to further distribute flows to alleys and open areas between buildings along the east-west leg. Aerial photographs

TABLE 3-2

NORTH - SOUTH LEG
COLLECTION SYSTEM FLOWS

(A) LOCATION	(B) STREET FLOW (cfs)	(C) DESIGN FLOW (B X 1.25)
Moreland	5	6
Portland	15	19
Roosevelt	35	44
Garfield	30	38
McKinley	30	38
Pierce	40	50
Fillmore	60	75
Taylor	100	125
Polk	150	188
Van Buren	235	294
Monroe	145	181
Adams	170	213
Washington	335	419
Jefferson	190	238
Madison	85	106
Jackson	25	31
Harrison	5	6
Buchanan	10	13
Lincoln	35	44
Grant	70	88
Sherman	120	150
Buckeye	170	213
Pima	165	206
Cocopah	175	219
Mohave	250	313

were obtained for the areas between the Inner Loop corridor and the next nodes "up-basin". Street, alley, clear street right-of-way and open area widths were determined from the aerial photographs. These widths of flow were combined with observations of up-basin street patterns and engineering judgement to arrive at peak flows for the offsite drainage system design. Because of the approximate nature of this distribution procedure, designers used a safety factor of 1.25 for inlet design capacities. Table 3-3 summarizes the design flows used for the east-west collector system. These design flows were derived from the original hydrology in the VE Report using the allocation procedures described.

III. STORM WATER INTERCEPTOR FLOWS

The Storm Water Interceptor (SWI) was designed to convey the 50-year flows from the east-west collector system to the drainage tunnels. Drop structures at 7th Avenue and 10th Street discharge SWI flows to the North Tunnel, while the Moreland Street drop structure picks up the portion of the SWI flow from 19th Street to the east. SWI design is covered in Chapter 6.

Table 3-4 summarizes the design inflows for the SWI. These were calculated by adding the street, sheet, and storm sewer flows from Table 3-3.

Design flow for the Squaw Peak storm drain between Station 3+79, where a 60" RCP line joins the storm drain, and the junction with the SWI is 737 cfs. Upstream of station 3+79, the design flow of the Squaw Peak storm drain is 532 cfs. Refer to Figure 4-2, Sheet 18.

The design flow for the segment of the SWI between 19th Street and the Squaw Peak junction is 324 cfs. Between the Squaw Peak junction and the Moreland Street drop structure, the design flow for the SWI is 1117 cfs.

TABLE 3-3

EAST - WEST LEG
COLLECTION SYSTEM FLOWS

LOCATION	STREET FLOW (cfs)	SHEET FLOW (cfs)	SEWER (cfs)
15th Avenue	---		
		35	
13th Avenue	110		
		25	
11th Avenue	115		
		35	
9th Avenue	175		115 (155)*
		60	
7th Avenue	300		
		60	
5th Avenue	170		
		35	
3rd Avenue	100	35	
		35	
Central Avenue	135		385(535)*
		35 (Alley)	
1st Street	115		
		25	
2nd Street	90		
		25 (Alley)	
3rd Street	75		
		50	
5th Street	85		
		75	
7th Street	145		195(141)*
		20	
		50	
9th Street	60		
		50	

TABLE 3-3 (CONTINUED)

LOCATION	STREET FLOW (cfs)	SHEET FLOW (cfs)	SEWER (cfs)
10th Street	60		
		0	
11th Street	60		
		15	
12th Street	80		295(500)*
		15	
13th Street	50		
		10	
14th Street	40		
		5	
15th Street	55		
		10	
16th Street	80		130(118)*
		15	
17th Street	50		
		10	
18th Street	45		
		15	
19th Street	60		
		20	
20th Street			

* Figure in parenthesis is capacity that should be available for storm sewer flow if there is no overland street or sheet flow.

TABLE 3-4

STORM WATER INTERCEPTOR FLOWS

A. Segment 1 (from 15th Avenue to 7th Avenue)

Location	Total Inflow (cfs)	Cumulative Flow (cfs)
15th Avenue		
13th Avenue	145	145
11th Avenue	140	285
9th Avenue	325	610
7th Avenue	420	1030

B. Segment 2 (from 19th Street to 9th Street)

Location	Total Inflow (cfs)	Cumulative Flow (cfs)
20th Street		
	20	20
19th Street		
	75	95
18th Street		
	55	150
17th Street		
	65	215
16th Street		
	220	435
15th Street		
	60	495
14th Street		
	50	545
13th Street		
	65	610
12th Street		
	390	1000
11th Street		
	60	1060
10th Street		
	110	1170
9th Street		
	60	1230

IV. APPURTENANT STRUCTURE FLOWS

A. Drop Structures and Drop Shafts

Drop structures convey flows from the SWI to the North and East Tunnel at the upstream ends of the tunnels. Drop shafts convey runoff from local collector systems into the North, East, and West Tunnels at points along the tunnels.

Three drop structures and 14 drop shafts are used in the Inner Loop drainage system. Design flows and contributing areas are summarized in Table 3-5. Chapter 7 covers design procedures for drop shafts and drop structures.

B. Second Street Junction Structure

Design flows into the 2nd Street junction structure are a result of the accumulated flows in the east and west branches of the SWI and flows discharged from the 3rd Avenue, Central Avenue, 3rd Street and 7th Street drop shafts. The design flow into the west side of the junction structure is 2220 cfs. The design flow into the east side of the junction structure is 2020 cfs. Chapter 7 covers junction structure design.

V. OUTFALL STRUCTURE FLOWS

Each outfall structure is designed to convey a peak flow of 5000 cfs. This is sufficient to handle the 50-year outflow from the tunnels. Outfall structure design is covered in Chapter 8.

Both the East and West Tunnel outfalls are provided with a pump station. These pump stations are designed to dewater their respective tunnels in approximately 7 days, depending on the pumping schedule. Each pump station has 2 submersible electric pumps with a capacity of 3,300 gpm each, for a total station capacity of 6,600 gpm.

TABLE 3-5

DROP STRUCTURE AND DROP SHAFT FLOWS

Drop Structures

Location/Design Flow	Contributing Area
7th Avenue/1032 cfs To North Tunnel	For box culvert from west (draining surface runoff and storm sewers intercepted east of 15th Avenue and west of 7th Avenue), for 7th Avenue street flow and for sheet flow from between 7th and 5th Avenues.
10th Street/1326 cfs To North Tunnel	For box culvert from east (draining surface runoff and storm sewers intercepted west of 20th Street and east of 9th Street) and for 9th Street street flow plus 50 cfs sheet flow between 7th Street and 9th Street.
Moreland Street/2573 cfs To East Tunnel	For box culvert from future Papago East, Squaw Peak storm drain, pipe from 20th Street, and for Moreland and Portland Streets street flow.

Drop Shafts

Location/Design Flow	Contributing Area
3rd Avenue/510 cfs To North Tunnel	For street flow intercepted from 5th Avenue and 3rd Avenue and Central Avenue, plus sheet flow between 5th Avenue and 3rd Avenue and between 3rd and Central Avenues, plus discharges from 3rd Avenue pump station.

TABLE 3-5 (CONTINUED)

Drop Shafts

Location/Design Flow	Contributing Area
Central Avenue/535 cfs To North Tunnel	For storm sewer flow in Central Avenue.
3rd Street/500 cfs To North Tunnel	For street flow from 1st Street, 2nd Street, 3rd Street, and 5th Street, and in between 1st and 5th Streets, plus discharge from 3rd Street pump station, plus flow in alley between Central Avenue and 1st Street.
7th Street/435 cfs To North Tunnel	For street and storm sewer flow from 7th Street, plus sheet flow between 5th Street and 7th Street, plus 20 cfs sheet flow from east of 7th Street.
Pierce Street/245 cfs To East Tunnel	For street flow intercepted from Roosevelt, Garfield, McKinley, Pierce, and Fillmore Streets.
Taylor Street/368 cfs To East Tunnel	For street flow intercepted from Taylor and Polk Streets, and half the street flow in Van Buren Street.
Adams Street/600 cfs To East Tunnel	For half the street flow in Van Buren and Washington Streets, and for street flow intercepted from Monroe and Adams Streets.
Madison Street/468 cfs To East Tunnel	For street flow intercepted from Jefferson, Madison, and Jackson Streets, and half the street flow in Washington Street.

TABLE 3-5 (CONTINUED)

Drop Shafts

Location/Design Flow	Contributing Area
Buckeye Road/485 cfs To East Tunnel	For street flow and surface flow intercepted from Harrison Street southward to just north of Pima Street.
Mohave Street/540 cfs To East Tunnel	For street flow intercepted from just north of Pima Street to and including Mohave Street flow, plus storm sewer flow from south (Maricopa Interchange).
Fillmore Street/140 cfs To West Tunnel	For City of Phoenix storm sewer flow.
Grant Street/170 cfs To West Tunnel	For City of Phoenix storm sewer flow.
Tonto Street/135 cfs To West Tunnel	For City of Phoenix storm sewer flow.
Maricopa Freeway/60 cfs To West Tunnel	For City of Phoenix storm sewer flow.

CHAPTER 4 - COLLECTION SYSTEM DESIGN

SYNOPSIS

This chapter provides a discussion of design for the on-site and off-site storm water collection systems. On-site drainage is the result of direct rainfall within depressed portions of the project right-of-way; off-site drainage is the result of overland flow from outside the project right-of-way. The discussion includes a description of the major contributing flow areas, a listing of the design criteria supplied to the design consultants and a description of the design procedures used in sizing the storm water inlets and connecting pipes. A schematic diagram of the entire on-site and off-site integrated system is presented at the end of this chapter.

I. GENERAL PROJECT BACKGROUND

The I-10 Inner Loop is depressed below the existing grade between 11th Avenue and Fillmore Street. It disrupts existing city storm sewers flowing to the south across the project corridor. Because of this, provisions have been made to intercept the storm water flows from the north, along the northern project right-of-way. These flows are collected and conveyed in a closed conduit to the Salt River. The existing city storm sewers south of the project will continue to function with reduced flows.

The on-site flows are a result of direct rainfall within the project's depressed section. These were determined by the Rational Method. The off-site component is from overland flow entering the project from the north and east. The overland flows were determined using the United States Soil Conservation Service (SCS) TR-20 computer program.

II. SUMMARY OF COLLECTION SYSTEM DESIGN

A. On-Site Flow Collection

1. General Overview

The on-site storm drain system is designed to intercept storm runoff generated within the depressed freeway, ramps, bridges, and HOV lanes. The design storm is the 50-year frequency storm event. Catch basin inlets are located in the gutterline and in depressed areas behind curbs and in the medians. Where necessary, slotted drains have been added to improve the interception capacity of inlets, and to decrease spread of flow on the pavement. For drainage design purposes the covered section has been treated as an open area.

The I-10 pavement drainage collected along the outside curbs is picked up in grated catch basins or downdrains. The runoff collected along the inside curbs is directed into median catch basins using curb openings nad spillways. Each series of inlets connects to the collector system which runs along the northern and eastern edges of the Inner Loop.

The median drainage system consists of a series of inlets (ADOT Standard C-15.80) sized to intercept 100% of the runoff from a 50-year storm with ponding to a maximum elevation 1.0 foot below the roadway shoulder.

The pavement drianage in fill areas along the ramps is collected in downdrains and conveyed to the areas bordered by I-10 and the ramps. These areas have been designated as retention areas, and the dikes have been sized to retain the 50-year flow with one foot of freeboard.

2. **15th Avenue to Central Avenue**

The storm drain system handling drainage from the freeway includes a trunk line located on the north side of the west bound lane between 11th Avenue and Central Avenue. Lateral lines pick up drainage from various points along the freeway and ramps and discharge into the trunk line. The trunk line carries the flows to the pump station located between Culver Street and the freeway, at 3rd Avenue. The pump station discharges into the North Tunnel via a drop shaft. Storm water from the elevated portion of the freeway west of 11th Avenue is intercepted, discharged into the Storm Water Interceptor and conveyed to the east.

Connections to the tunnels are made using drop shafts and larger capacity drop structures. The drop structure in this portion is located at 7th Avenue.

3. **Central Avenue to 16th Street**

Storm water intercepted east of 15th Street is conveyed to the 16th Street pump station for discharge into the Storm Water Interceptor. Storm water intercepted between 2nd Street and 15th Street is conveyed to the 3rd Street pump station and discharged into the North Tunnel. Water collected between Central Avenue and 2nd Street is conveyed to the 3rd Avenue pump station for discharge into the North Tunnel.

4. **16th Street to Roosevelt Street**

The storm drain system collecting dirainage from the freeway includes a trunk line located outside of the pavement along I-10 Eastbound north of Pierce Street. It crosses I-10 at approximately Station 7738 + 50 where it follows I-10 Westbound outside of the pavement. At various points along I-10 drainage

is discharged to the trunk line by lateral lines. The trunk line carriers the drainage to the pump station at 16th Street. The storm water is pumped into the Storm water Interceptor.

Drainage for the McDowell Road ramps is split into two separate trunk lines. One is located at Ramp McDowell-C, discharging into the 108" Sqaw Peak Storm Drain. The other, at Ramp McDowell-D, discharges intot he Storm Water Interceptor at 20th Street. An additional lateral at Ramp McDowell-C discharges into the I-10 trunk line.

The drainage for the East Papago Extension to 24th Street is conveyed by one trunk line which begins at the inlet on Ramp 24-B, Station 22 + 04, and runs north to the East Papago Storm Drain, a 10′ x 10′ box culvert. At various points along Ramps 24-A, 24-B and Lane E-W, drainage is collected and discharged into the trunk line by lateral lines. It flows north and is discharged into the East Papago Storm Drain. Drainage design for future Ramp E-S is accounted for in the final design.

5. **Roosevelt Street to Harrison Street**

Pavement drainage is intercepted by downdrain inlets or catch basins. Flows are carried down embankments in downdrain pipes and either allowed to flow overland to the nearest inlet or are conveyed by pipe to local storm drain systems.

Median spillways, aprons and catch basins are positioned to limit the pavement spread and provide proper spillway capacity. Spillways are designed for 100% catchment.

At Van Buren Street, pipes from the median catch basins connect to the primary North-South storm sewer which flows along the east side of I-10 to a drop shaft north of Van Buren Street. At

Washington and Jefferson Streets pipes from the median catch basins connect to new storm sewer pipes in Ramp 'D' and Ramp 'A' which flows to the primary North-South storm sewer.

6. **Harrison Street to Buckeye Road**

The area between the I-10 mainline and Ramp N-E at Buckeye Road is used as a retention basin. The inflow hydrograph used for sizing the basin was based on the Pima County Hydrograph synthesis method.

7. **Buckeye Road to the Maricopa Freeway**

Flows collected by the on-site storm water inlets are piped to a drop shaft at Mohave Street. This drop shaft is connected to the East Tunnel.

Inlets from the Maricopa Interchange east to 24th Street are connected by pipes to a graded site. This area is drained by an existing 48-inch pipe which is adequate for the projected 50-year runoff without surcharge. A 100-year rain would surcharge the outlet, but ample storage is available in the graded site. Even if the outlet were to be completely plugged (no outfall), the included area is capable of completely retaining runoff from a 100-year, 2-hour storm. Most of the inlets along the northeast quadrant of the interchange are too low to drain to the graded site. These flows are piped north to the drop shaft at Mohave Street. In the northwest quadrant of the interchange, there is another graded site which is used in the drainage system. Inlets around its perimeter discharge to the open area and flow to a new outlet at the west end. The pump station serving the depressed Ramp W-N also discharges to this graded area. The outlet for this area is a pipe which connects to the existing 60-inch pipe on the south side of I-17.

8. 3rd Avenue to 3rd Street (Covered Deck)

The portion of I-10 extending from 3rd Avenue to 3rd Street is covered with a concrete deck structure. The deck surface will eventually be developed as a park area by the City of Phonexis. The deck itself is part of the Inner Loop freeway package. Construction of the deck includes on-site drainage systems to remove runoff from the deck and the immediately surrounding area.

The finished elevation of the landscaping is below existing grade, creating a depression which collects and traps runoff generated within the deck park area. The total contributing area amounts to 27.9 acres, as shown in Figure 4-1. The on-site collection system has 3 components; a surface drainage system for the park area, an underdrain system which rests directly on the deck structure, and a freeway collection system on the highway level below the deck.

a. Deck Surface Drainage

The major surface drainage collectors are provided in the Central Avenue Plaza area, since this is the lowest area on the deck. Special inlets and drain pipes in the northern portion of the Plaza area collect runoff and convey it to a special drop structure located immediately east of the fountain equipment room to the north of the deck. This drop structure discharges into the freeway drainage system, which carries storm flow to the 3rd Avenue pump station. These surface collectors are sized to maintain a high-water elevation below 1077.8 for the 50-year design storm, and below 1079.5 for flows greater than the 50-year event. These elevations were set after consultation with representatives of Ammann and Whitney, the deck structural designers. They indicate that the design high-water elevations will not cause overloading of the concrete deck structure.

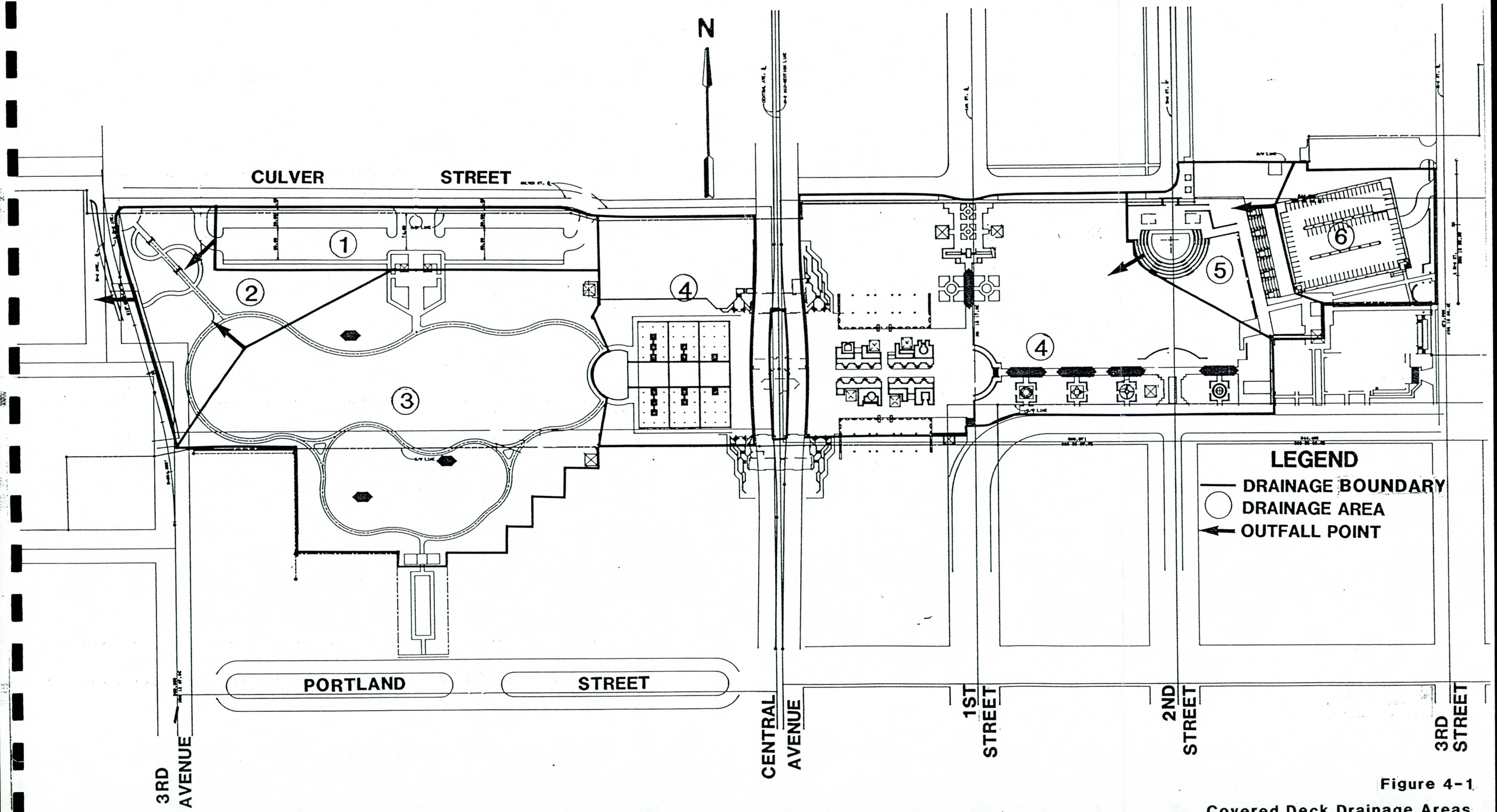


Figure 4-1
Covered Deck Drainage Areas

Additional overflow protection is provided by inlets inside the north fountain equipment room. These inlets are set at elevation 1078.1. The inlets inside the north equipment room are elevated above floor level 9 inches and covered with grates. Flows entering the room through holes in the curtain wall in sufficient quantity to build up to the grate elevation will then spill over into the inlets. This storm water is released directly onto the freeway shoulder by a spring tension gate which opens when water begins to spill into the inlet. This gate is expected to operate only during storm events in excess of the 100-year return period event. The depressed freeway will probably be closed during this event. Therefore, no additional risk should occur to travelers as a result of this overflow system. Water which does not overtop the elevated inlets will drain through the north foundation equipment room floor drains. The holes in the curtain walls are covered with filter fabric and function as air filters. The frames are hinged and pivot when storm water pushes on them.

The inlets adjacent to the south equipment room are set at elevation 1077.8. They appear as opening beneath benches set next to the equipment room wall. Water which enters these inlets is conveyed to the freeway drainage system by two special drop structures located between the south equipment room and the deck structure. This system should operate only during storms of a magnitude greater than the 50-year event.

Surface flows from east of the Central Avenue Plaza are conveyed to the plaza collectors by overland sheet flow. This will also occur when off-site flows between Central Avenue and 3rd Street in excess of the 50-year magnitude overtop the Culver Street curb line and move onto the deck. Drainage structures on the east half of the deck include a

catch basin in the depression provided for a future amphitheater and a collection system in the parking lot on the east portion of the deck. The amphitheater catch basin is connected to drop structure 444, just east of Ventilation Room #2, while the parking lot system is connected to drop structure 1117, on the south side of the deck. (See Figure 4-2, Sheet F). (See Figure 4-2, Sheet 7).

The area west of the Central Avenue Plaza is graded to route flows from the tree bosque to the Plaza area collectors. Most surface runoff generated west of the tree bosque flows overland to a depression on the south side of the park area. The remainder follows the pedestrian walkway under 3rd Avenue, where it is picked up by grate inlets at the 3rd Avenue pump station retaining walls.

At catch basin is located at the low point in the depression on the south side of the park area. This catch basin is connected to the nearby drop structure 413. The depression will function as a retention basin for overland flows. Revisions to drainage facilities may be required if the area is to become a permanent water feature in future park plans.

The off-site collection system from 3rd Avenue to Central Avenue has sufficient capacity to intercept 100-year return period flows. Storm events of greater magnitude may result in off-site runoff overtopping the Culver Street curb and flowing onto the deck. Finished grading of the park on the west portion of the deck will cause ponded water to begin overflowing, to the pedestrian underpass at elevation 1076.5. Stormwater runoff approaching the low point of the pedestrian underpass enters a 45 foot long grated catch basin at the low point close to the wall. This flow is conveyed under the walls to the freeway storm drain system. When the quantity

of stormwater approaching the catch basin is greater than what the freeway system can accept, the storm water will begin to pass through overflow release basins on the freeway side and run down the freeway. If the quantity of storm water approaching the low area continues to increase, water will begin to run through grated wall openings and then spill onto the freeway over the lower end of wall 7A-A(L5). This overflow system will prevent excess loading on the deck structure from ponded storm water.

b. Underdrain System

The under drain system provides drainage for the park soil which overlays the deck. It also provides protection for the waterproofing system and the concrete structure by routing subsurface water off the surface of the deck structure.

Individual deck units are generally sloped to either the north or south side of the structure. A divide occurs in some units which separates slopes to the north from slopes to the south. Grades range from about 0.25% to 10%.

A geotextile drainage board lies directly on the waterproofing. It covers most of the most of the the entire top surface of the deck structure. It does not cover the area under the north fountain equipment room, the depressions provided for future escalators in units 8 and 11, and the shallow trough which extends from the south edge of the deck along the Central Avenue centerline to the cutout for the future transit station. The drainage board conveys water

which percolates vertically through the soil to the edge of the deck. The water is collected there and routed either to drop structures provided for park surface drainage or to special drops which penetrate the vent duct wall on the north side of the deck.

The underdrain collector system consists of 6-inch to 12-inch diameter perforated PVC pipes, placed along the low sides of the deck structure. These pipes collect water from the drainage board and convey it to the drop structures or vent ducts. In addition to the collectors, a series of 4-inch diameter perforated PVC pipes intercepts underdrain flow on the deck surface. These interceptors tie into the collectors. They are designed to route flow away from particularly low-gradient areas of the deck surface. Figure 4-2, Sheet 7 shows the collector and interceptor system.

The underdrain collectors discharge into drop structures at the following locations:

- Drop structure 413, located at I-10 Median Station 7650+43.6, 128′ Rt.

- Drop structure 430, located at I-10 Median Station 7659+64, 147′ Lt. This drop structure is directly east of the north fountain equipment room.

- Drop structure 444, located at I-10 Median Station 7665+99.25, 134′ Lt. This drop structure is immediately east of Ventilation Room #2.

- Drop structure 1117, located at I-10 Median Station 7669+55.76, 132′ Rt.

The underdrain collectors discharge into the vent duct on the north side of the deck in two locations. One is at Park Centerline Station 29+49; the other is Park Centerline Station 36+99. Special cleanouts tap into the vent duct wall 2 feet below the bottom of the pile cap at these points. Discharges into the vent duct are expected to be minor, and should evaporate rapidly due to the large volume of air moving through the duct. Significant volumes of water may build up after an extended period of complete soil saturation. This water can be pumped from the low points in the vent duct floor to drains in Ventilation Room #1, which discharge into the freeway system.

c. **Freeway System**

Direct rainfall between 3rd Avenue and 3rd Street does not reach the freeway level because of the covered deck. The freeway drainage system between 3rd Avenue and 3rd Street will collect and convey water resulting from wash-down operations, fire emergencies, park surface drainage and underdrains. The volumes and peak flows conveyed to the freeway system from park surface drains are much greater than those resulting from fire emergencies or maintenance. Therefore, the freeway drainage system was designed to handle flows collected from the deck area and conveyed to the freeway level.

Catch basins are placed in the median areas and along the shoulder of the freeway to intercept wash-down and fire emergency water. The catch basins limit gutter flow lengths to a maximimum of 500 feet. Lateral pipes connect catch basins to the main storm sewer line.

Drop structures connect the park drainage system to the freeway system. Storm water is transported from the drop structures to the main freeway pipes by lateral lines. The drop structures, laterals, and the main freeway system should handle flows of a magnitude corresponding to the 100-year event without backflow onto the freeway.

Catch basins at the junctions of laterals from drop structures and the main freeway pipes will act as water release basins for storm flows greater than the 100-year event. This arrangement relieves back pressure from the surcharged pipes while allowing flow from the park level to the freeway level to continue through the drop structures. Freeway flooding will occur in this case, but the deck structure will still be protected from excess ponding depths.

The West Tunnel crosses under the freeway with a clearance of less than 2 feet between the tunnel's protective cradle and the freeway pavement. This presents an impenetrable barrier to the freeway drainage system pipes or any other buried facilities. Flows to the east of the tunnel are carried to the 3rd Street pump station, while flows to the west of the tunnel are carried to the 3rd Avenue pump station.

d. **Future Drainage**

The design of the integrated covered deck drainage system also accommodates future drainage needs as outlined in the current Master Plan for the covered deck. Stub-outs have been provided at appropriate drop structures on the park level. The current drainage system was designed to be incorporated into the final drainage system for the completed

park. In addition, stub-outs have been provided in the freeway drainage level for future transit platform needs.

B. Off-Site Flow Collection

1. General Overview

The off-site storm drain system is designed to intercept storm runoff from the drainage area bounded on the north by the Arizona Canal and the I-10 Freeway on the south. The east and west boundaries are controlled by street drainage patterns. The Grand Canal acts as a barrier to minor flows coming from the north. During the 50-year project design storm, overtopping of the canal would be expected to occur at major street intersections.

Surface runoff occurs in areas not served by storm drains and where the capacity of the storm drains is exceeded. This runoff generally follows the major streets in the area, since these offer the least resistance to drainage flows. The major street drainage pattern is to the west and south, which causes the surface runoff to split at each major intersection. The amount of flow to the south and west is dependent on the downstream slopes and geometric configuration of each street.

2. 15th Avenue to Central Avenue

Storm drains exist in every avenue from 15th Avenue to Central Avenue. The size of these storm drains range from 8 inch to 96 inch. The major storm drains are a 96 inch in 15th Avenue, a 75 inch in 9th Avenue and a 27 inch in 7th Avenue. The minor storm sewer lines consist of 6 inch, 8 inch, 10 inch and 12 inch lines, usually, but not always, located in the boulevards. Some of

these lines are abandoned irrigation lines and drain to sumps, other irrigation lines or connect to storm drains constructed in later years. With the exception of the 96 inch pipe in 15th Avenue, all storm drains are severed by the construction of I-10.

The storm drain system intercepting flows from the north includes a box culvert interceptor generally located in Moreland from 13th Avenue to 7th Avenue and in 7th Avenue from Moreland Street to Culver Street, and a deep tunnel located under Culver Street from 7th Avenue to 10th Street. The system includes a series of inlets to accept overland flow at appropriate locations, generally within city street right-of-way. Drop structures located at 7th Avenue and 10th Street provide connections to the tunnel for incoming storm drains and the surface interceptor. Also, drop shafts at 3rd Avenue and Culver and Central and Culver provide connections to the tunnel for incoming storm drains, the pump station outlet and the 96-inch storm drain in Central Avenue.

Special Design Considerations

a. **Use of Existing Storm Drains** - Existing storm drains were utilized when connections to the storm drain tunnel were either not needed or not practical. All storm drains north of the I-10 corridor drain to the tunnels. All new inlets except for those located along 7th Avenue at Willetta, Lynwood and McDowell drain into new storm drains.

   New storm drains south of the I-10 corridor connect to existing storm drains. The intent was to utilize the existing system, and to supplement it with additional pipe and inlet structures. The most significant use of an existing storm drain is the connection to the existing

75-inch storm drain in 9th Avenue with inlet structures along 7th Avenue in the vicinity of Ramp 7A-C and inlet structures along Ramp 7A-C.

It is anticipated that connection to the existing storm drain system will have minimal impact on the operation of the existing system for the following reasons:

- Storm drains which are severed by the freeway have no inflow from areas north of I-10. Additional capacity is therefore available in the existing system.

- Storm drain inlets south of I-10 basically replace existing inlets.

b. **Emergency Overflow Routes** - In the event that the design storm is exceeded or the storm sewer system is partially or completely plugged, emergency overflow routes will be needed to protect the existing developed areas north of I-10. The emergency overflow routes are as follows:

- Area between 15th Avenue and 11th Avenue: Excess storm water will flow overland to the surge pond located between 13th Avenue and 11th Avenue. If pond capacity is exceded, storm water will flow overland along a designated "cobbled" route to the freeway.

- Area between 11th Avenue and 7th Avenue: Excess torm water will flow through holes in the retaining/noise barrier wall. Hole size and elevation will be set to maintain a minimum of 0.5 feet freeboard between high water and the top slab elevation of the lowest habitable space.

- Area along 7th Avenue: Excess storm water will flow along the 7th Avenue corridor and down Ramps 7A-A, B, C, and D.

- Area between 7th Avenue and 5th Avenue: Excess storm water will flow overland through holes in the retaining/noise barrier walls, as well as around the wall and onto Ramp 7A-A.

- Area between 5th Avenue and 3rd Avenue: Excess storm water will flow along the proposed pedestrian pathway to the low points then onto the freeway through holes in the retaining wall.

- Area between 3rd Avenue and Central Avenue: Excess storm water will flow overland to the covered deck section then onto the freeway.

- Surge pond at west end of Storm Water Interceptor - In order to provide expansion room for surges in the storm drain tunnel, a low area is provided for a "surge pond" at the west end of the Storm Water Interceptor. The pond and adjoining swales also intercept drainage between 15th Avenue and 11th Avenue and direct the runoff to the Storm Water Interceptor.

3. Central Avenue to 20th Street

Street flows are generally collected by means of curb opening inlets. Single and multiple grate inlets are placed in depressed paved areas at street closures and bus bays, and in depressed off-street areas such as the alley between Fifth and Seventh Streets. These depressions are used primarily to collect overland flows. In Second Street, where a valley gutter

separates a parking area from through traffic lanes, grated inlets were also installed in the valley gutter.

At street closure locations (Fifth, Tenth, Eleventh, and Thirteenth Streets), 9-foot openings are provided in the freeway retaining walls to permit flows in excess of the 50-year design flows to pass through. The openings are designed to pass one-half of the 50-year flows that could concentrate at these locations with maximum backwater limited to 0.5 feet below the nearest adjacent finished floor elevation.

Sumps, either at curb returns or at street closures, are designed with a maximum depth of one foot. The drainage swale collecting overland flow between Seventh and Ninth Streets is designed to convey the 50-year runoff, exclusive of what is intercepted by the grate inlets within the swale. Flows in excess of the 50-year runoff will discharge out of the swale and pass through the freeway retaining wall openings.

A collector pipe under Culver Street conveys runoff intercepted from Central Avenue to Third Street to a drop shaft located at Third Street. A second interceptor along the north right-of-way collects runoff from an area between Seventh and Ninth Streets to just east of Fifth Street for discharge into a drop shaft located at Seventh Street. The storm drain collectors for Tenth through Nineteenth Streets connect directly into the SWI at their respective Culver Street locations.

In addition to improving the existing storm water collection system adjacent to and north of the corridor, the system provides for future expansion of the storm drain collectors northward on Third Street, where a manhole is provided for future tie-in; and at Twelfth Street, where a stub is provided for a future storm drain. Future expansion would be undertaken by the City of Phoenix.

The net effect on existing drainage patterns is increased effectiveness of existing storm drain systems south of the project, due to the diversion of flows from the area north of the project corridor. The storm collection system immediately north of the corridor was upgraded, thereby reducing the occurrence and extent of localized street flooding adjacent to the project site. This project does not relieve street flooding further north of the I-10 corridor, due to the number of storm drains of undercapacity of existing systems. The western margin of the project from Central Avenue to Fourth Street intercepts flows tributary to Cave Creek, possibly reducing some of the flooding potential along Cave Creek.

4. **20th Street to Roosevelt Street**

Flows reaching this portion of the project are expected to overtop the Culver Street curb between 21st Street and 24th Street, and are collected by an area drain prior to reaching the depressed ramp McDowell-C.

5. **Roosevelt Street to Harrison Street**

Off-site runnoff at Washington Street and Jefferson Street was assumed to split at the centerline of the streets. A six inch curb height in Washington Street allows conveyance of about 13 cfs (1/2 street). A swale, parallel to Washington and behind the sidewalk, intercepts water flowing over the top of the curb. A weir flow into the swale from the street was generated and the required length necessary to intercept flows in excess of the street gutter capacity was calculated. This indicated the swale will carry about 187 cfs. The flow remaining in the gutter is then intercepted with a standard ADOT Curb Opening Catch Basin. The runoff in both the swale and the street catch basin flows to a special grated catch basin connected to the primary North-South storm sewer which carries flows to the drop shaft at Van Buren Street.

During construction of the East Tunnel facilities, the new Washington Street and Jefferson collection systems were connected to the existing City of Phoenix storm sewer system. This connection was removed upon completion of the East Tunnel.

Off-site flows in Van Buren Street are intercepted in the same manner as Washington Street. The west curb on 21st Street at Fillmore, Taylor and Polk is depressed so that off-site surface discharges flowing westerly are intercepted by a swale along the east I-10 right-of-way line. These flows are collected by special grated catch basins positioned at the low point in the swale. The drainage flows in Monroe Street and Adams Street are intercepted by both the new frontage road and the swale along the east I-10 right-of-way line. Madison Street from 22nd Street to I-10 is graded to direct water westerly to a proposed special grated catch basin located in a drainage swale along the I-10 right-of-way or access control line. There is a cul-de-sac where Jackson ends at the I-10 right-of-way. Runoff flows through a depressed curb section in the cul-de-sac into a swale along the east I-10 access control line. New catch basins are connected by pipes to an existing 24-inch storm sewer in 20th Street.

6. **Harrison Street to Buckeye Road**

The runoff at Harrison Street is predominantly sheet flow, as is the case at Grant Street, and the area between Sherman Street and Ramp E-N. Drainage ditches are placed within the I-10 right-of-way to intercept this sheet flow. The flow is then conveyed to ADOT standard drop inlets (Standard C-14.30). The captured off-site drainage enters a pipe system running parallel to I-10. This pipe is located at or beyond the fill slope to minimize the depth to the pipe and manhole depth.

7. Buckeye Road to the Maricopa Freeway

The off-site drainage near Buckeye Road is piped north across Buckeye Road to the drop shaft at Buckeye Road and the I-10 right-of-way. The existing 42-inch pipe on the east side of the freeway has been replaced by a new pipe from Yuma Street north, crossing Buckeye road. South of Yuma Street, the existing 42 inch pipe collects the off-site and drainage flow from the new inlets. At Pima Street, the capacity of the existing pipe is exceeded and a new pipe runs from Pima Street to the drop shaft at Mohave Street.

Off-site flows reaching this segment are intercepted in three ways. Flows on major streets (Buckeye Road and Mohave Street) are intercepted by means of special single curb opening inlets designed in accordance with HEC-12. Off-site storm water flowing west along the south side of Buckeye road east of Ramp B-A because of an existing building and inadequate right-of-way. At some future time, it is anticipated that Buckeye road east of the freeway will be widened to match the width at the underpass. At that time, a major inlet can be installed if necessary.

The current design intercepts minor flows with an inlet at the east curb return. Excess flow follows the curb return around to the south to the sag in Ramp B-A. The design flows are intercepted in a sump condition by a curb opening inlet operating as a weir. When Buckeye Road is widened, the small inlet can be inexpensively relocated. A new 50-foot radius return can be installed without disturbing the major inlet structure in the sag. The low point in the sag is located about 103.5 feet south of and .79 foot below (not including the inlet depression) the south gutter line of Buckeye Road. Spread limitations were waived for this location.

A headwall inlet is located at the southern extension of 21st Place and the northeast side of the Maricopa Interchange. Off-site areas south of Mohave and west of 24th Street drain towards this point.

Natural drainage which is interrupted by the north leg of this segment is collected by open ditches on the east side of the freeway. Off-site flow at Pima Street enters the end of one of the ditches, while flows in other locations spill over the sides of the ditches. Where dictated by velocities, the ditches are protected from erosion by rip-rap and geotextile lining.

III. DESCRIPTION OF DESIGN PROCEDURES

A. Design Criteria

The following general design criteria for storm water drainage were provided to the design consultants.

1. On-Site Flow Interception

The new storm sewer shall be located such that it will not intersect the sanitary sewer at any point. However, the sanitary and storm sewers may cross within two feet of each other. At these points, the clay sanitary sewer pipe is to be replaced with ductile iron pipe. The conflicts with other utilities such as gas, water, and electric will be resolved by relocating the existing utility line. The minimum cover over the pipes will be 18 inches.

The drainage system for I-10 shall include all inlets, manholes, sewers, ditches, culverts, pump stations or other hydraulic and erosion control appurtenances required to:

- Not adversely affect the historic runoff pattern.

- Properly dispose of storm runoff disrupted or generated by the freeway and its associated construction.

- Protect the roadway and slopes from damage by erosion.

- Maintain clear traffic lanes for the design storm.

Design Specifications

The following publications are to be used in conjunction with these criteria as guides in developing the plans for this project.

- "A Policy on Geometric Design of Rural Highways," AASHTO, 1965.

- "A Policy on Design of Urban Highways and Arterial Streets," AASHTO, 1973.

- "Division of Highways - Standard Drawings," ADOT, 1974 and current revisions.

- "Standard Specifications for Road and Bridge Construction," ADOT, 1969, and ADOT's current supplemental specifications.

- "Drafting Standards for Use in Office and Field," ADOT, 1982.

- "Reinforced Concrete Box Culvert Manual," ADOT, March, 1981.

- ADOT Drainage Design Services, "Hydrologic Design for Highway Drainage in Arizona," subsequent revisions thereto.

- ADOT Drainage Design Services - Hydrologic Design Notes distributed in 1969 and 1972.

- "Design of Urban Highway Drainage, the State of the Art," U.S. Department of Transportation, August 1979.

Pavement Drainage

- Method used to estimate design flow--Rational Method.

- Design Frequency--10 years.

- Time of Concentration--10 minutes minimum or sum of overland and gutter flow time if more than 10 minutes.

- Intensity--Phoenix Rainfall Intensity - Duration - Frequency Curves.

- Runoff Coefficients.

  - Paved Surfaces ---------- 0.9
  - Highway Slopes and Ditches ---------- 0.5
  - Urban Residential Districts ---------- 0.4-0.6
  - Parks and Recreation ---------- 0.3-0.4
  - Cultivated Areas ---------- 0.2-0.4

- Method used for inlet analysis -- Johns Hopkins University Method or "Design of Urban Highway Drainage, The State of the Art" by U.S. DOT, FHWA, 1979.

- Mannings "n"

  - Concrete pavement and shoulder ---------- 0.016
  - Asphaltic concrete pavement and shoulders --------- 0.016

- Allowable Spread -- The maximum allowable water spread on pavement shall not exceed a depth of 0.50 feet or the criteria given in Table 4-1.

Storm Sewers

- Method used to estimate design flow -- Rational Method.

- Design Frequency.

  - 10-year flowing just full.

  - 50-year storm hydraulic gradient shall not exceed an elevation 6 inches below the low steel of any catch basin grate.

- Time of Concentration.

  - Time to first pavement inlet -- 10 minutes minimum.

  - Time of concentration for overland flow -- Sum of overland and gutter flow time if more than 10 minutes.

- Intensity -- Phoenix Rainfall Intensity - Duration - Frequency Curves.

- Runoff Coefficient.

  - Paved Surfaces ---------- 0.9
  - Highway Slopes and Ditches ---------- 0.5
  - Urban Residential Districts ---------- 0.4-0.6
  - Parks and Recreation ---------- 0.3-0.4
  - Cultivated Areas ---------- 0.2-0.4

TABLE 4-1

Maximum Allowable Water Spread

Roadway Type	Maximum Water Surface Width	*Storm Frequency
Rural 4-lane	Lt. - Shoulder width Rt. - Shoulder width plus 1/2 adjacent traffic lane width	10 Years
Urban 4 or 6-lane	Lt. - Gutter width plus any shoulder width Rt. - Shoulder, parking or distress lane width	10 Years
4 or 6-lane undivided	Shoulder, parking or distress lane width plus 1/2 adjacent traffic lane width	10 Years
2-lane undivided	Gutter and/or shoulder, parking or distress lane width	10 Years
22′ or 24′ Ramps (Including accel. and decel. lane)	Lt. - 2′ Rt. - 8′	10 Years

* 50 years for underpasses or other depressed roadways where ponded water can be removed only through a storm drain system.

- Method for Hydraulic Analysis -- Manning's Equation and Continuity Equation.

- Manning's "n"

  - Concrete Pipe ---------- 0.012
  - Box Culvert ---------- 0.015
  - Corrugated Metal Pipe ---------- 0.024
  - Cast Iron Pipe ---------- 0.013
  - Steel Pipe ---------- 0.011

- Minimum Velocity -- 3 feet per second desirable for design flow.

- Minimum Pipe Size -- 18" Laterals, 24" Main Line.

- Minimum and Maximum Cover for Concrete Pipe - ADOT Drawing No. C-13.03.

- Maximum length of pipe between manholes or access points.

  - Under 36" ---------- 400 ft.
  - 36" - 60" ---------- 500 ft.
  - Over 60" ---------- 1,000 ft.

Ditches

- Method used to estimate design flow -- Rational Method.

- Design Frequency -- 10 year.

- Time of Concentration.

  - Overland flow time.
  - Minimum time to ditch inlet -- 10 minutes.

- Velocities -- Ditch velocities greater than 5 feet per second will require ditch protection by one of the following methods:

  - Aggregate lining
  - Riprap
  - Reinforced concrete paving.

- Intensity -- Phoenix Rainfall Intensity - Duration - Frequency Curves.

- Runoff Coefficients

  - Paved Surfaces ---------- 0.9
  - Highway Slopes and Ditches ---------- 0.4-0.5
  - Urban Residential Districts ---------- 0.4-0.6
  - Parks and Recreation ---------- 0.3-0.4
  - Cultivated Areas ---------- 0.2-0.4

- Capacities

  - Concrete Lined.
    (1) Capacity ---------- Manning's Equation
    (2) Manning's "n" factor ---------- 0.015.

  - Aggregate Lined.
    (1) Capacity ---------- Manning's Equation
    (2) Manning's "n" factor:
        0.030 -------- Aggregate D50(2"-4")
        0.035 -------- Aggregate D50(4"-6")

  - Erodible Channels
    (1) Capacity ---------- Manning's Equation
    (2) Manning's "n" factor ---------- 0.025

- Minimum Desirable Slope -- 0.2%

Culverts

- o Method used to estimate design flow.

  Drainage area less than one square mile - Rational Method.
  Drainage area greater than one square mile - SCS TR-20 method.

- o Design Frequency -- 50 years.

- o Maximum Allowable Headwater for Design Storm.

  The most critical of the following:

  - One foot below PI of shoulder.
  - Prevent flooding around buildings
  - Height of the culvert plus 4 feet

- o Manning's "n"

  - Concrete Pipe ---------- 0.012
  - Corrugated Metal 1/2" Corrugations ---------- 0.024
  - Box Culverts ---------- 0.015

- o Minimum Size

  - Pipe Culvert ---------- 24"
  - Box Culvert ---------- 4' x 4'

- o Minimum and Maximum Cover:

  ADOT Drawing Nos. C-13.03, C-13.06, C-13.08

o Culvert Outlet Protection.

- Below 7 fps ---------- None
- 7 fps to 15 fps ---------- Riprap
- 15 to 20 fps ---------- Rock Lining
- Over 20 fps ---------- Energy Dissipator

2. Off-Site Flow Interception

Off-site flows reaching the project drainage area from north and east of the right-of-way were determined during the hydrologic analysis conducted for the Value Engineering Study by Howard, Needles, Tammen & Bergendoff.

Two of the considerations used in the design of the off-site drainage system are:

(1) In no case will storm water have a detrimental backwater effect on adjacent properties due to project constructions.

(2) No unfenced areas will be allowed to pond water in excess of one foot in depth.

B. Design Procedures

1. On-Site Flow Interception System Design

a. General Overview

The Rational Method was used to determine peak flows from on-site drainage areas. The rational formula is:

$Q_p = CiA$ Where:

Q_p = Peak runoff in cubic feet per second (cfs)

C = A coefficient representing the ratio of runoff to rainfall. Coefficients of 0.9 and 0.5 were used for pavement and non-pavement areas respectively.

i = Rainfall intensity (inches per hour)

A = Drainage area in acres.

Two assumptions underlie the Rational Method: 1) The frequency fo runoff is the same as that of the rainfall producing the runoff, and 2) Peak flow occurs when all parts of the watershed are contributing runoff at its outlet. The time from the start of rainfall to the peak flow is defined as the "Time of Concentration". A minimum time of concentration of 10 minutes was used for all drainage areas defined in this design to determine inlet spacing.

Inlet interception was calculated using ADOT and HEC-12 methods. Water spread and depth were determined using HEC-12 nomographs and a nomograph developed for the Type A special curb and gutter used in this project.

1. Continuous Grade - Roadway grate inlet with special or vertical curb and gutter conditions (ADOT and HEC-12 equations).

a. Determine spread on pavement using HEC-12 Chart 3. Then given cross slope (S_x), determine depth.

$d = S_x (T) + 0.08'$

d = depth at deepest point (ft.)

S_x = cross slope (ft./ft.)

T = Spread (ft.)

$d' = d - w (S_x)$

d' = depth at grate edge (ft.)

w = effective width of grate (ft.)

b. Determine carryover frontal flow:

$$Q_{cf} = 0.56\ (z/n)(S_o)^{1/2}(d')^{8/3}$$

z = $1/S_x$

n = 0.016 coefficient of roughness

S_o = Roadway longitudinal slope (ft./ft.)

c. Determine frontal interception:

$$Q_{if} = Q_t - Q_{cf}$$

Q_{cf} = frontal carryover

Q_t = total flow

d. Determine velocity:

$$V = Q_t/A$$

A = cross sectional area of water over the grate ($ft.^2$)

e. Determine grate length required to intercept Q_{if}:

$$L_b = V/2\ (d + d_b)^{1/2}$$

d_b = depth of grate steel (ft.).

V = Velocity (fps)

f. Determine side flow intercepted:

$$Q_{is} = 1/\left(1+\frac{0.15V^{1.8}}{S_x L^{2.3}}\right) x Q_{cf}$$

L = length of grate (ft.)

g. Determine total intercepted flow:

$$Q_{it} = Q_{if} + Q_{is}$$

Q_{if} = frontal intercepted
Q_{is} = side flow intercepted

h. Determine carryover frontal flow:

$$Q_{ct} = Q_{cf} - Q_{is}$$

Q_{cf} = frontal carryover
O_{is} = side flow intercepted

2. Continuous grade - Median and off-road inlet, type C-15.80.

a. With given values, using chart 16 of the HEC-12 circular, determine d/B:

B = width of grate (ft.)
n = coefficient of roughness - decomposed granite
z = median side slope (ft./ft.)
S = longitudinal slope (ft./ft.)
Q = Total flow (cfs)

b. Determine the depth of water at inlet:

$d = B \times d/b$

d = depth of water (ft.)

c. Determine the ratio of frontal flow to total flow using chart 17 of the HEC-12 circular, E/O:

d. Determine the area of flow:

$A = d(B+(z \times d))$

e. Determine velocity:

$V = Q/A$ V = Velocity (fps)
Q = Total flow (cfs)
A = area ($ft.^2$)

f. Determine the grate inlet frontal flow interception efficiency using chart 7 of the HEC-12 circular, R_f:

g. Determine the grate inlet side flow interception efficiency using Chart 8 of the HEC-12 circular, R_s:

h. Determine the efficiency of the inlet:
$E = R_f(E_o) + R_s\ (1-E_o)$

i Determine the total intercepted flow:
$Q_i = EQ$
E = Inlet efficiency
Q = Total flow

j. Determine the total carryover flow:

$Q_b = Q - Q_i$

Q_i = Intercepted flow

3. Sag condition - roadway or field inlets in a sump (ADOT methods).

a. Weir equation when water depth $\leq$ 0.4':

$Q_i = 3.0\ (Pe)d^{1.5}$

Q_i = Inlet interception capacity

Pe = Effective perimeter of grate (taking into account 2.0 clogging factor) (ft.^{2})

d = depth (ft.)

b. Orifice equation when water depth > 0.4':

$q_i = c_o\ a\ (2gd)^{0.5}$

c_o = orifice coefficient = 0.67

a = effective clear opening area of the grate, (ft.^{2})

g = 32.16 (ft./sec.^{2})

c. determine q at which maximum depth of 0.45' is reached by working backwards using chart 3.

$$\frac{q_i}{s_1^{1/2}} = \frac{q_2}{s_2^{1/2}}$$

q_i = inlet interception at 0.45' (cfs)

s_1 = actual longitudinal slope (ft./ft.)

Q_2 = flow at max depth (cfs)

S_2 = longitudinal slope at max depth (ft/ft.)

d. determine any carryover flow:

$$Q_c = Q_t - Q_i$$

Q_c = carryover flow

Q_t = total flow

Q_i = intercepted flow

b. **15th Avenue to Central Avenue**

The on-site drainage collection system design used the rational method along with the ADOT and HEC-12 procedures previously mentioned, with these additions:

In the spacing of inlets on a limited-access type of roadway where only on-site runoff will be encountered the full permissible flooding width of the gutter and pavement was utilized. The depth of flow along the curb will be greatest under these conditions and the inlets will operate under maximum efficiency.

The first inlet was located by determining the length of roadway necessary to generate the discharge that will occupy the maximum permissible pavement spread. The second and successive inlets were located at a distance which is just long enough to generate the discharge intercepted by the previous inlet. Thus, at the design discharge the pavement will be flooded to its maximum permissible spread along the section under design.

The last inlet must intercept both the flow that bypassed the previous inlet and the discharge generated by the length of the last pavement reach. The last inlet was therefore designed for 100% interception.

Hydraulic Grade Line

Whenever one of the following conditions occured,

a. the full flow capacity (Q_{full}) of the selected pipe was less than the design discharge Q.

b. A pipe of a larger diameter discharged into a pipe of a smaller diameter.

c. It was impossible to line up the crowns of pipe runs at changes in pipe size.

d. The tail water at the outlet of the sewer run submerged the sewer pipe.

The hydraulic grade line was calculated to be sure that the backwater created by such a design is not large enough to cause blowouts at inlets or manholes above the run. The following procedure was used to calculate the hydraulic grade line:

1. Calculate the friction slope S_f and the head loss for each reach as previously outlined.

2. Plot the hydraulic grade line by adding the head loss to the tailwater elevation and continue upstream.

If the hydraulic grade line was sufficiently below the inlets and manholes, the design was satisfactory. If the hydraulic grade line was at or above the inlets and manholes, blowouts would occur and the design was revised.

c. **Central Avenue to 16th Street**

The following design procedures were used for this portion of the project:

1. Separate collection systems were designed for the Third Street and Sixteenth Street pumping stations. The 50-year storm peak discharges from this portion of the project are approximately 168 cfs and 24 cfs respectively.

2. The recommended pipe network alignments have been designed to minimize potential disruptions of freeway traffic due to damage repair or routine maintenance. This has been achieved by limiting the number of pavement crossings, and by locating manholes behind curbs or in the median only. Manholes 1204 and 1205 have been placed within the road shoulder to permit the storm drain to clear the zone of influence of the north abutment footing of the Seventh Street Bridge. The pipes are set back sufficiently from freeway ramps to permit future expansion to a full 30-foot width. Future expansion into the median would require adjustments to the proposed storm drain system.

3. Minimum slotted drain lengths were calculated for the required flow interception. The lengths were further adjusted to minimize the number of pipe cuts during construction and to make efficient use of material.

4. Surface runoff depth and pavement spread criteria have been met. An exception was made on the super-elevated, west-bound HOV ramp in the vicinity of station 7674+27 because of a structural limitation to the installation of slotted drain. Approximately 0.32 cfs bypass the first HOV ramp grate inlet and is picked up by subsequent ramp inlets.

d. **16th Street to Fillmore Street**

The on-site drainage computations and storm water inlet design for this segment of the project used the Rational Method along with ADOT and HEC-12 methods (see general design procedures). The following design assumptions and constraints were used:

1. Manning's "n" - Pavement - 0.016, Concrete Pipe - 0.012, decomposed granite channel linings - 0.03.

2. Minimum Depth of Pipe Cover - 3.0'.

3. Minimum Flow Velocity - 3.0 feet per second.

4. Maximum Flow Velocity - 10.0 feet per second.

5. Minimum Slotted Drain Length - 10 feet.

6. Minimum Pipe Diameter - 18 inches.

e. **Fillmore Street to Harrison Street**

1. The on-site drainage computations and inlet design used the Rational Method along with the HEC-12 equations listed previously with these additions:

a. Time of Concentration - T_c

$$T_c = T_o + T_g + T_p$$

With: $$T_o = \frac{K\ L^{0.6}\ n^{0.6}}{i^{0.4}\ S^{0.3}}$$

T_o = Time of overland flow in seconds

L = Overland flow length, ft

n = Manning's roughness coefficient

i = Rainfall rate, in/hr

S = Average slope of the overland area

K = 56

Tg = Time of gutter flow (sec.), length of gutter reach ÷ V_2

$$V_a = \frac{1.12}{n}\ S^{.5} S_x^{.67} T_a^{.67}$$

V_a = Average velocity, ft/sec

n = Manning's coefficient

S = Longitudinal slope, ft/ft

S_x = Cross-slope, ft/ft

T_a = Spread at average velocity

T_1 = Spread at upstream end of gutter reach (Value taken to be zero)

T_2 = Spread at downstream end of gutter reach (Value taken to be max. allowable spread)

Therefore: $T_1/T_2 = 0$

And: $T_a = .65T_2$ (from HEC No. 12, pg. 17)

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

T_p = Time of Pipe flow (seconds), pipe length ÷ V

V = Velocity, ft/sec

n = Manning's "n"

R = Hydraulic radius in ft

S = Slope, ft/ft

Multiple Reaches:

Min T_c taken to be 10 min. per ADOT - Storm Sewer System Design, October 1972

All Reaches: T_c = 10 min. or calculated value whichever is greater

b. Project Standard Detail D-1 Special Catch Basin on Continuous Grade:

$Q_i = EQ = Q(R_f E_o + R_s (1-E_o))$, HEC No. 12, March 1984

Q_i = Intercepted flow, cfs

E = Efficiency

Q = Total flow, ft^3/sec

R_f = Frontal flow interception efficiency

E_o = Ratio of frontal flow to total flow

R_s = Side flow interception efficiency

$R_f = 1 - 0.09 (V - V_o)$

V = Gutter Flow Velocity

V_o = Splash over velocity

$E_o = Q_w/Q$

Q_w = Flow in width W, cfs

W = Width of Grate, ft

Q = Total flow, cfs

$$R_s = 1/(1 + \frac{0.15V^{1.8}}{S_x L^{2.3}})$$

S_x = Cross-slope, ft/ft

L = Grate Length, ft

c. Slotted Drain Project Standard Detail D-2 on Continuous Grade:

Criteria shall be in accordance with Technical Memorandum No. 3.1, dated October 29, 1982.

$L = 1.257\ Q^{.404}\ S_o^{A}\ Z_E^{.841}/n^{.235}$ for weir flow, 100% interception

$L = .394\ Q^{.649}\ S_o^{.410} Z_E^{.445}/n^{.811}$ for submerged flow, 100% interception

$$A = .2048 + Z_E/394.8 + 3.996\ S_o - S_o Z_E/19.24 - 31.42\ S_o^2 - Z_E^2/46{,}956 + CD/349.3 - CD^2/10{,}944$$

CD = 0 for Special Curb & Gutter, D-1, and Std. Det. C-05.10 Type 'A'

L = Total flow capture length, ft for 100% interception

Q = Flow rate, cfs

S_o = Longitudinal slope, ft/ft

Z_E = Reciprocal of effective cross-slope, ft/ft

n = 0.016

The larger value of L is to be used.

Efficiency $E = 1.0 - 0.9169(1.0 - {}^{LA}/L)^{1.7914}$

LA = Actual slot length, ft

L = Total flow capture length, ft

Clogging Factor = 1.25

d. Slotted Drain Project Standard Detail D-2 in a sump location: Criteria is in accordance with Technical Memorandum 3.1, dated October 29, 1982.

$Q = C_w L (d)^{1.5}$ when operating as a weir, $d \leq 0.15$ ft

Q = Flow rate, cfs

C_w = 3.0

L = Total flow capture length, ft

d = Depth, ft

$Q = C_o L \frac{SW}{12}(2 g d)^{0.5}$, when operating as an orifice, $d > 0.15$ ft

Q = Flow rate, cfs

C_o = 0.9519 (log d) + 1.1688 for 0.15 ft < d < 0.4 ft

C_o = 0.79 for d > 0.4 ft

L = Total flow capture length, ft

SW = Slot width, inches

g = Gravitational acceleration, ft/sec^2

d = depth, ft

e. Manholes

Use Manhole Std C-18.10

Maximum pipe length between manholes:

Under 36" - 400 ft

36" - 60" - 500 ft

Over 60" - 1000 ft

Hydraulic Losses:

Shaft - $H_m = .05\ V^2/2g$ (downstream)

Expansion - $H_E = KV^2/2g$

Junction Loss per 10.06

Losses are additive

f. Storm Sewer Pipe

18-inches minimum diameter for connecting pipes/laterals

24-inches minimum diameter for collectors

Friction loss by Manning's Equation:

$h_f = 29 \quad n^2 \quad L\ V^2 \div (R^{4/3}\ 2g)$

h_f = Pipe friction loss in ft

n = 0.012

L = Pipe length, ft

V = Velocity, ft/sec

R = Hydraulic Radius, ft

g = gravitational acceleration, ft/sec^2

g. Junctions

Use Connections per Project Std. Det. D-4 or use Manhole Std. C-18.10. Junction loss equation from Street & Highway Drainage, Vol II, Pg. VII-25.

$$\Delta Y = \frac{(Q_2\ V_2 - Q_1\ V_1\ \cos \phi_1 - Q_3\ V_3\ \cos \phi_3)^2}{g\ (A_1 + A_2)}$$

ΔY = Change in hydraulic gradient, ft

Q_2 = Flow downstream, ft^3/sec

Q_1 = Flow upstream, ft^3/sec

Q_3 = Flow lateral, ft^3/sec

V_2 = Velocity, downstream, ft/sec

V_1 = Velocity, upstream, ft/sec

V_3 = Velocity, lateral, ft/sec

ϕ_1 = Angle downstream (ext.) & upstream pipe, degrees

$\emptyset_3$ = Angle downstream (ext.) & lateral, degrees

A_2 = Area downstream, ft^2

A_1 = Area upstream, ft^2

g = gravitational acceleration, ft/sec^2

2. The following criteria were used in the design of the pavement drainage for this segment of the project:

a. I-10 Outside Lanes

10-year Design Frequency

The sag vertical curve at Sta 7793+33 shall not be treated as a depressed roadway which requires 50-year frequency design.

Allowable spread = 12 feet or water surface width corresponding to curb height, whichever is less.

Use Special Curb & Gutter, Project Standard Detail P-1, (h=3" & W=2'). Use Special Catch Basin, Project Standard Det. D-1.

Where guard rail is used, use Type A Curb & Gutter Std. C-05.10 (h=7"). Use Modified Downdrain Std. C-04.20 or Catch Basin, Type 3, Std C-15.20.

b. I-10 Inside Lanes

10-year Design Frequency.

Allowable Spread = 10 feet or water surface width corresponding to curb height, whichever is less.

Use Special Curb & Gutter, Project Standard Detail P-1 (h=3" & W=2.50'). Use Special Median Spillway & Apron, Project Standard Detail D-6 & D-7 or Special Catchbasin, Project Standard Detail D-1.

Where guard rail is used at bridge approaches, no catch basins are located in the curb and gutter.

c. W-E Ramps and Access Roads Pavement

10-year Design Frequency.

Allowable Spread = 8 ft on Rt.
Allowable Spread = 2 ft on Lt.
Allowable Spread = 4 ft at W-J Ramp Termini on Lt.

Use Special Curb and Gutter, Project Standard Detail P-1 (h=6" and W=2.50') on Lt. Use Special Catch Basin, Project Standard Detail D-1.

Use Type A Curb and Gutter, Std. C-05.10 (h=7") on Rt. Use modified Downdrain Std C-04.20 or Catch Basin, Type 3, Std C-15.20.

f. Harrison Street to Buckeye Road

The following procedures and criteria were used for this segment of the project. Computations were based on the Rational Method and HEC-12 procedures previously listed.

Two major controls set the final location of each pipe. These are 1) the location of existing utilities, especially minimization of conflicts with the sanitary sewer, and 2) the need to maintain adequate cover above the new pipes. The new

storm sewer was located such that it does not intersect the sanitary sewer at any point. However, the sanitary and storm sewers may cross within two feet of each other. At these points, the clay sanitary sewer pipe was replaced with ductile iron pipe. The conflicts with other utilities such as gas, water, and electric were resolved by relocating the existing utility line. The minimum cover over the pipes is 18 inches.

Three general types of catch basin inlets were used: 1) inlets located at the curb, to intercept pavement flow, 2) inlets located in the medians, and 3) drop inlets located at the new drainage ditches to intercept off-site flow.

The first type of curb opening inlet utilized for this project is the ADOT Standard C-15.20 catch basin. These catch basins are used along the outside curbs of the Sky Harbor ramps when the ramps are located in fill and downdrains could not be safely used. Another type of curb opening inlet used is the ADOT standard downdrain (Standard C-4.20). These are used to direct pavement drainage into the areas between the freeway and the ramps. The downdrain must be protected by a guard rail. In those areas where a guard rail is not used, curb opening catch basins are used instead of downdrains.

The final type of catch basin used for pavement drainage is ADOT Standard C-15.30. This is a grated inlet which is used when the presence of a concrete barrier precludes the use of a curb-opening catch basin.

After the inlets were sized and their locations determined, a system of conduits was designed to convey the storm runoff. The layout of these conduits follows the recommended and approved alternative discussed in the Inital Drainage Report.

Corrugated Metal Pipe (CMP) was dropped as an alternative due to the high soil corrosivity, as indicated by the soil report.

These additional steps were used in the design of the conduit system:

1. In order to determine the quantity of flow each conduit must carry, a trial pipe diameter and slope were chosen such that when the pipe is carrying the computed discharge for a 10-year storm it is flowing just full. The velocity was then calculated for the design discharge, from which the time of flow in the pipe was computed. The procedure was repeated for the next pipe downstream. Since the final pipe sizes were chosen at a later step in the design, these calculations were performed for only one type of pipe material.

2. After the tentative pipe sizes and gradients had been selected, a hydraulic grade line was computed for each conduit. They hydraulic grade line coincides with the level to which the water would rise in a vertical tube connected to the storm sewer flowing under pressure. By determining the elevation of the hydraulic grade line at the inlets and manholes, the possibility of blowouts was checked. In calculating the hydraulic grade line, the energy losses in the storm drain sytem due to pipe friction, the changes in direction of flows, and turbulence caused by manholes were considered.

   The calculation of the hydraulic grade line began at the outlet of the storm sewer and proceeded upstream. The starting elevation was the highest of the following:

   a. Hydraulic grade line in the outfall pipe.
   b. Critical depth of flow at the outlet.

c. Crown elevation of the outlet pipe.

Possible conflicts with existing sanitary sewer lines between Buchanan Street and Lincoln Street also influenced the vertical placement of the storm drain system.

The energy losses through the manholes were computed. For design purposes, head losses that would occur through the catch basin due to the expansion and contraction of flow were calculated. For the flow expansion from the pipe into the catch basin, the following equation was used:

$$H_L = K_e \frac{V_1^2}{2g} \left[1 - \left(\frac{A_1}{A_2} \right) \right]^2$$

Subscripts 1 and 2 denote the upstream (pipe area) and downstream (catch basin area) sections repectively. K_e , the expansion coefficient, varies from 1.0 for a sudden expansion to 0.2 for a well-designed transition. An expansion coefficient of 0.6 was used for the ADOT catch basins with a mitered entrance from the catch basins into the pipe. For the head losses due to contraction, the following equation was used:

$$H_c = K_c \frac{V_2^2}{2g} \left[1 - \left(\frac{A_2}{A_1} \right)^2 \right]$$

The contraction coefficient (K_c) varies from 0.1 to 0.5. A value of 0.3 was assumed for the ADOT catch basins.

Using a conservative estimate that the area of the catch basin is three times that of the area of the pipe, the head

loss due to expansion is 0.27 hv, and the head loss due to contraction is 0.23 hv. Therefore, the total loss through the catch basin is 0.5 hv.

g. **Buckeye Road to the Maricopa Freeway**

On-site drainage was designed based on the Rational Method and HEC-12 equations previously noted.

h. **Covered Deck**

Drainage facilities in the covered deck section were designed based on the Rational Method and equations previously described. Additional design procedures and constraints were considered as a result of the deck structural limitations. They are summarized below.

Because the deck is depressed below surrounding grades, gravity drainage into the off-site system is not possible. Water depth limitations were provided by the structural designer Ammann and Whitney. Drainage system design and grading plans were prepared to keep maximum water surface elevations below the maximum allowable ponding depths for all storms up to and including the 100-year event.

Bubble-up inlets on the freeway level were designed to allow the high-capacity drains on the deck to continue to function when the freeway system is surcharged. Flows from the drop structures will rise through the catch basins onto the freeway. This should not occur until the 100-year storm magnitude is exceeded.

The 3rd Avenue pump station has a design peak inflow of 166 cfs for the 50-year design storm. The deck and park drainage

features collect runoff from an area larger than the freeway without a deck. Sufficient retention is available to delay peak flows to the pump station until they do not affect the station design inflow.

2. Off-Site Flow Interception System Design

a. General Overview

The off-site drainage is divided into two sections; drainage discharge into the Storm Water Interceptor either through new or renovated storm drain systems and drainage discharge into existing City of Phoenix storm drain systems which will remain independent of project facilities. Drainage structures conveying runoff to the Storm Water Interceptor are designed for the 50-year frequency storm. Facilities connected to existing City of Phoenix storm drains are designed for the 2-year storm as per City of Phoenix design standards.

b. 15th Avenue to Central Avenue

On urban type roadways where cross-roads and off-site drainage areas contribute flow, inlets may have to be located utilizing less than the maximum pavement spread.

Some inlets were located at intersections to prevent gutter flow from crossing traffic lanes of the intersecting roads, at major points of off-site drainage inflow or located due to other than hydraulic considerations. Since the maximum permissible pavement spread and gutter flow depth could not be utilized, inlet interception will not be very efficient and a greater number of inlets was required.

Gutter inlets are not efficient for intercepting off-site drainage. Every effort was made to intercept off-site drainage with open channels or special inlet structures before the flow reaches the pavement.

c. **Central Avenue to 16th Street**

The design of the off-site collection system is based on the flows provided. Inlet design uses the same methods outlined in the section on on-site interception system design.

d. **16th Street to Fillmore Street**

The off-site flow collection system design uses the same methods as the previous section.

e. **Roosevelt Street to Harrison Street**

Off-site storm water flow design uses the methods previously mentioned with these additions.

Van Buren, Washington and Jefferson Flow Interception:

The water depth for the 50-year return frequency storm is expected to average about 1-1/2 feet in these major streets. The runoff discharges shall be intercepted by swales which parallel the major streets. Because of uncertainties in the runoff discharge distribution, the swale length was determined by assuming 100% of flow is intercepted along the side of the swale which is adjacent to the street gutter. Weir flow occurs over the top of the curb and into the swale; therefore, the swale length required to intercept 100% of flow above the top of curb is:

$$L_T = K\,Q^{.42}\,S^{.3}\left(\frac{1}{n\,S_x}\right)^{.6}$$

L_T = Length required for 100% Interception, ft.

K = 0.60

Q = Flow rate in cfs

S = Longitudinal curb and gutter slope, ft./ft.

n = 0.016

S_x = Cross-slope of sidewalk, ft./ft.

The water remaining in the gutter up to the top of curb will be intercepted by a Catch Basin, Type 3 - Double wing Std C-15.20, which is positioned downstream from the swale. This design provides 100% catchment of off-site runoff for the 50-year return frequency storm.

Sheet flow occurs throughout the entire project length. Therefore, additional swales parallel to the I-10 roadway are provided. Runoff flows are proportioned to the swale interception length on each side of the catch basin located at the low point in the swale. The swales are sized to carry the intercepted flow to the catch basin by Manning's Equation:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

V = Velocity, ft/sec

n = .035 for grassed swale per letter from Management Consultant dated October 16, 1985.

R = Hydraulic Radius in ft

S = Slope, ft./ft.

Max V = 5 ft/sec

Minimum longitudinal slope = .0020 ft/ft

Maximum water surface elevation is 6" below adjacent local

street gutter.

All ponding shall be within right-of-way.

Swales are sized assuming free outfall.

Special Grated Catch Basins

Grate Capacity

Special Grated Catch Basins will be designed for a sump condition. The ponding depths will be in the transition stage between weir and orifice flow.

f. **Harrison Street to Buckeye Road**

Design of the off-site drainage system follows the same procedures as previously mentioned in the general overview.

g. **Buckeye Road to the Maricopa Freeway**

Design of the off-site drainage system follows the same procedures used in the other sections of the project.

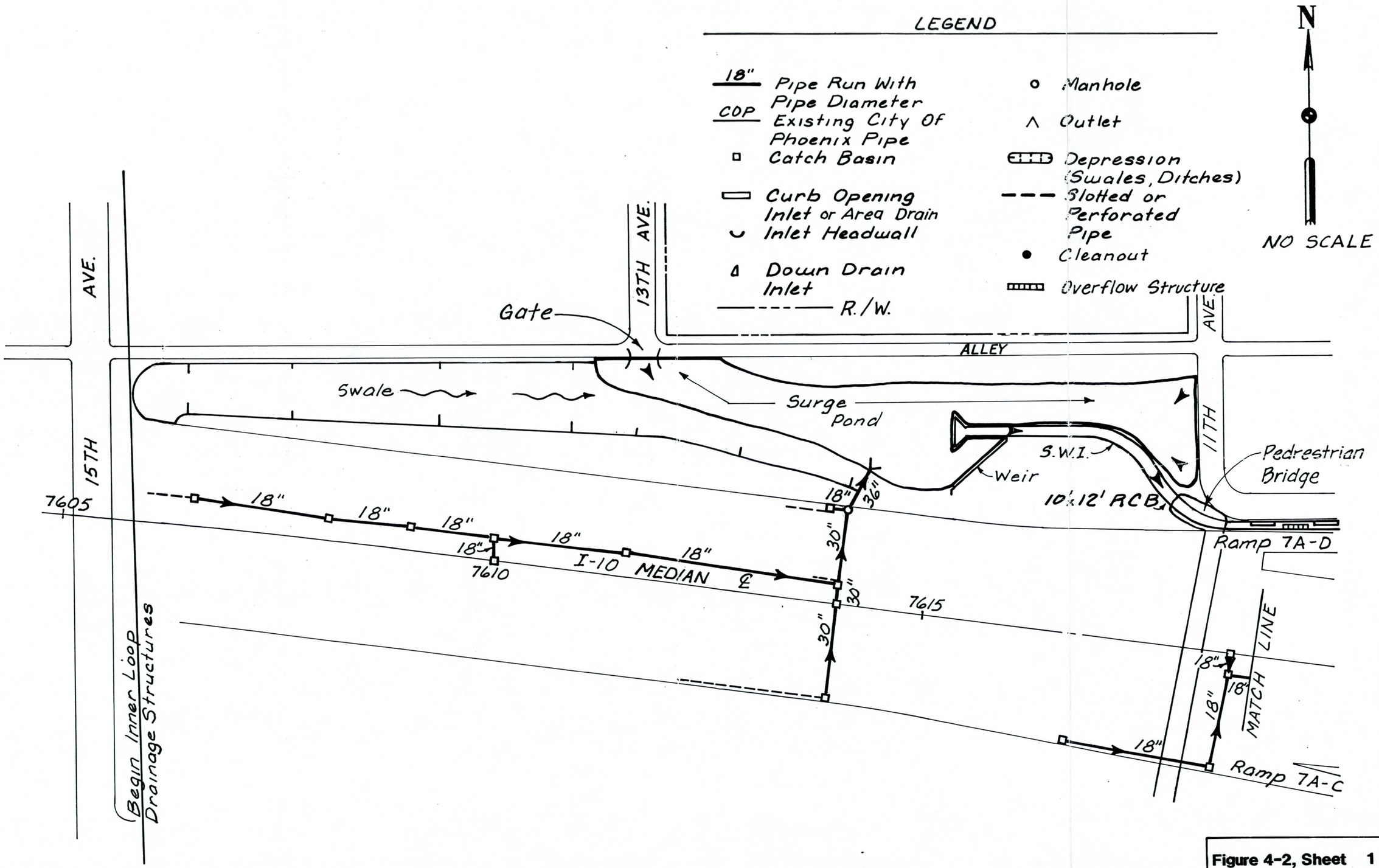


Figure 4-2, Sheet 1
Inner Loop Drainage System
15th Ave - 11th Ave

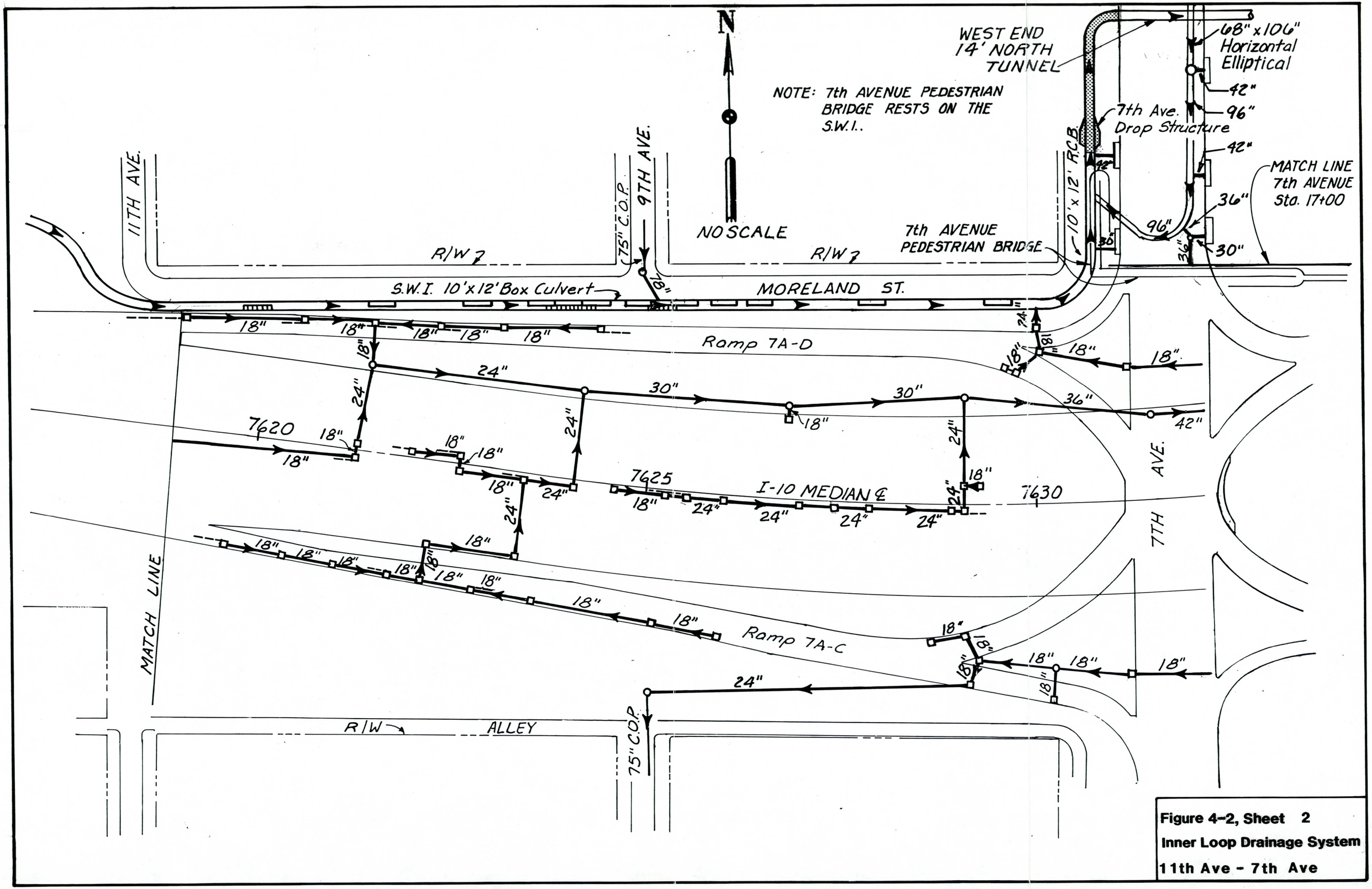


Figure 4-2, Sheet 2
Inner Loop Drainage System
11th Ave - 7th Ave

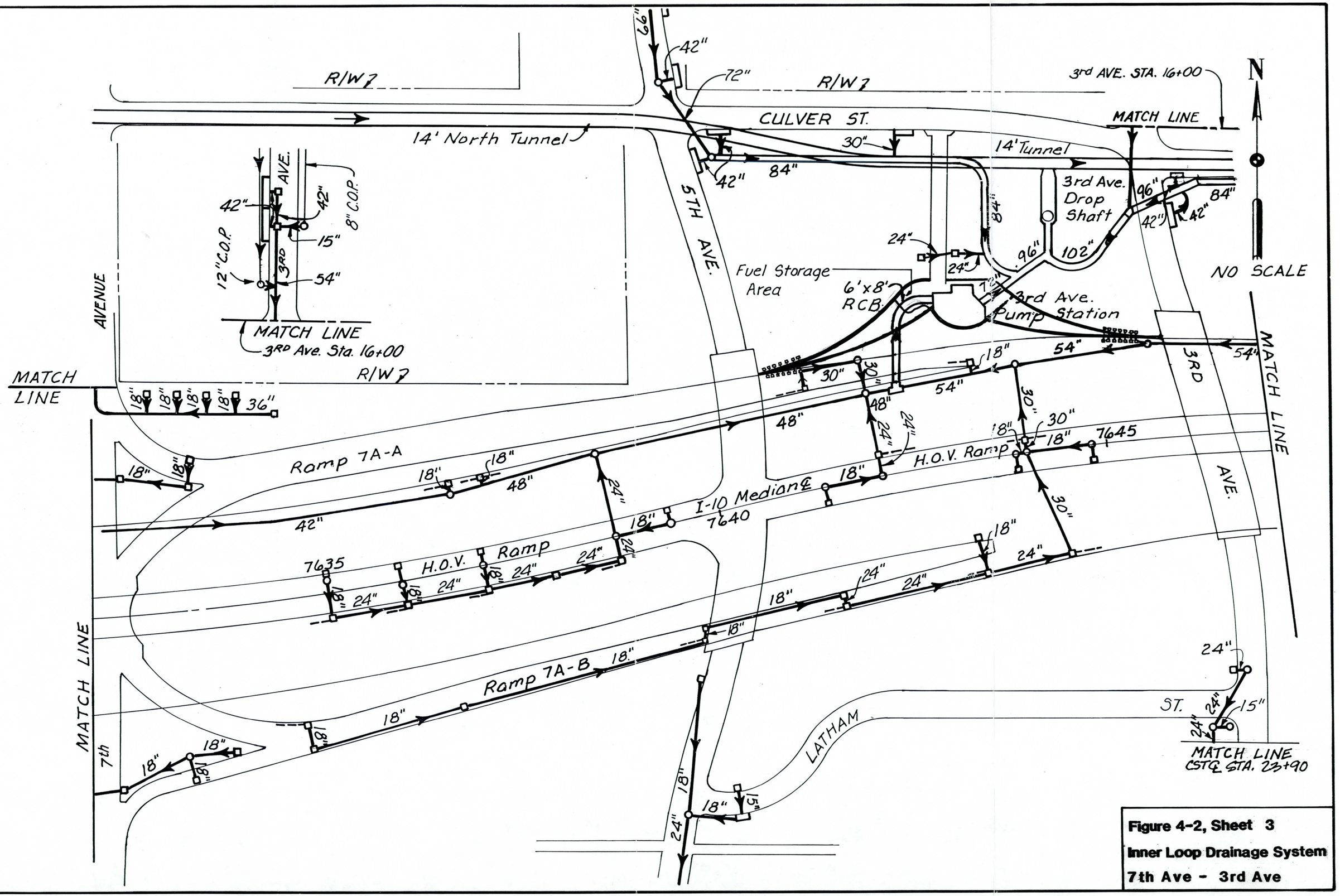


Figure 4-2, Sheet 3

Inner Loop Drainage System

7th Ave - 3rd Ave

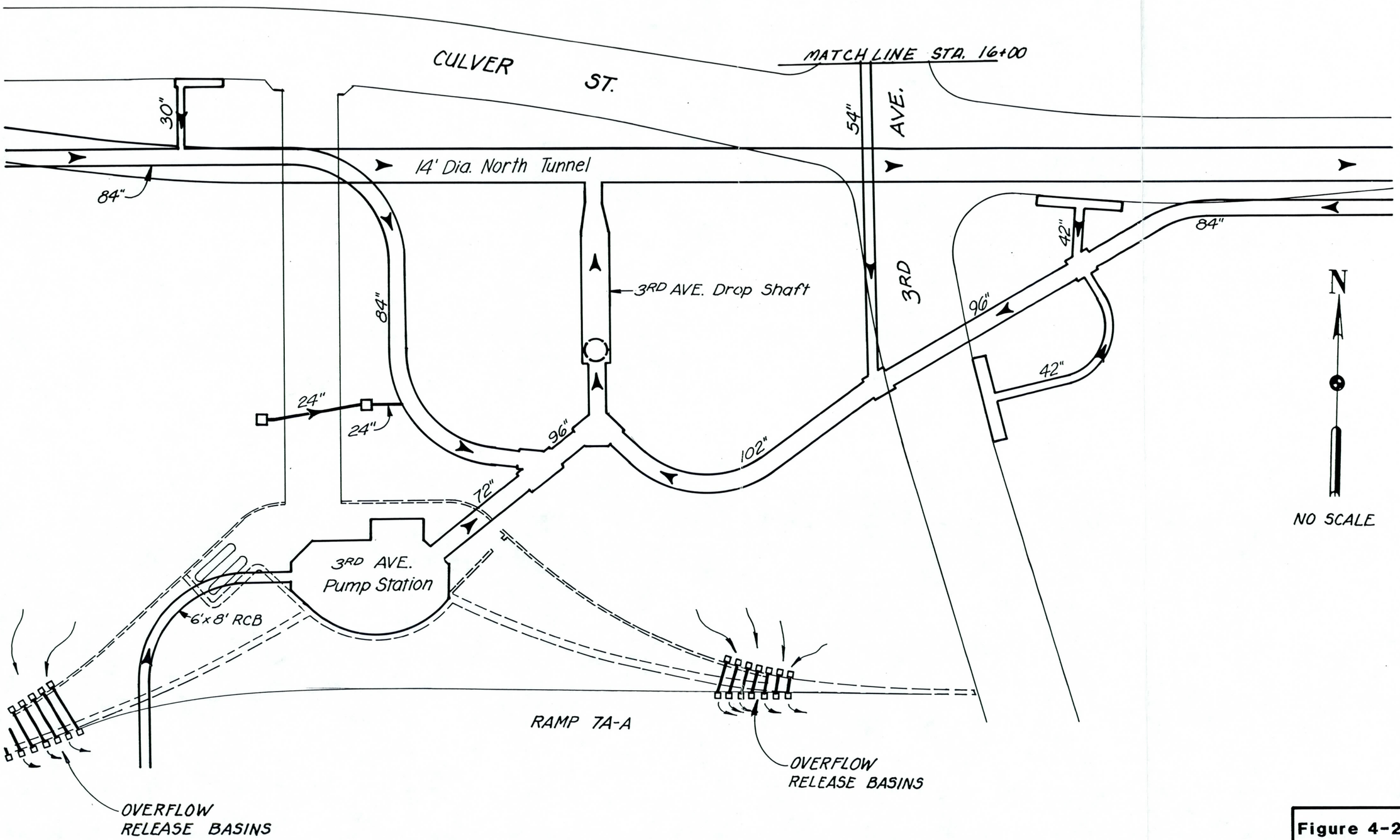


Figure 4-2, Sheet 3 A
Inner Loop Drainage System
7th Ave. - 3rd Ave.

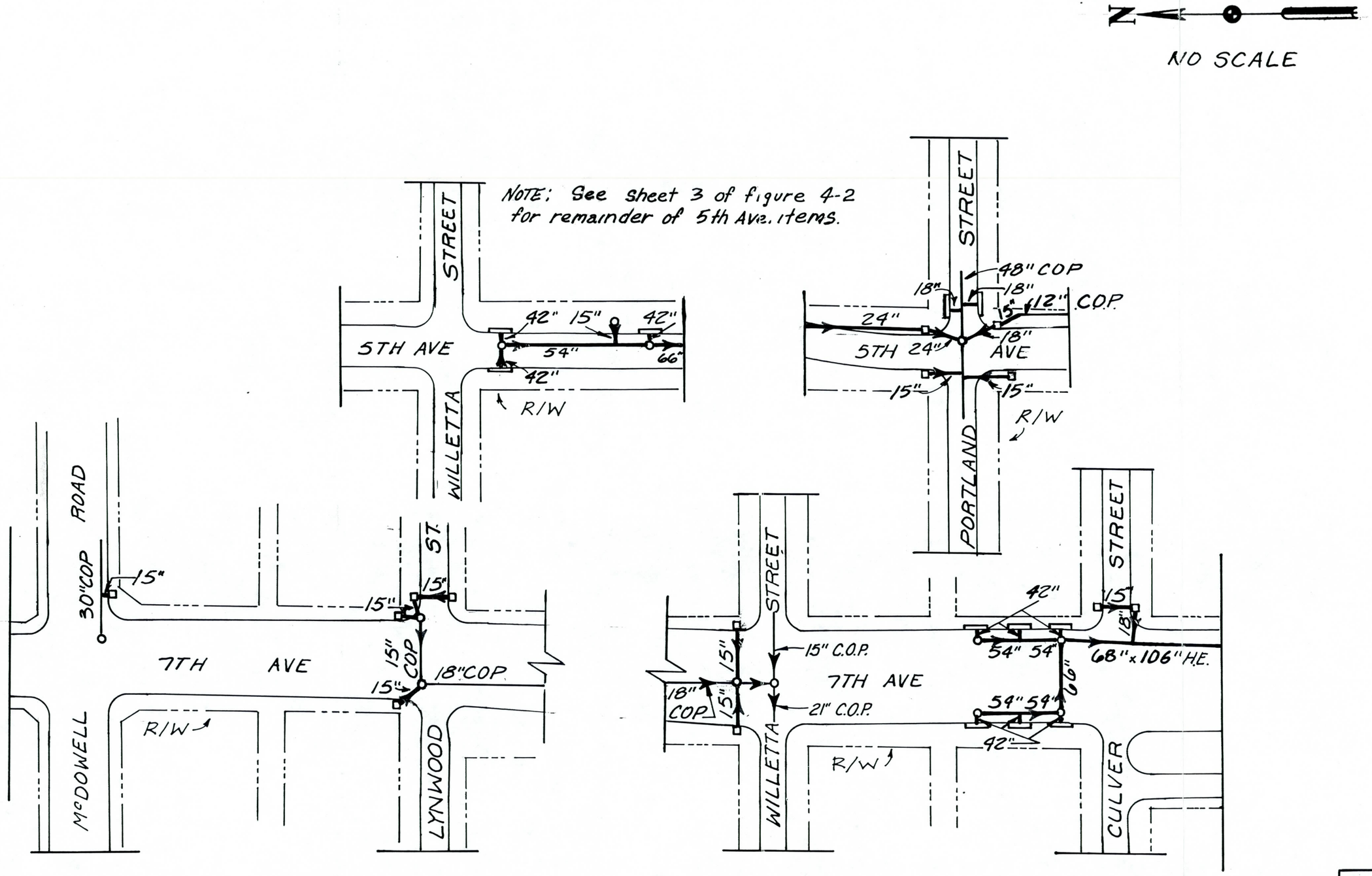


Figure 4-2, Sheet 4
Inner Loop Drainage System
7th Ave and 5th Ave

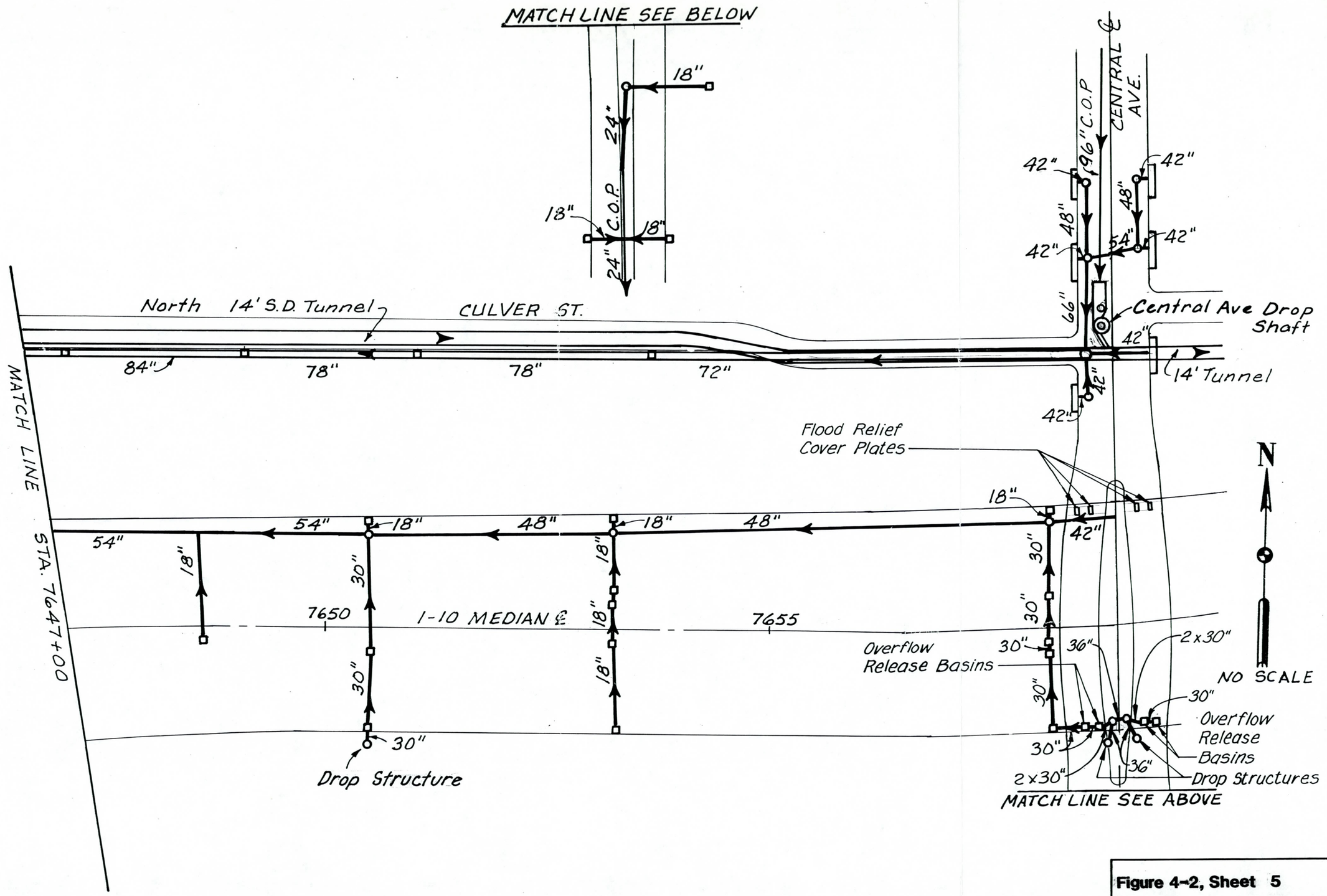


Figure 4-2, Sheet 5
Inner Loop Drainage System
3rd Ave - Central Ave

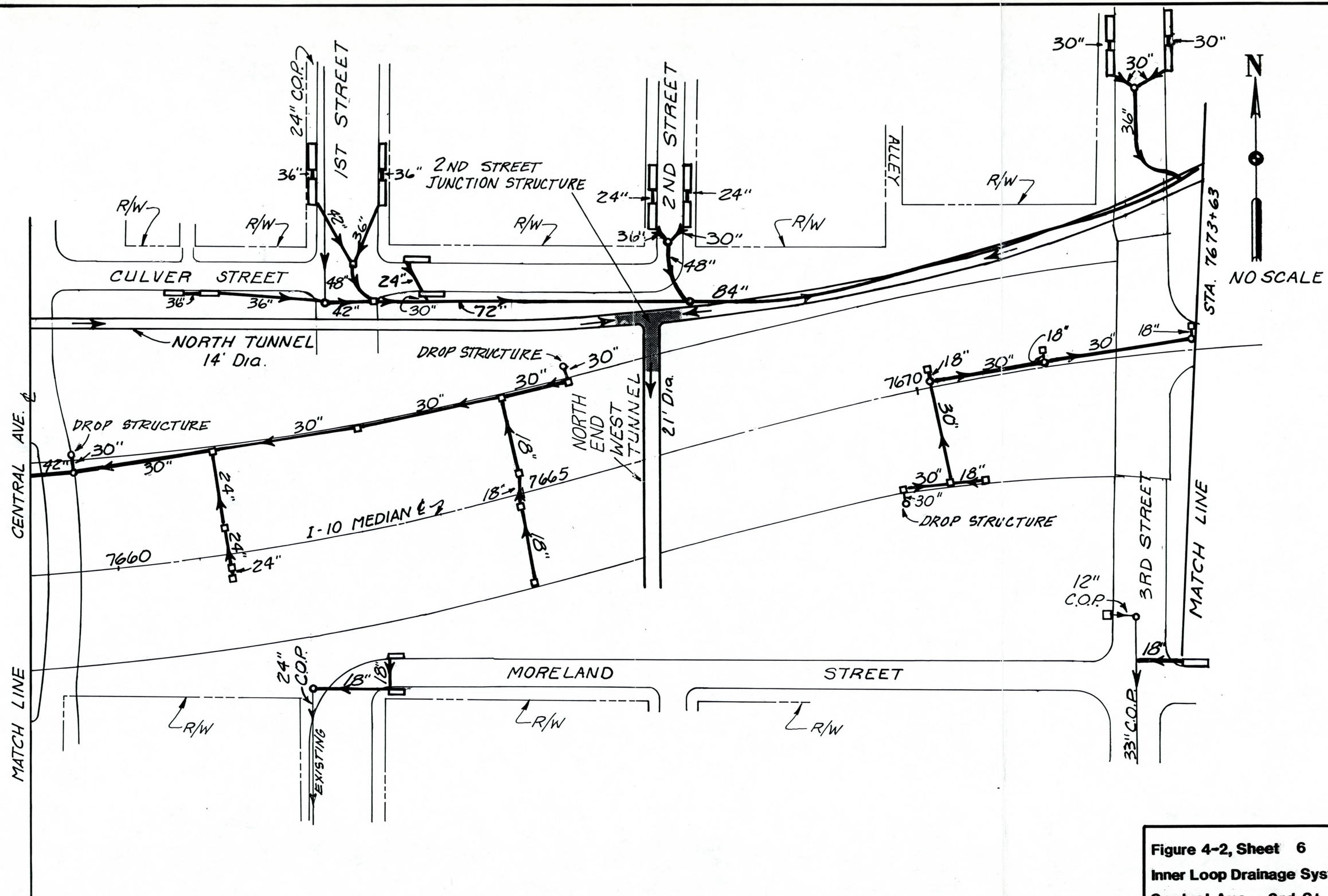


Figure 4-2, Sheet 6
Inner Loop Drainage System
Central Ave - 3rd St

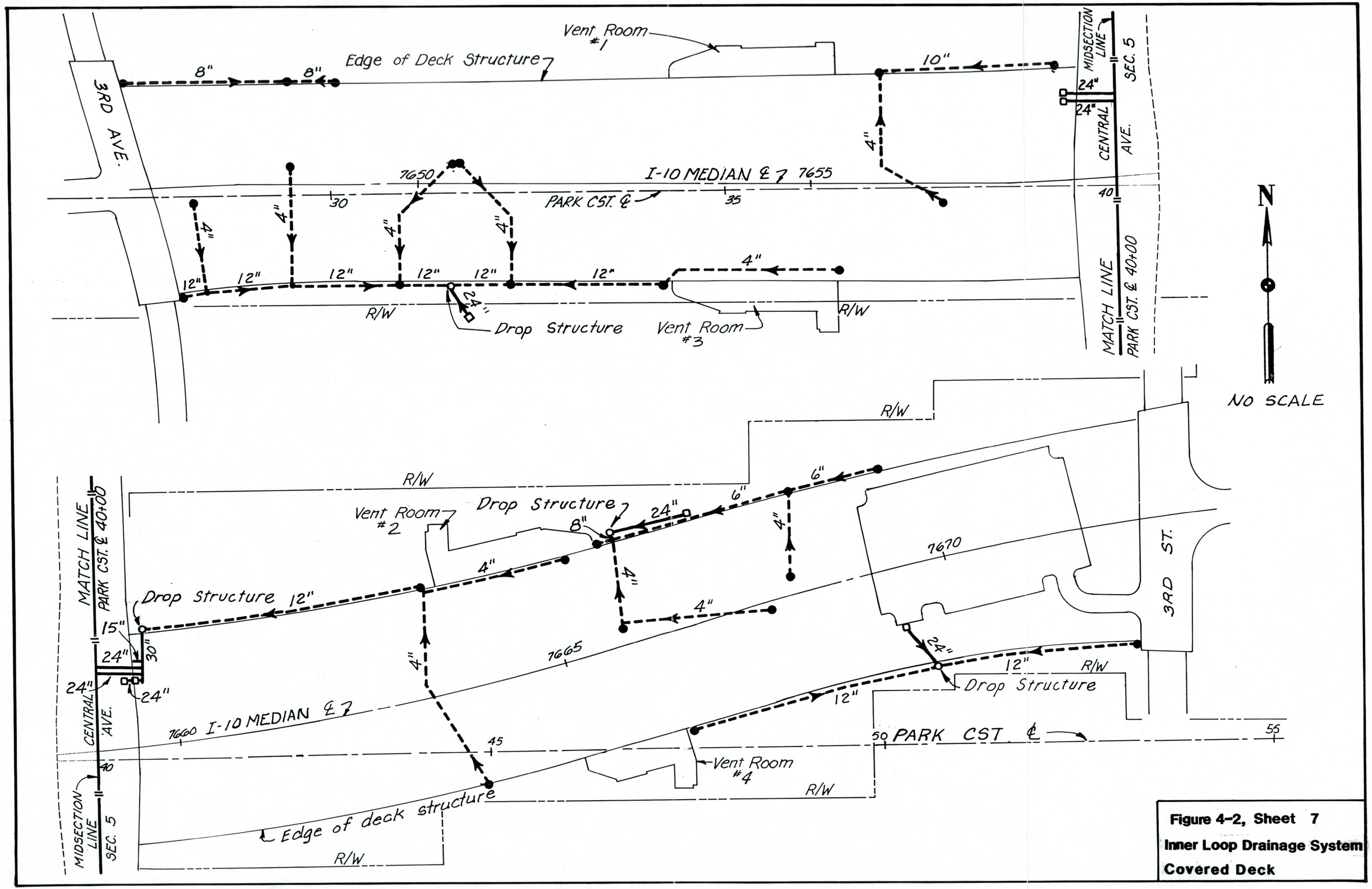
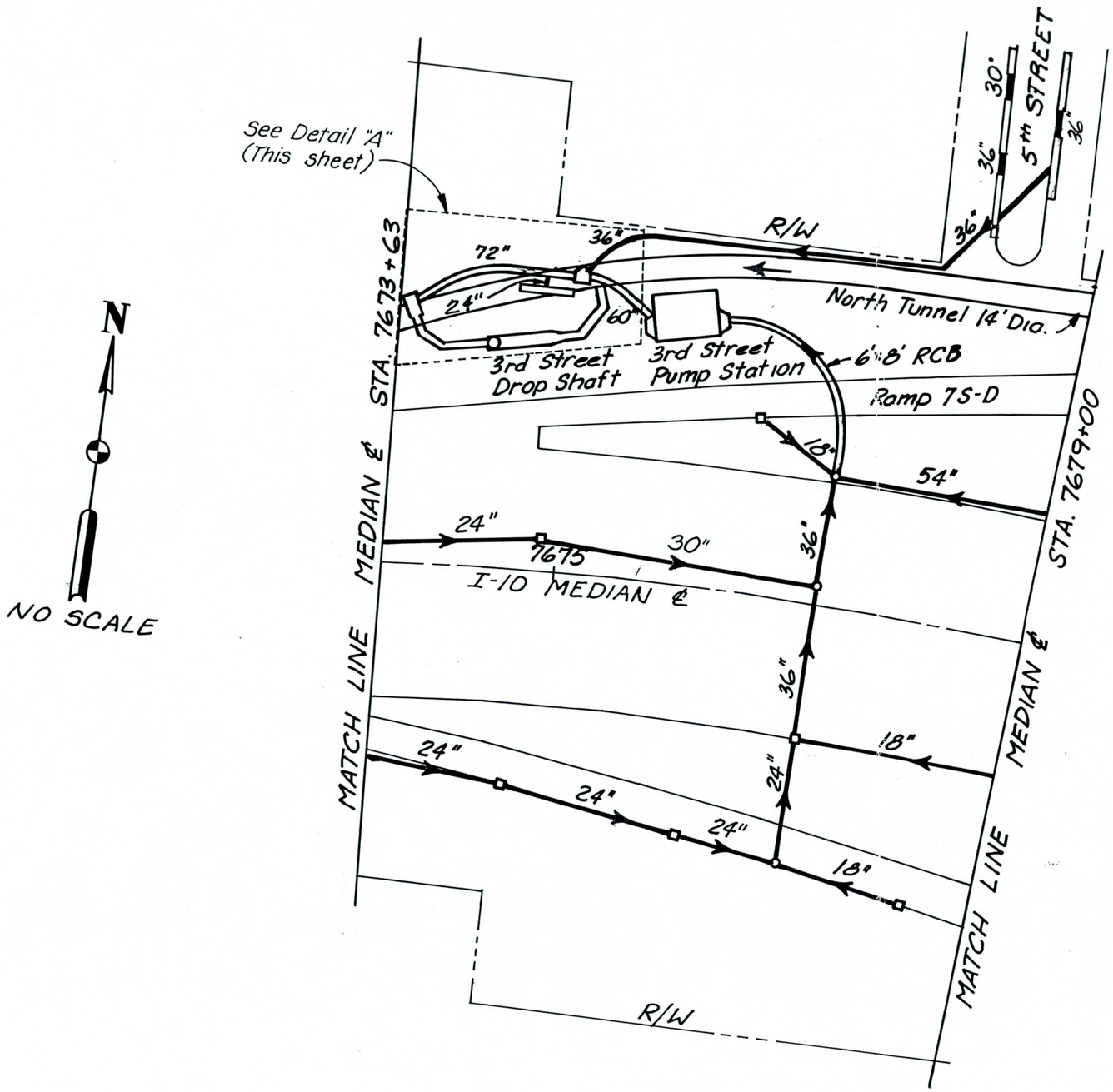


Figure 4-2, Sheet 7
Inner Loop Drainage System
Covered Deck



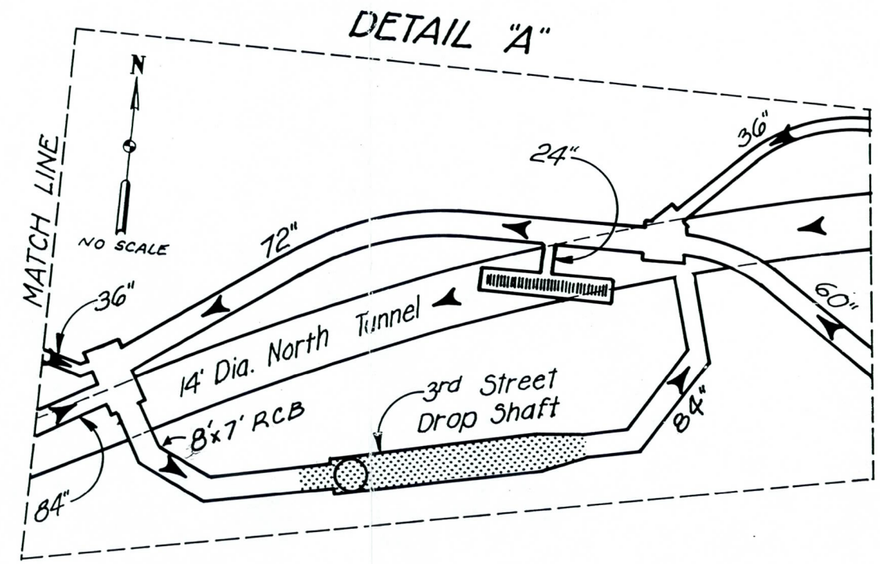


Figure 4-2, Sheet 8
Inner Loop Drainage System
3rd St - 5th St

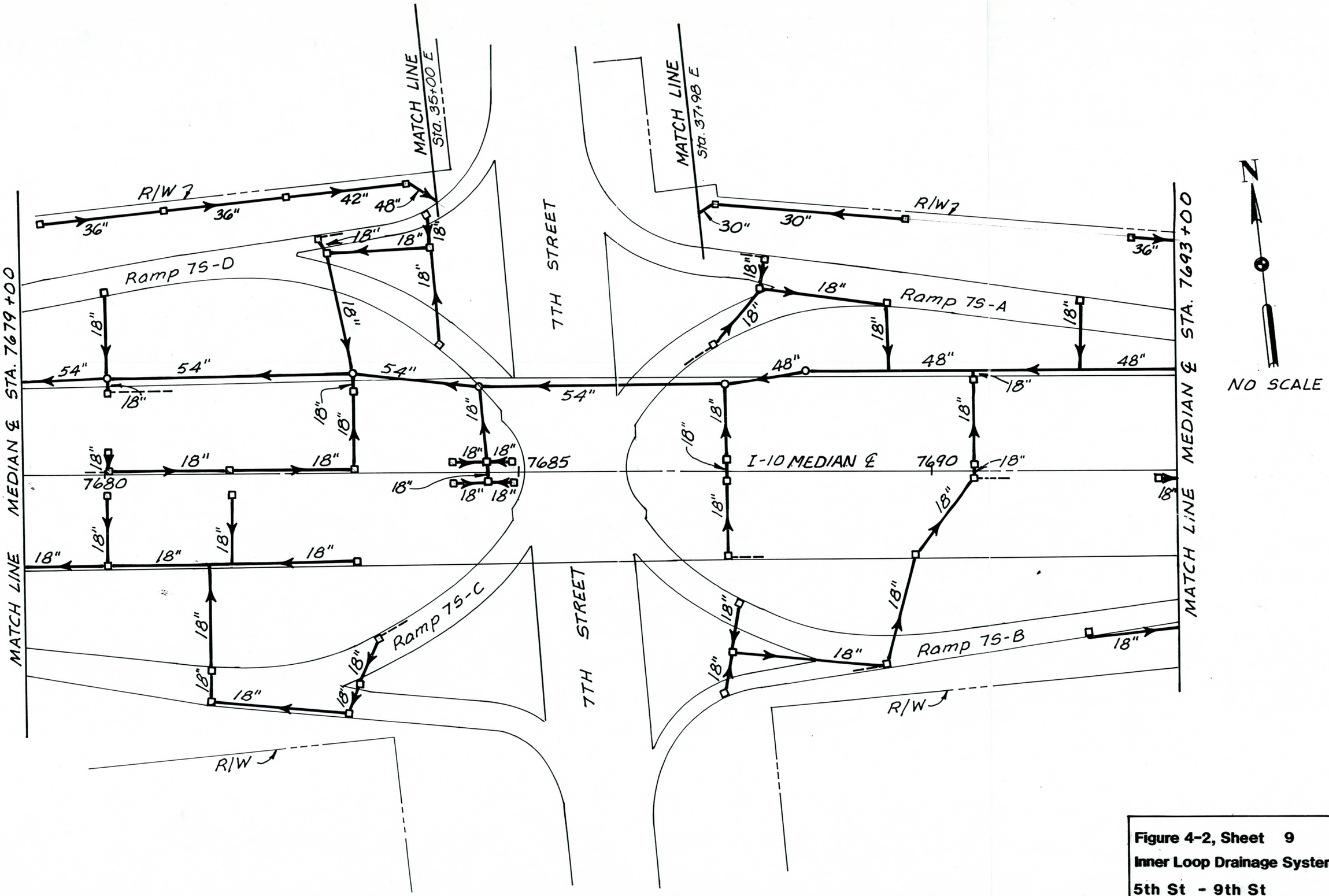


Figure 4-2, Sheet 9
Inner Loop Drainage System
5th St - 9th St

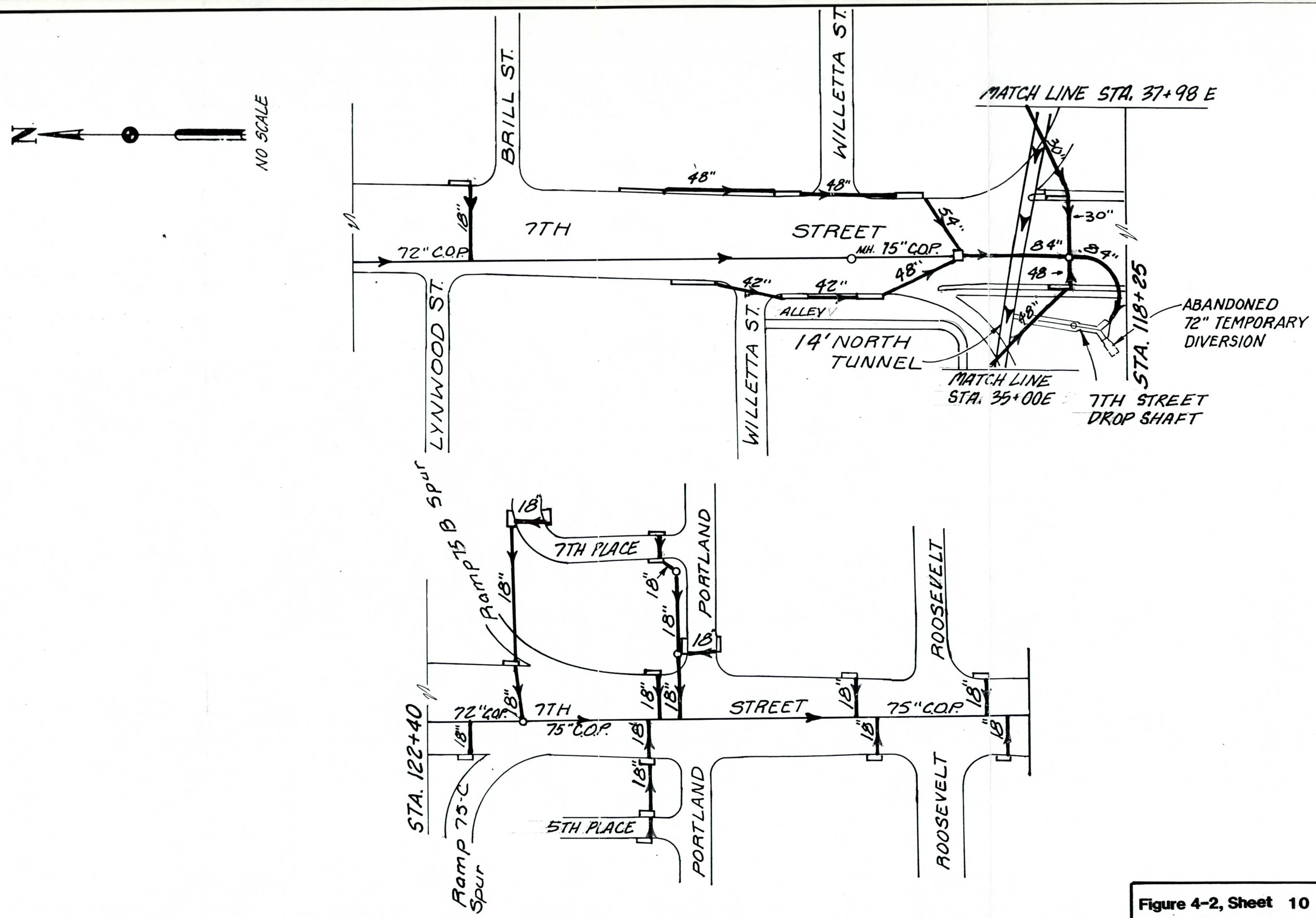


Figure 4-2, Sheet 10

Inner Loop Drainage System

7th St.

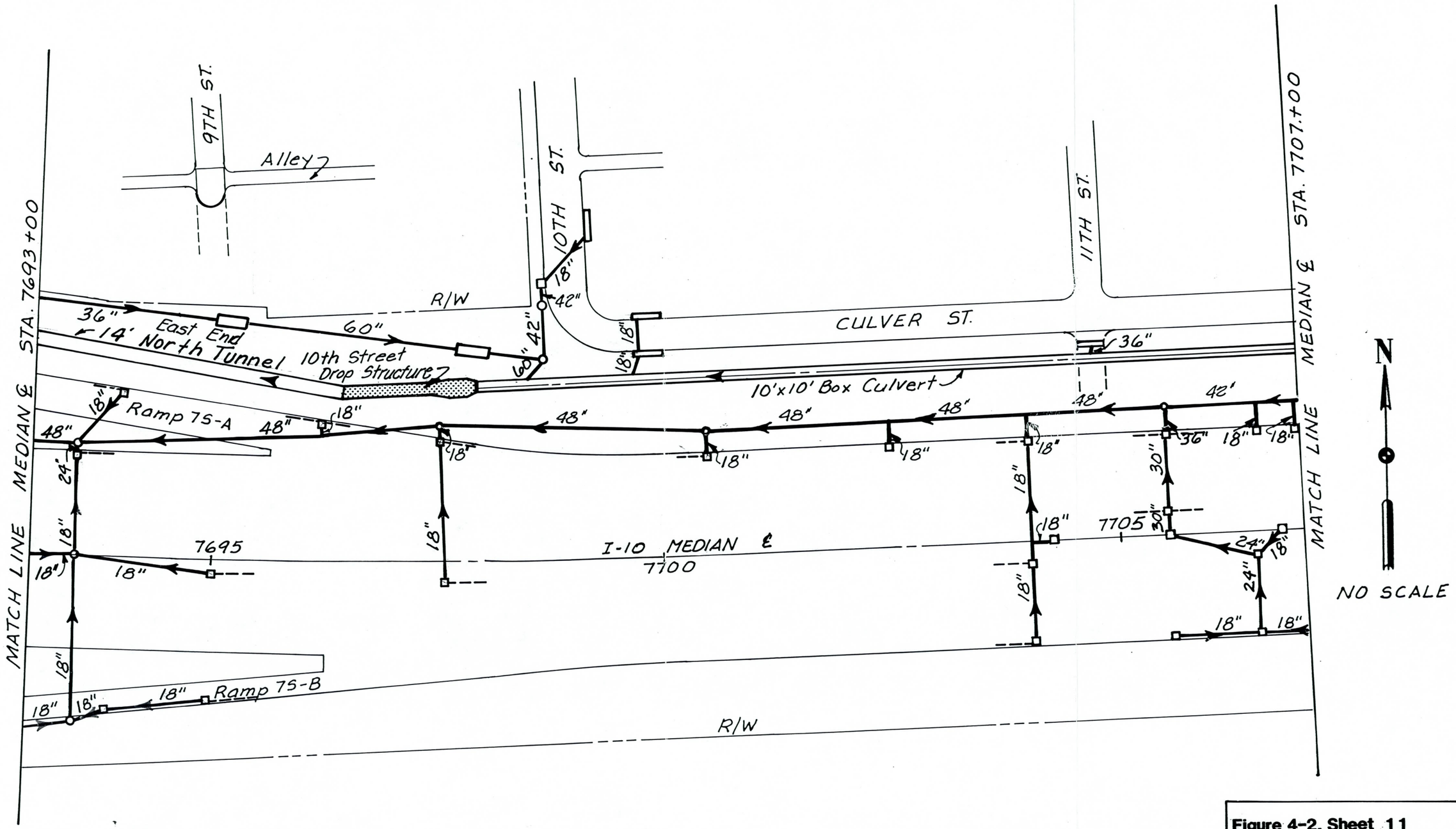


Figure 4-2, Sheet 11
Inner Loop Drainage System
9th St. - 11th St.

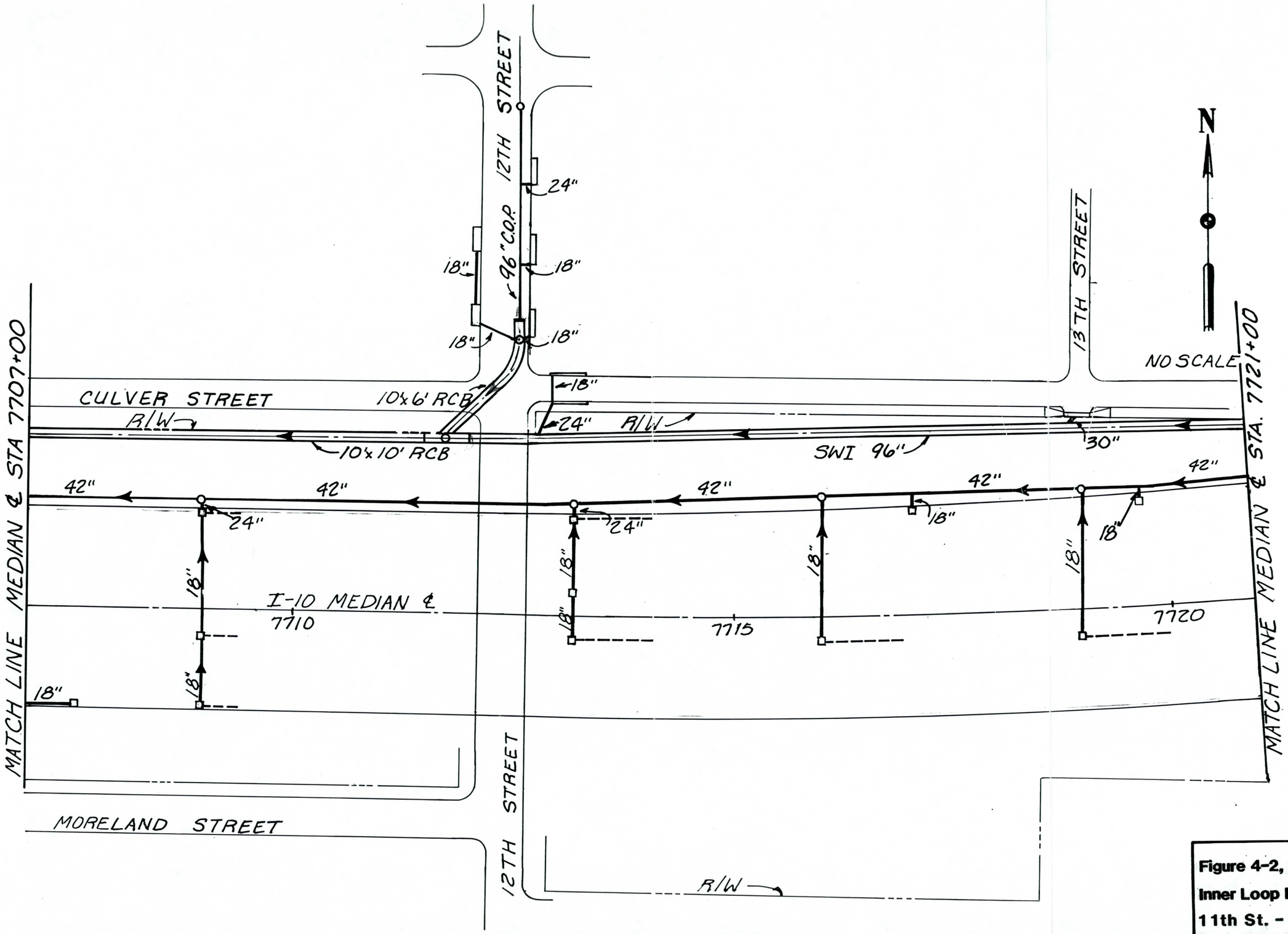


Figure 4-2, Sheet 12
Inner Loop Drainage System
11th St. - 13th St.

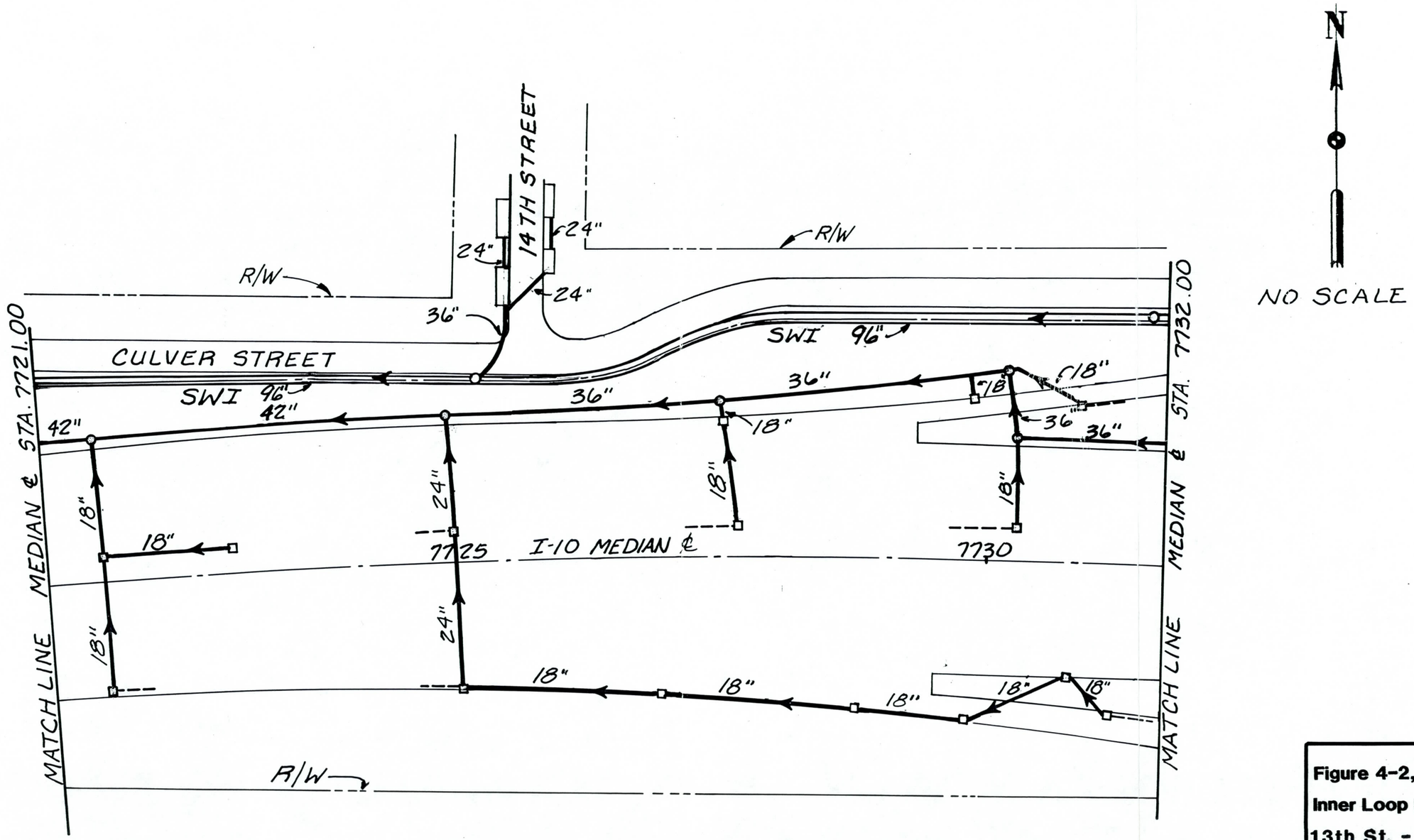


Figure 4-2, Sheet 13
Inner Loop Drainage System
13th St. - 15th St.

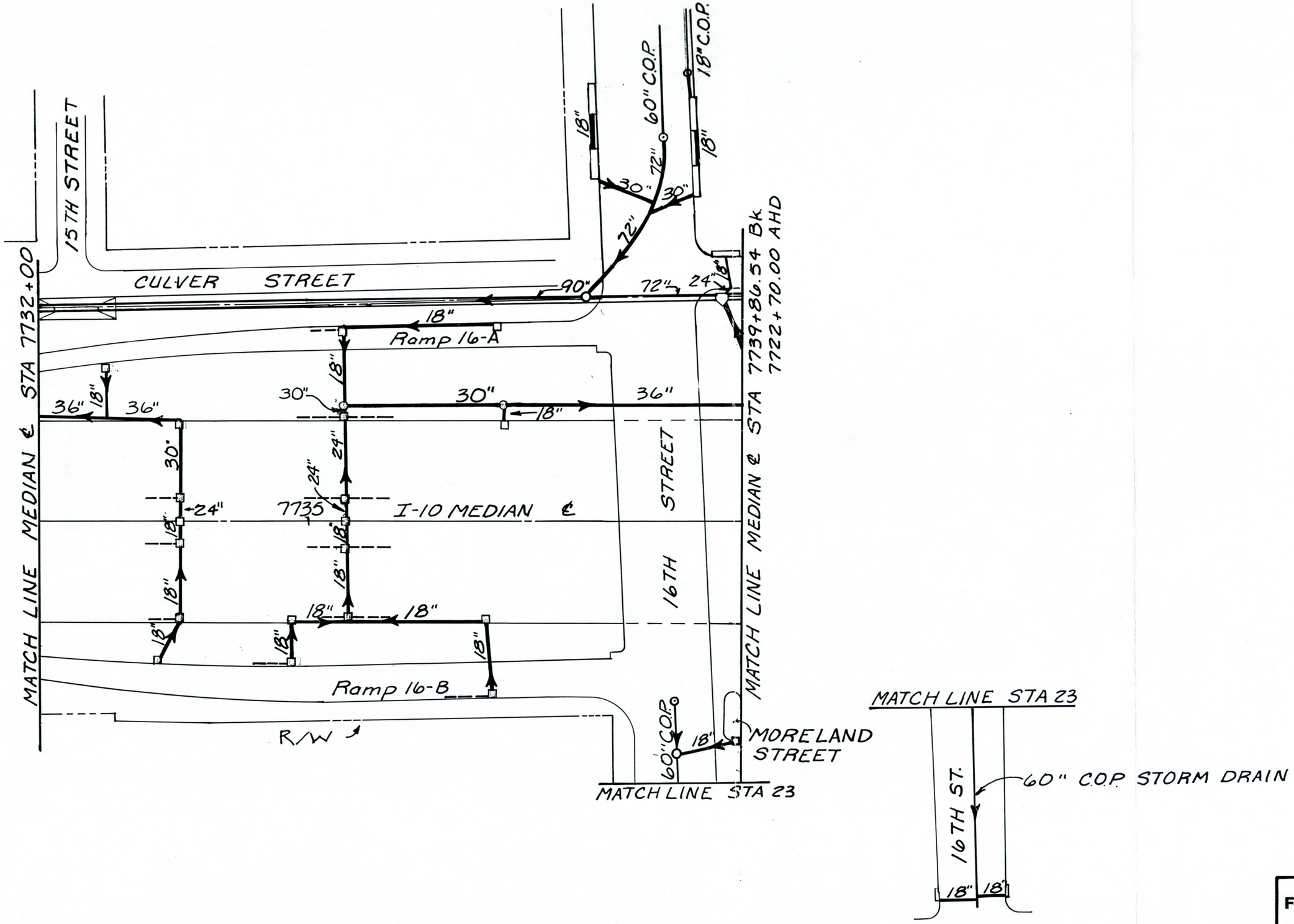


Figure 4-2, Sheet 14
Inner Loop Drainage System
15th St. - 16th St.

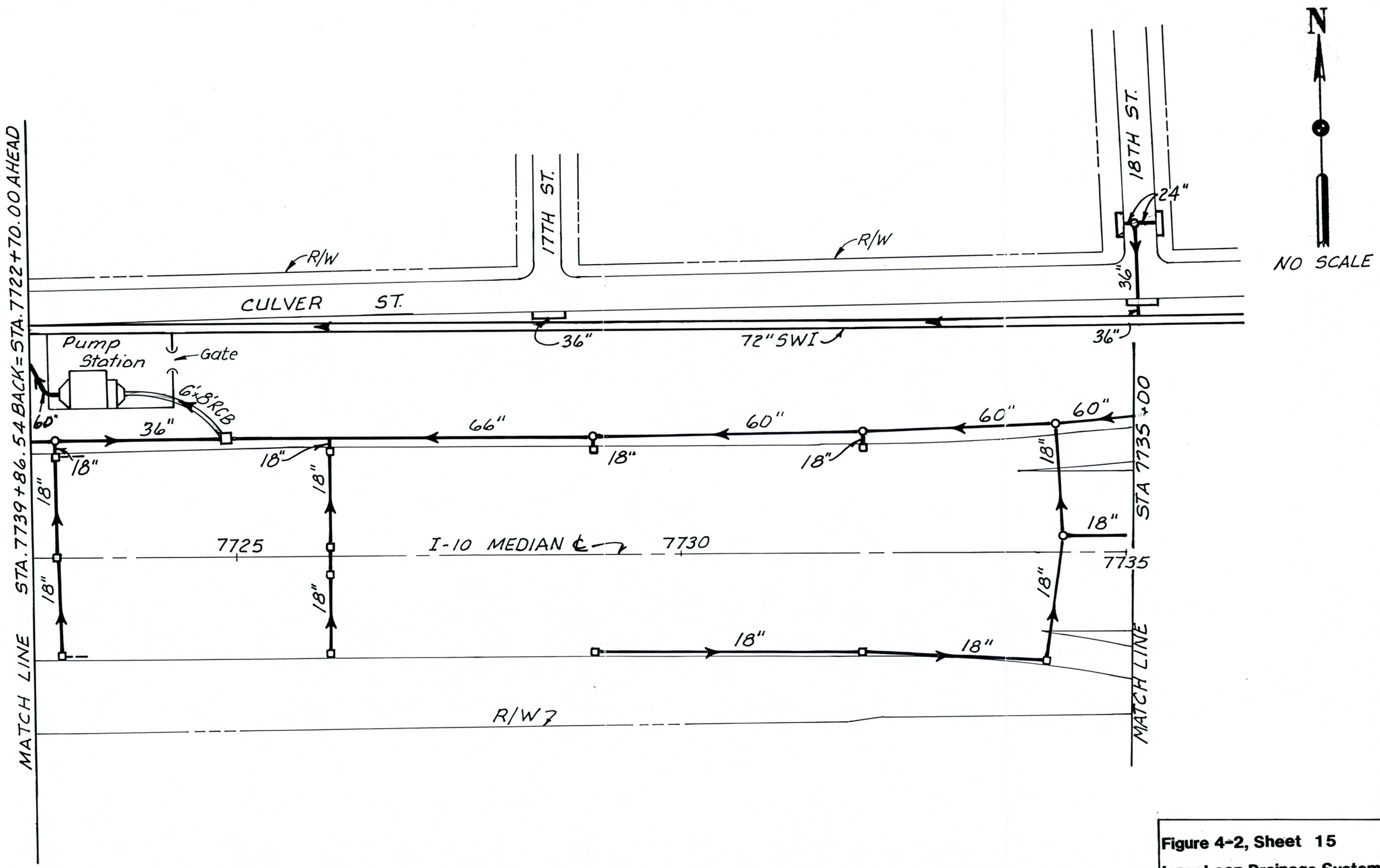


Figure 4-2, Sheet 15
Inner Loop Drainage System
16th St. - 18th St.

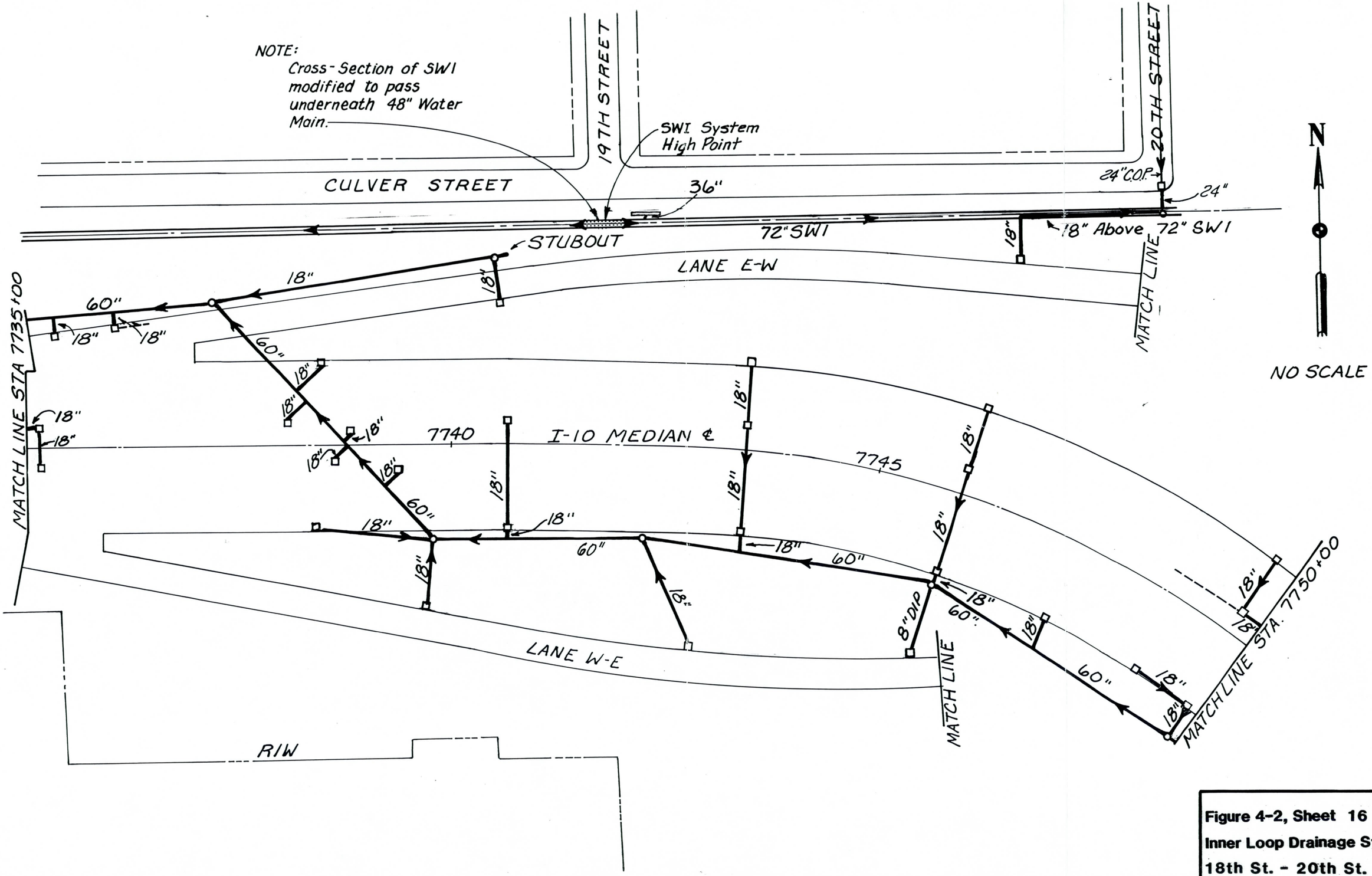


Figure 4-2, Sheet 16
Inner Loop Drainage System
18th St. - 20th St.

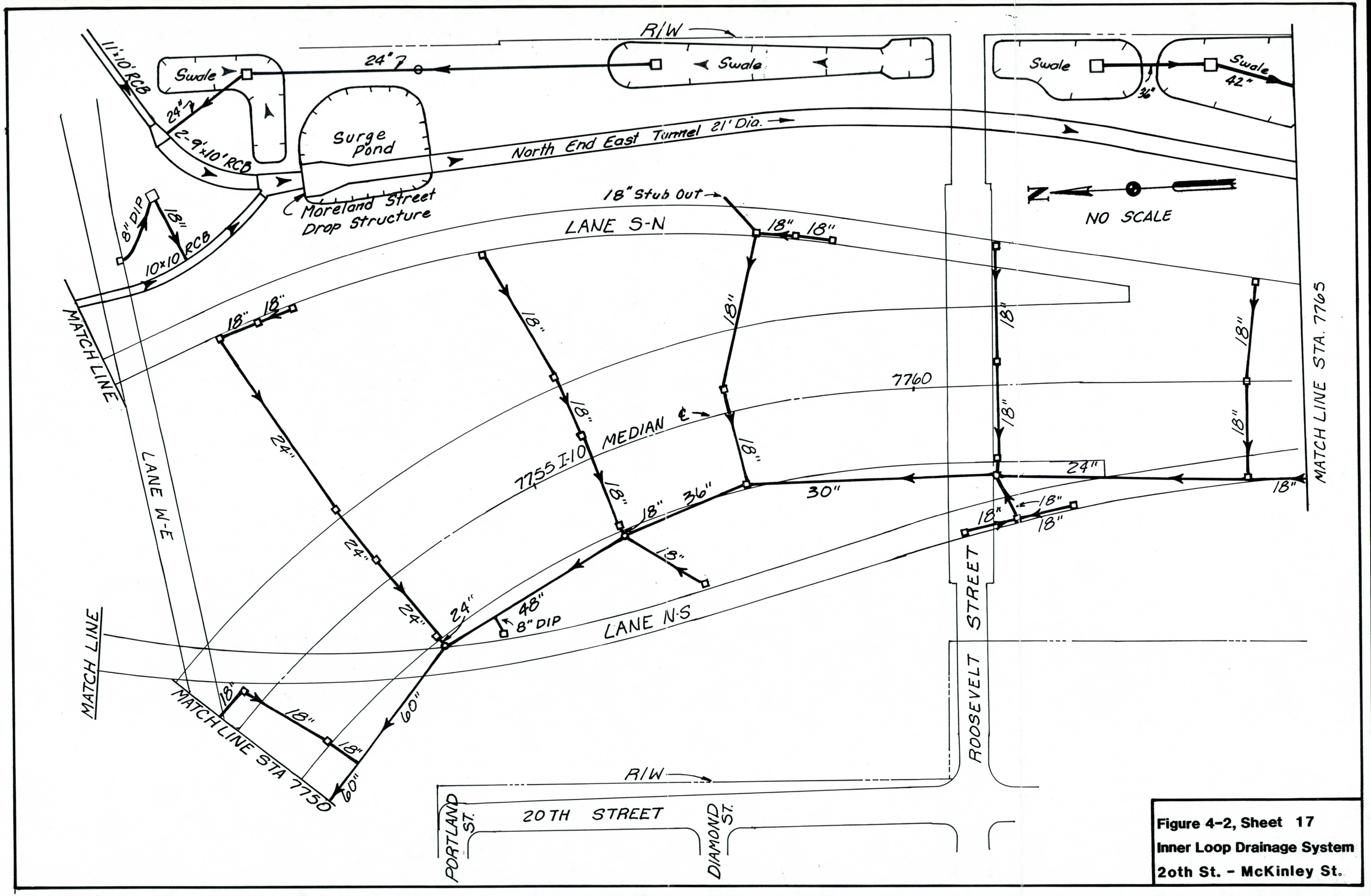


Figure 4-2, Sheet 17
Inner Loop Drainage System
20th St. - McKinley St.

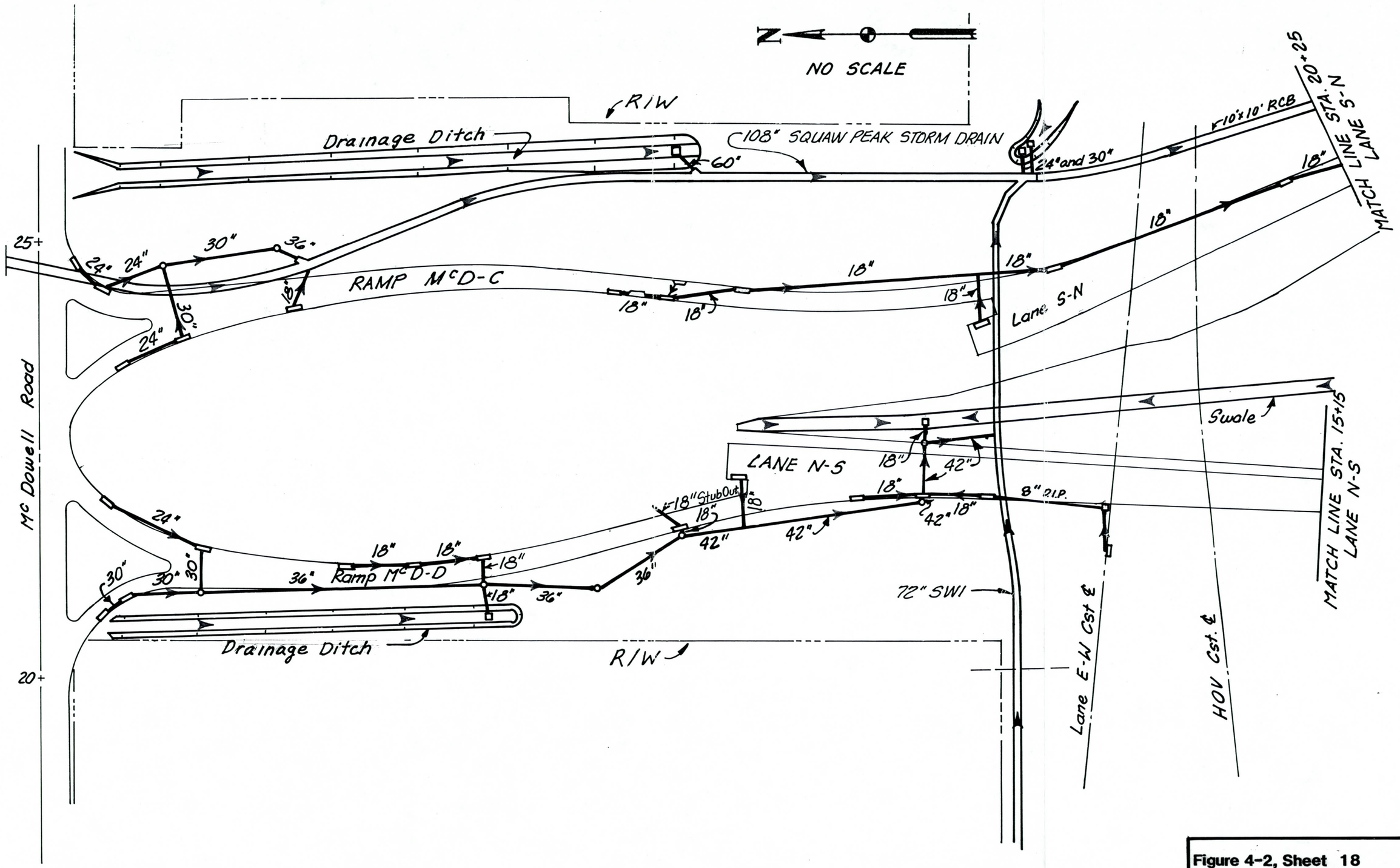


Figure 4-2, Sheet 18
Inner Loop Drainage System
McDowell Road Ramps

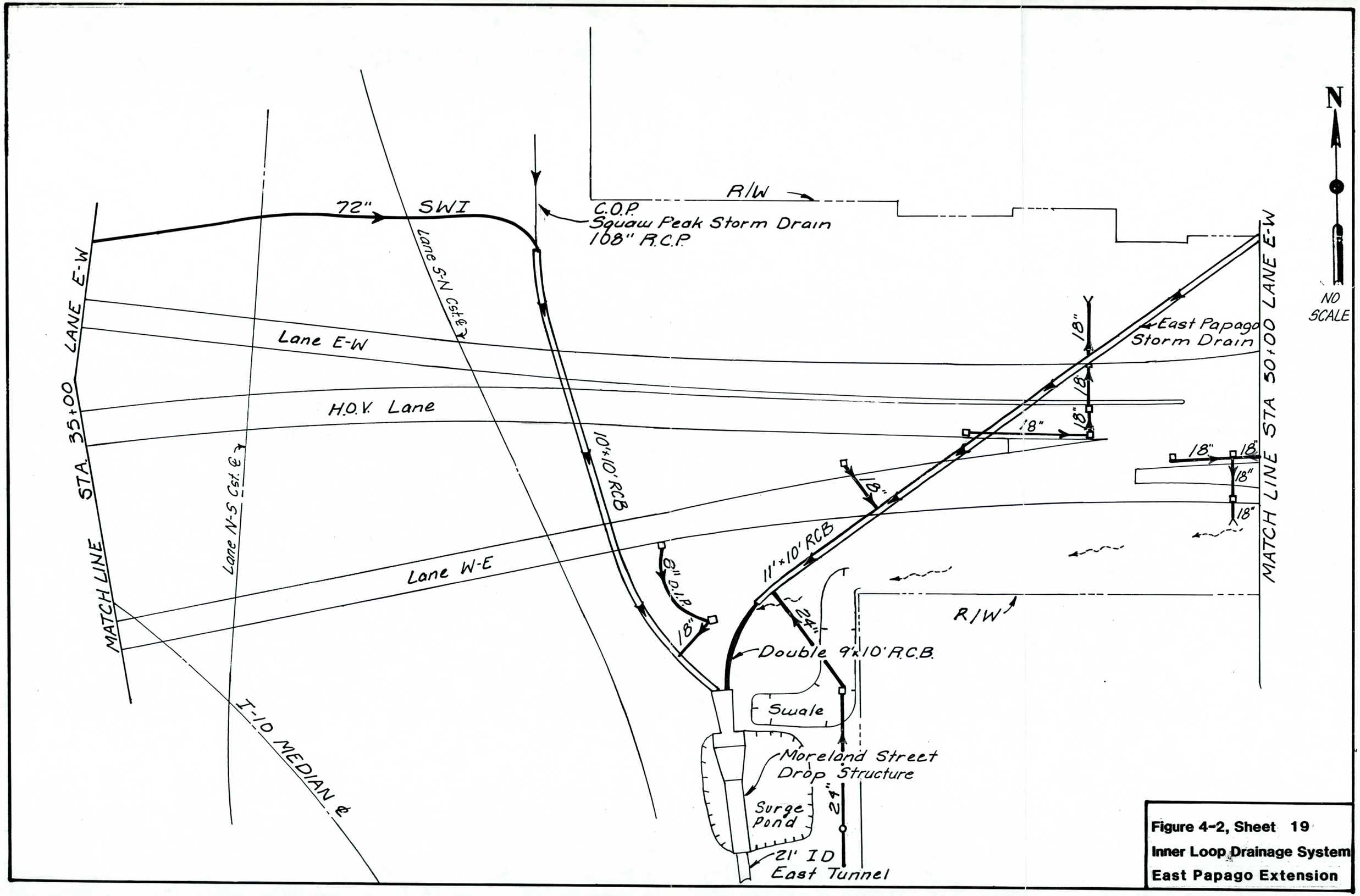


Figure 4-2, Sheet 19
Inner Loop Drainage System
East Papago Extension

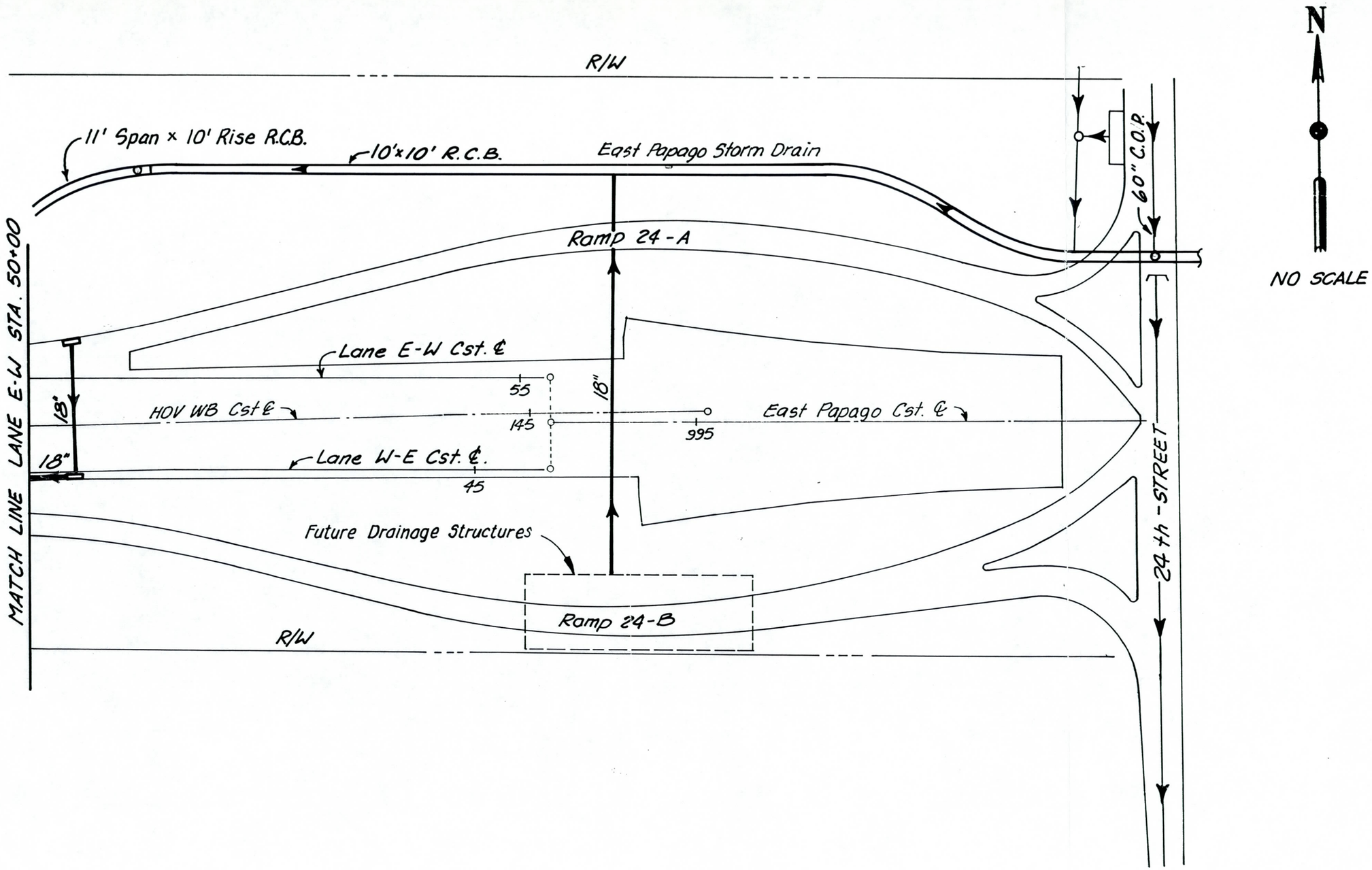


Figure 4-2, Sheet 20
Inner Loop Drainage System
East Papago Extension

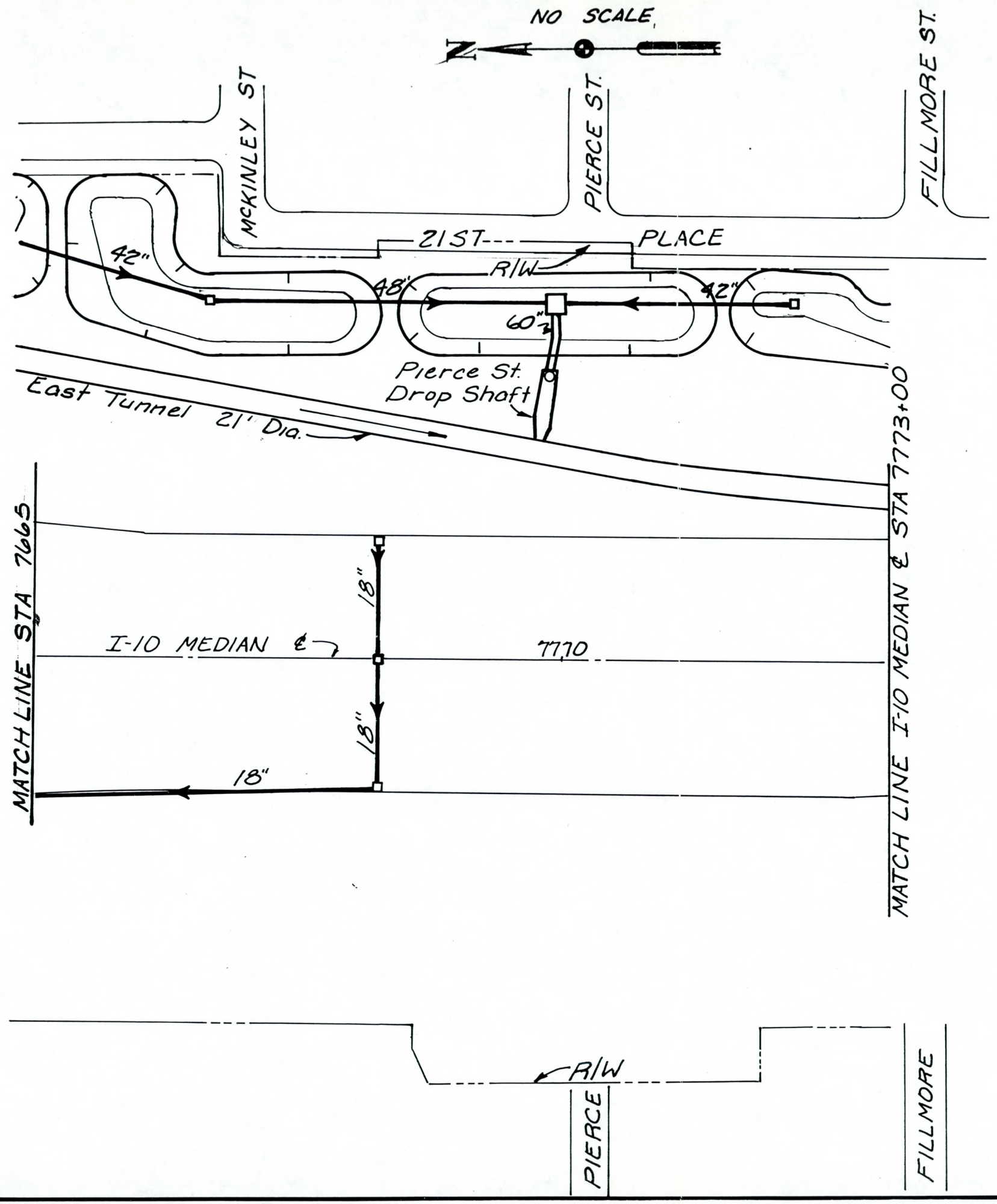


Figure 4-2, Sheet 21
Inner Loop Drainage System
McKinley St. - Fillmore St.

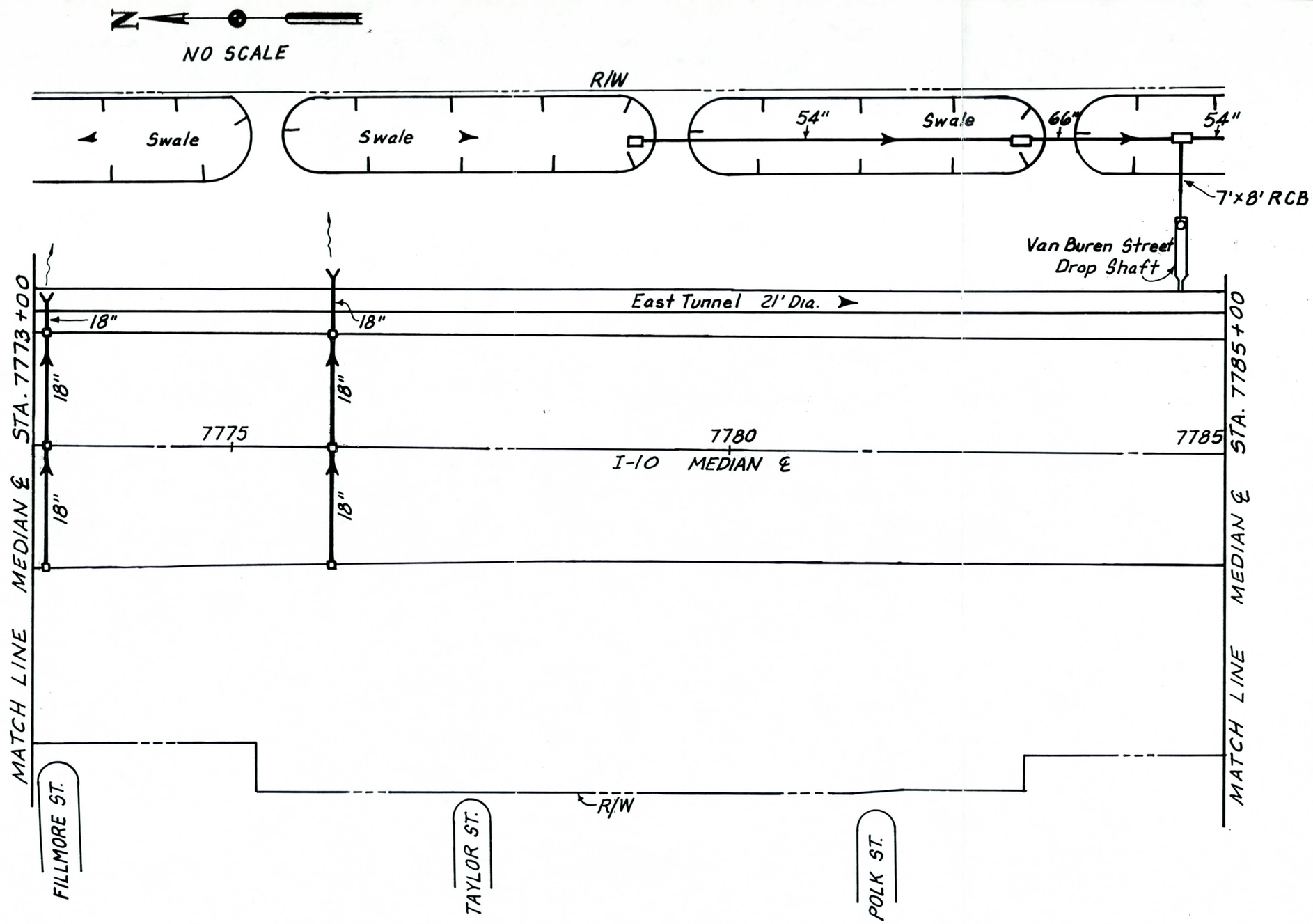


Figure 4-2, Sheet 22
Inner Loop Drainage System
Fillmore - Van Buren St.

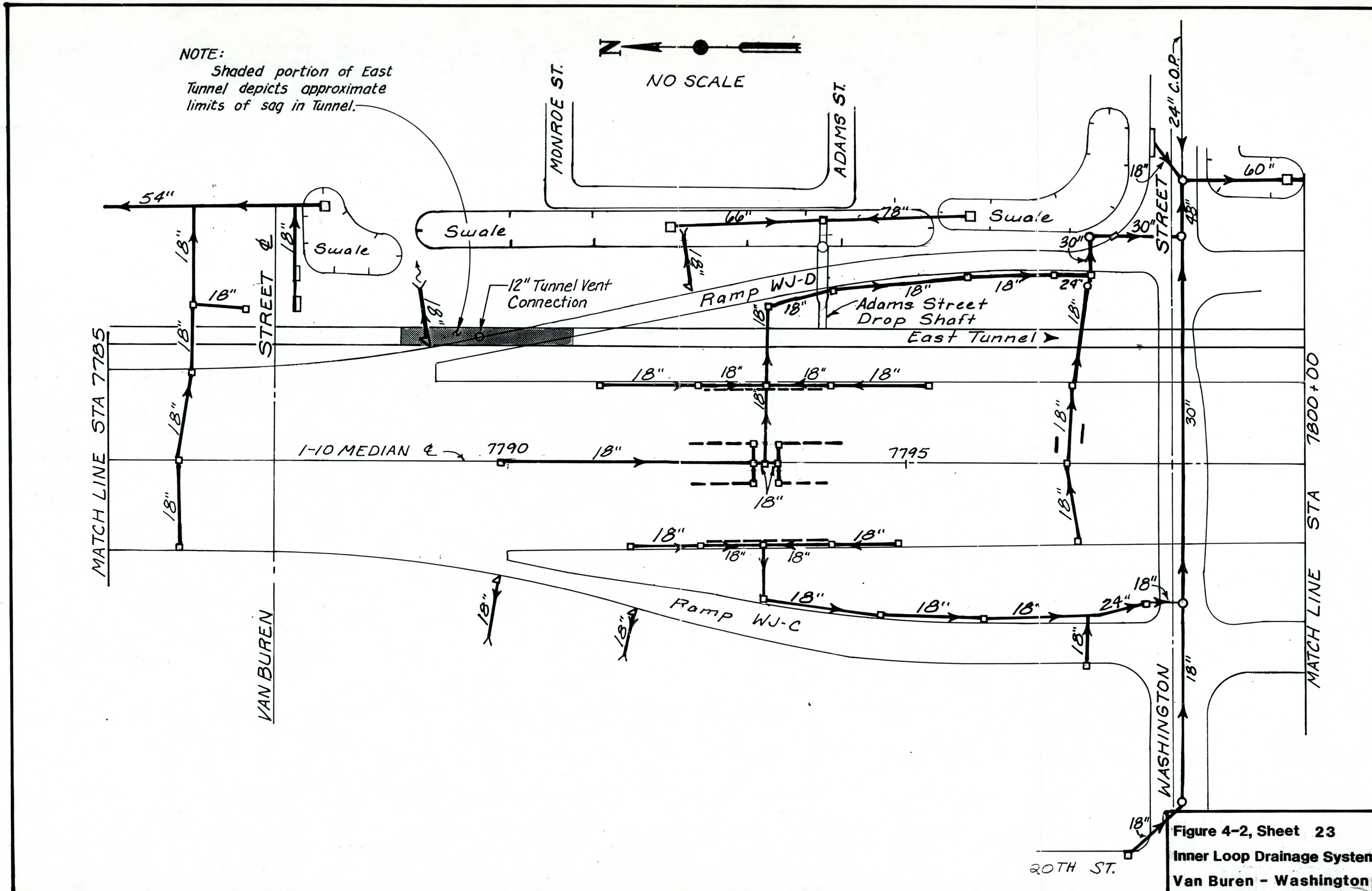


Figure 4-2, Sheet 23
Inner Loop Drainage System
Van Buren - Washington

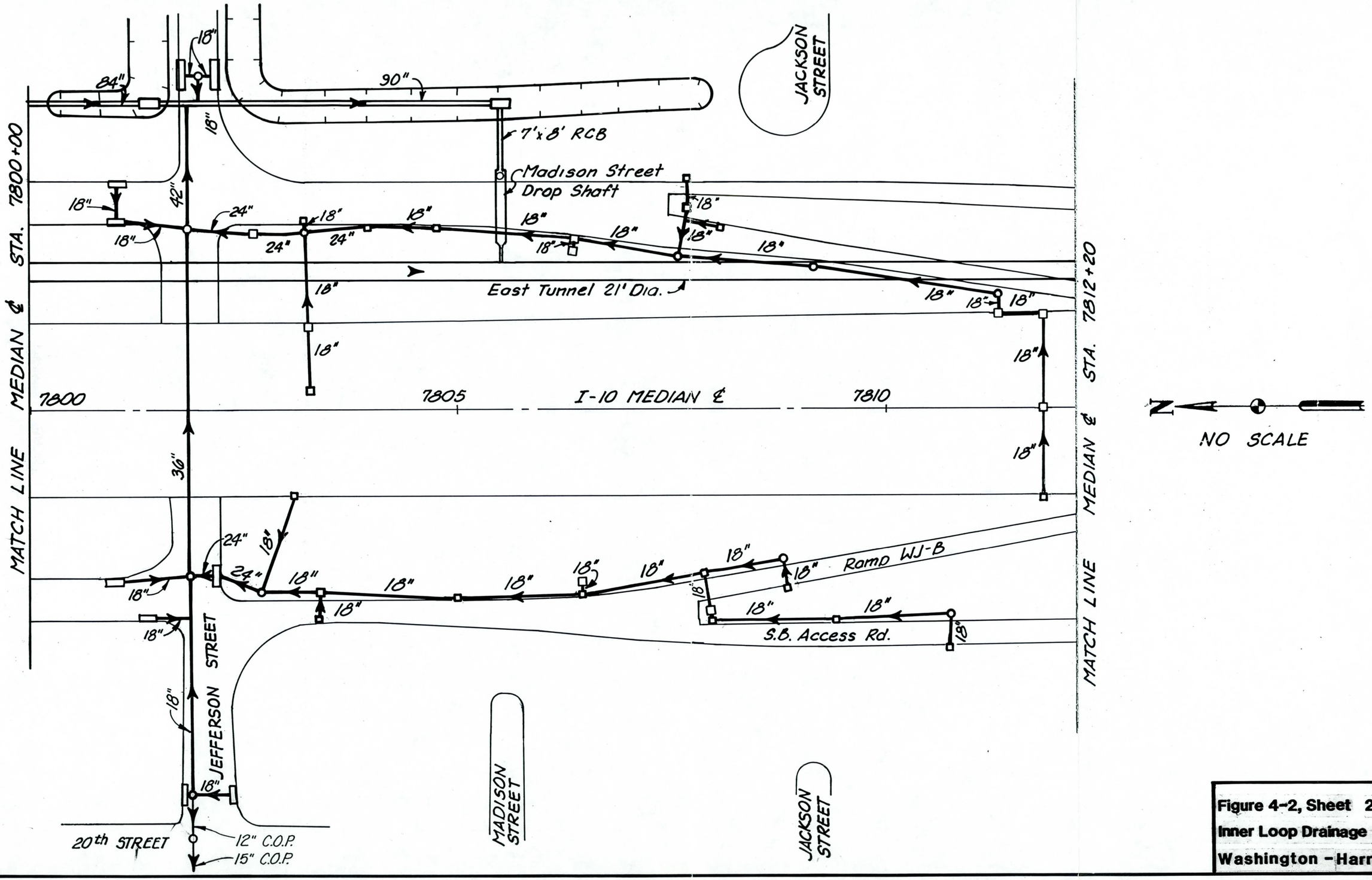


Figure 4-2, Sheet 24
Inner Loop Drainage System
Washington - Harrison St

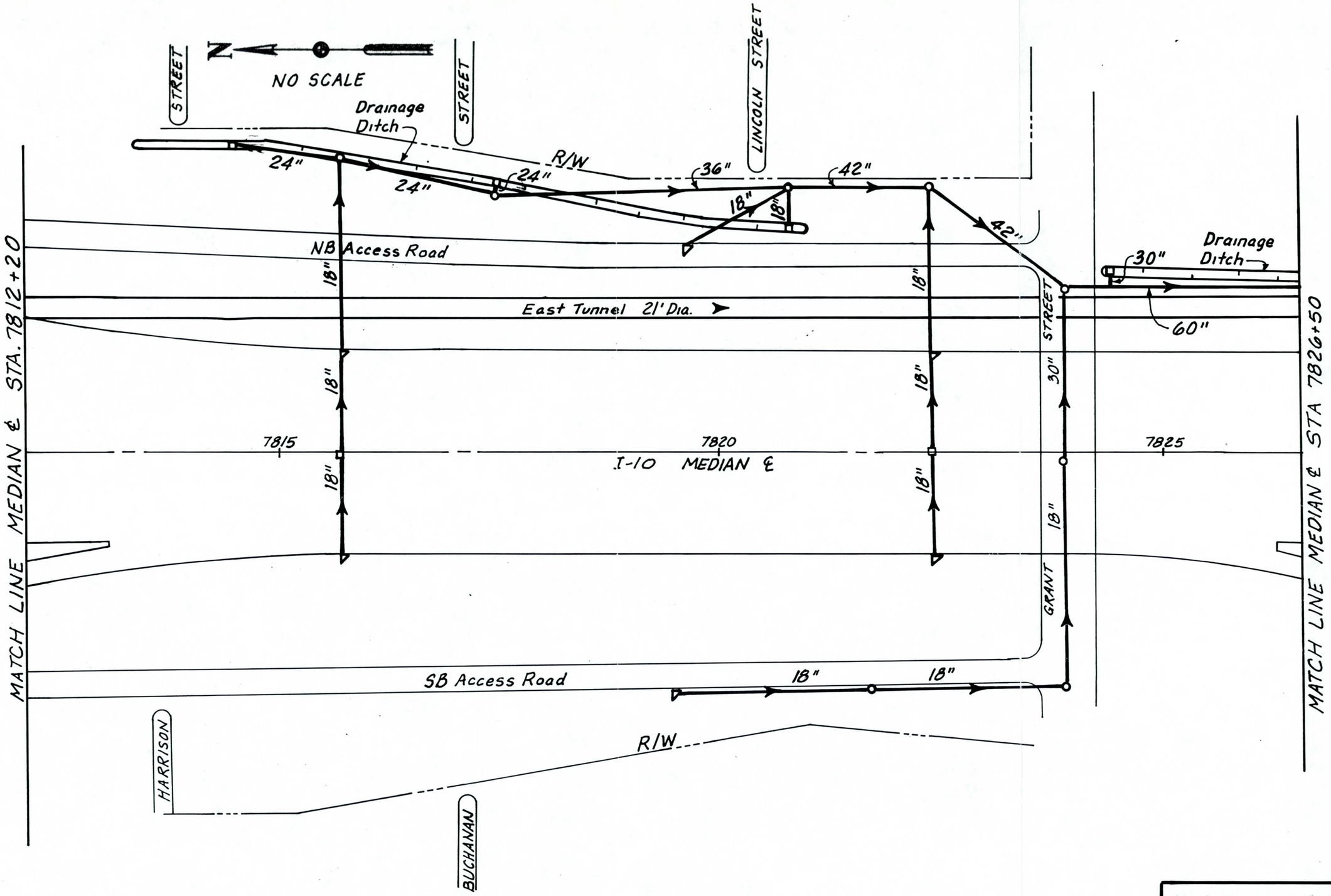


Figure 4-2, Sheet 25
Inner Loop Drainage System
Harrison - Sherman St.

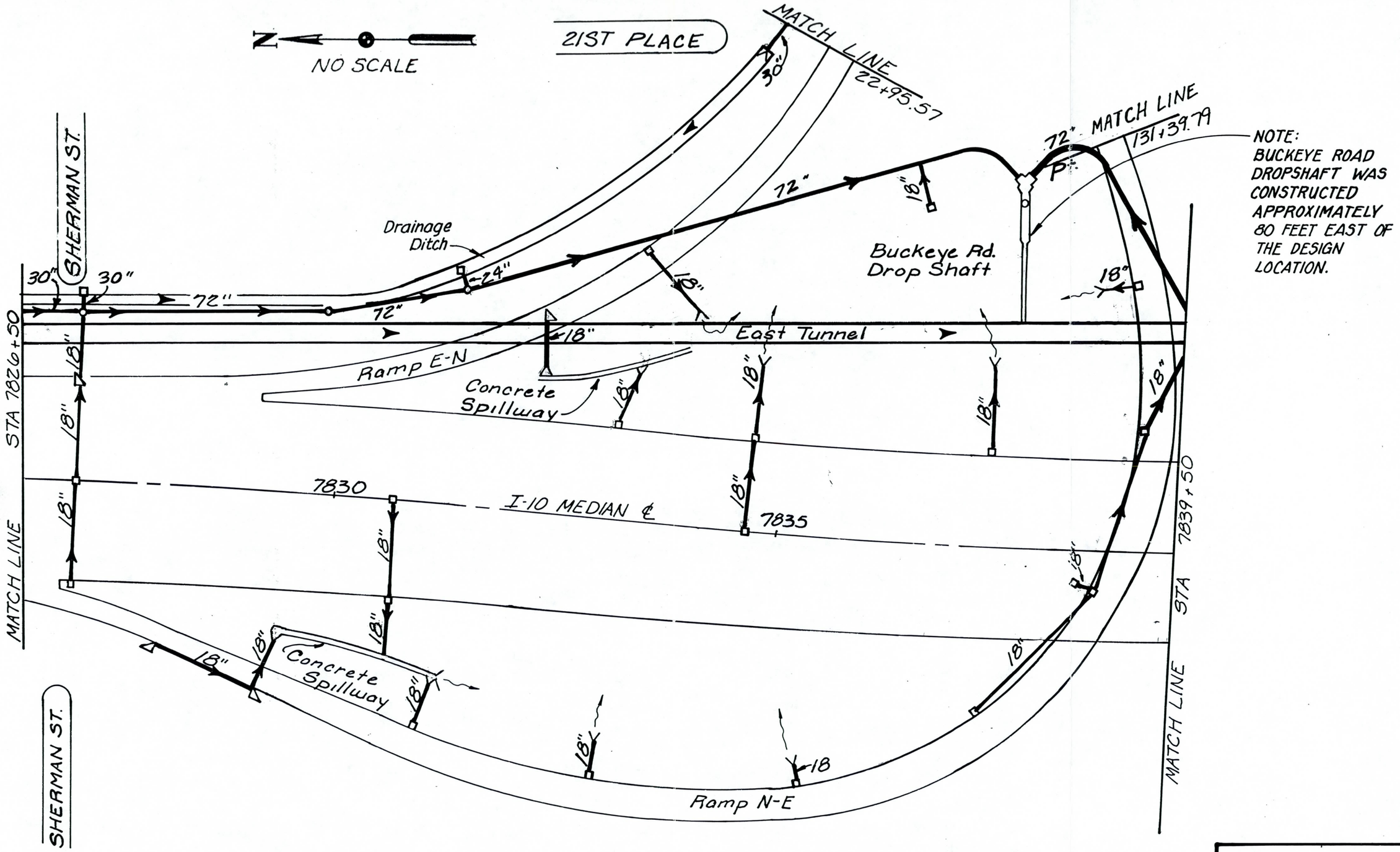


Figure 4-2, Sheet 26
Inner Loop Drainage System
Sherman - Buckeye Rd.

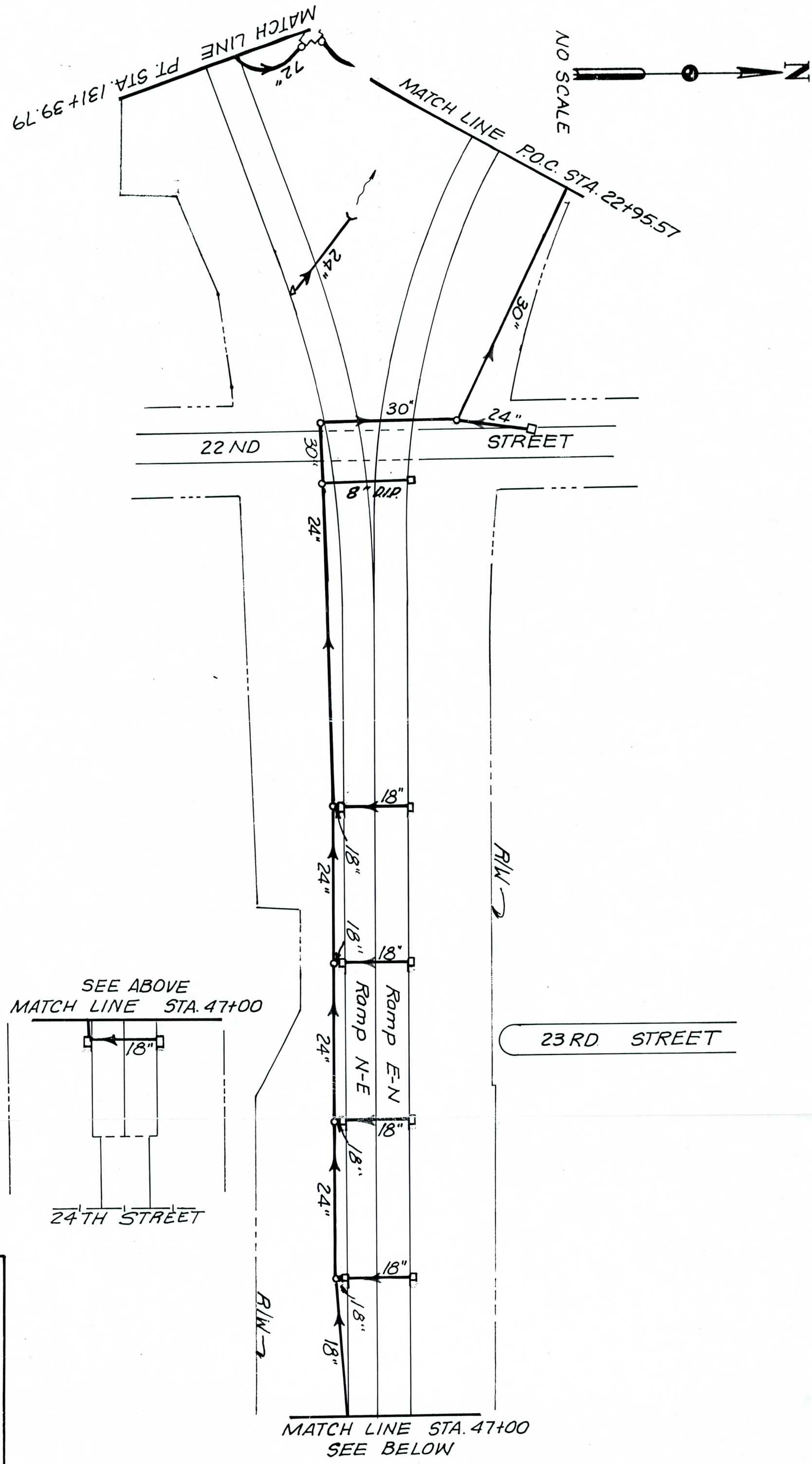


Figure 4-2, Sheet 27
Inner Loop Drainage System
Sky Harbor Access Ramps

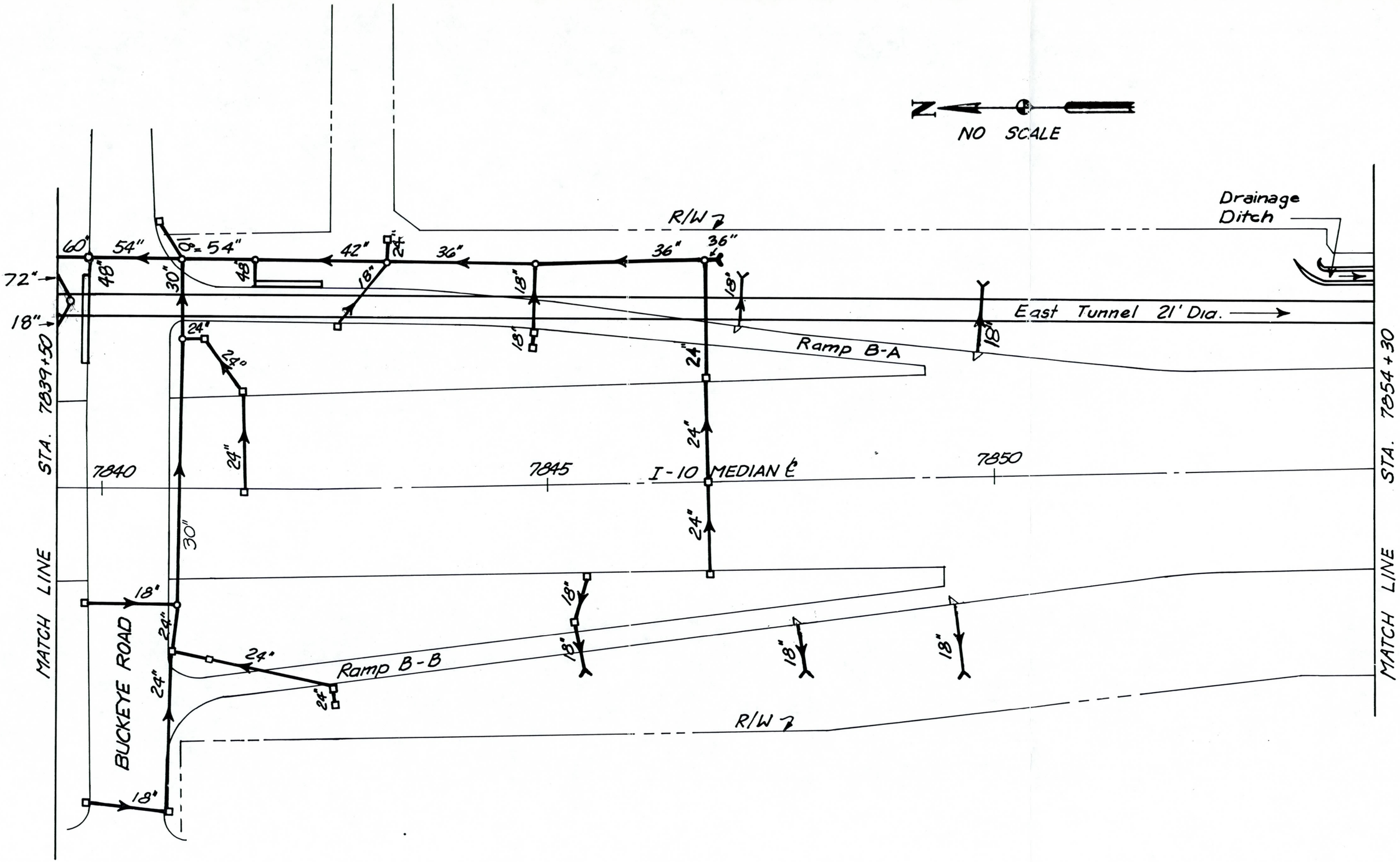


Figure 4-2, Sheet 28
Inner Loop Drainage System
Buckeye Rd - Mohave St.

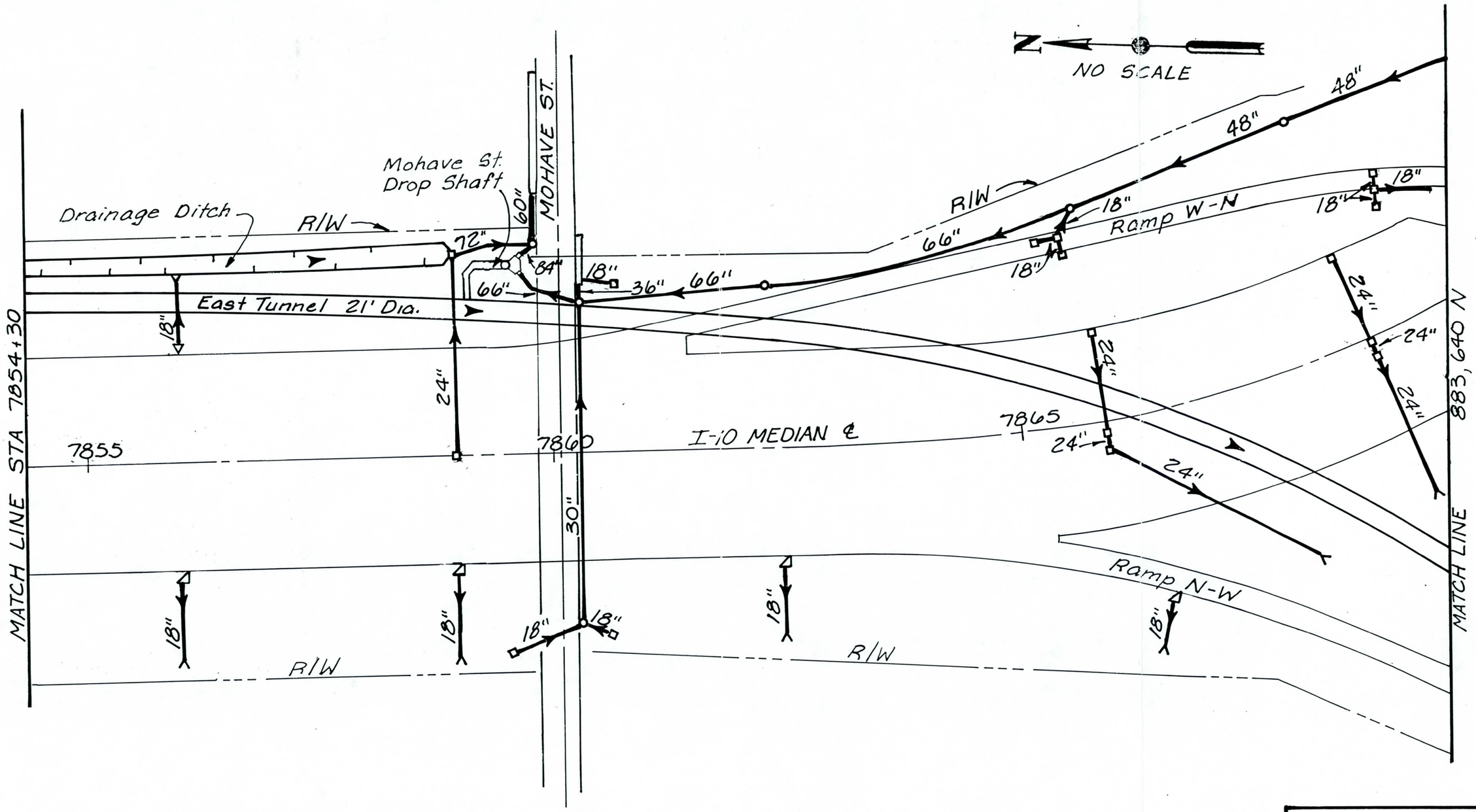


Figure 4-2, Sheet 29
Inner Loop Drainage System
Mohave St. - Maricopa TI

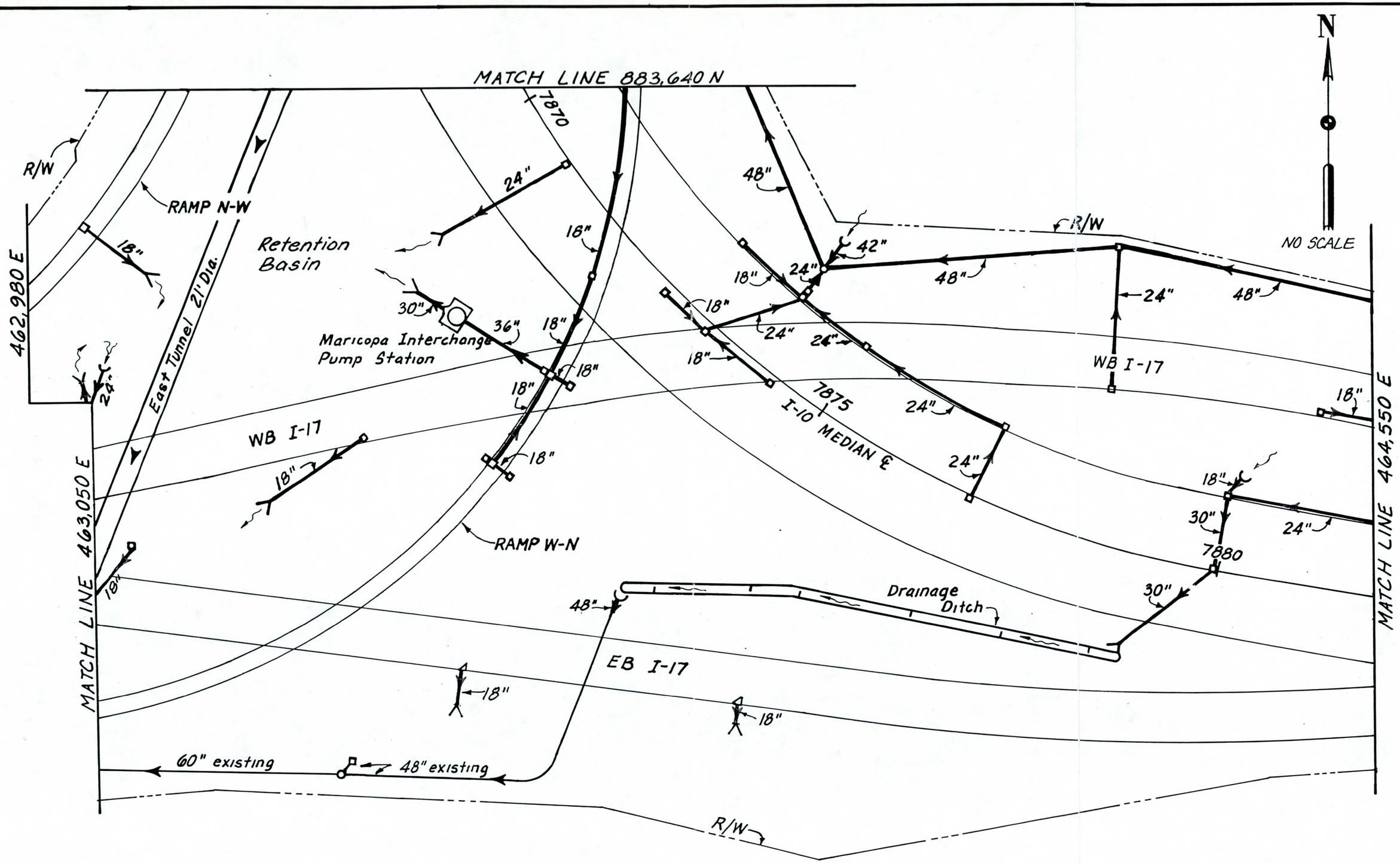


Figure 4-2, Sheet 30
Inner Loop Drainage System
Maricopa TI

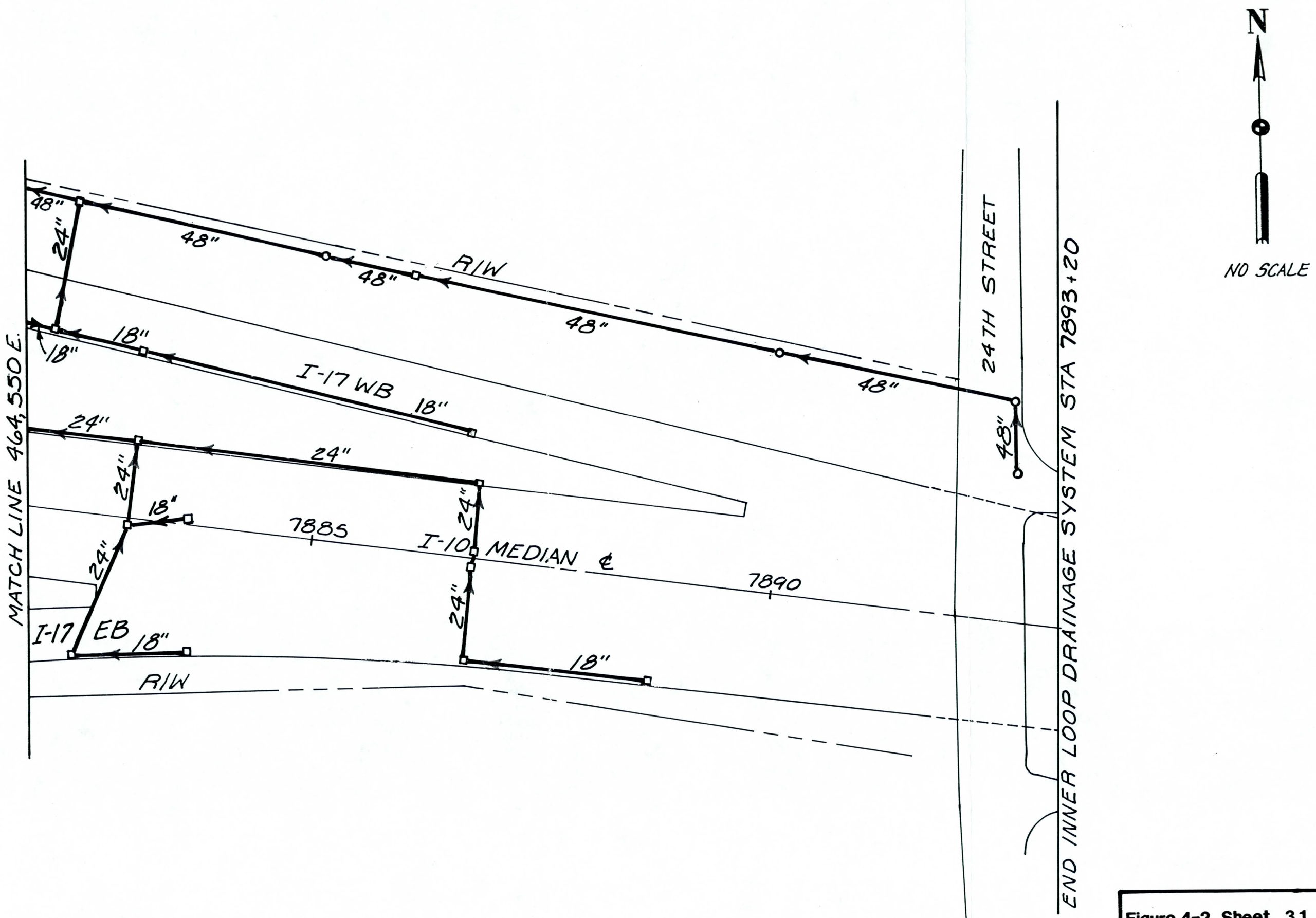


Figure 4-2, Sheet 31
Inner Loop Drainage System
Maricopa TI

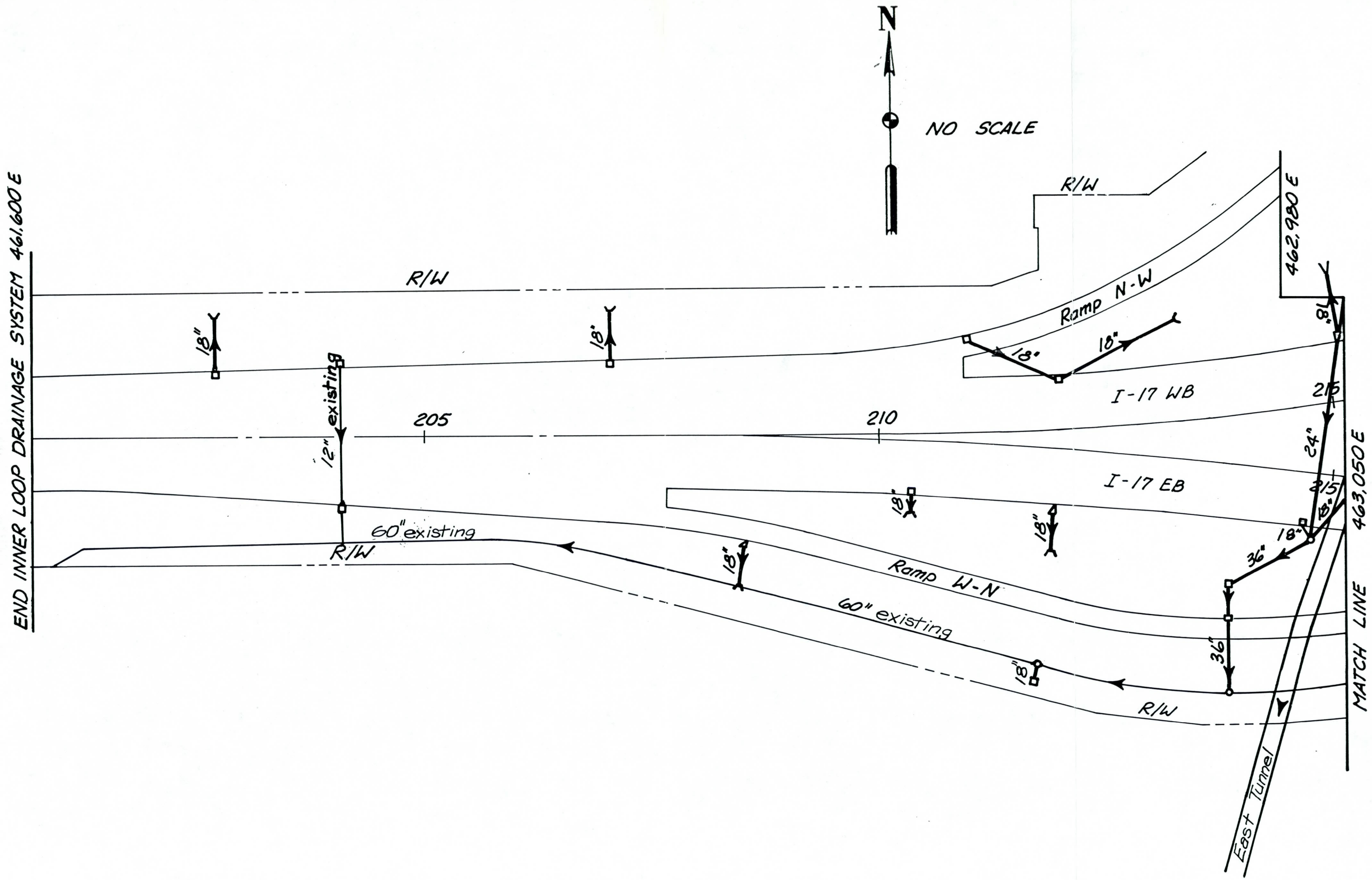


Figure 4-2, Sheet 32
Inner Loop Drainage System
Maricopa TI

CHAPTER 5 – FREEWAY PUMP STATIONS

SYNOPSIS

This chapter covers the design of the four freeway pump stations used in the Inner Loop storm drainage system. Site plans and station layouts are included.

I. INTRODUCTION

Four pump stations remove storm water from depressed portions of the I-10 Inner Loop freeway. On-site collector systems convey runoff to these stations, which pump the storm water into the main Inner Loop storm drain system for eventual discharge to the Salt River.

II. LOCATIONS

Three of the pump stations are located immediately north of the east-west leg of the Inner Loop at Third Avenue, Third Street and Sixteenth Street. These sites were chosen because they have good access, good foundation material, and they are separated from freeway traffic. One basic design was adapted for use at each of these sites.

The fourth pump station is located at the Maricopa Traffic Interchange. It pumps storm water from the depressed ramp W-N to a retention basin just north of I-17. The design of this station differs from the other three, since it only drains one ramp of the interchange.

A. Third Avenue Pump Station

The Third Avenue pump station is located north of ramp 7A-A between Third Avenue and Fifth Avenue. Access is provided by a gravel road running south off Culver Street. Figure 5-1 shows the complete site plan.

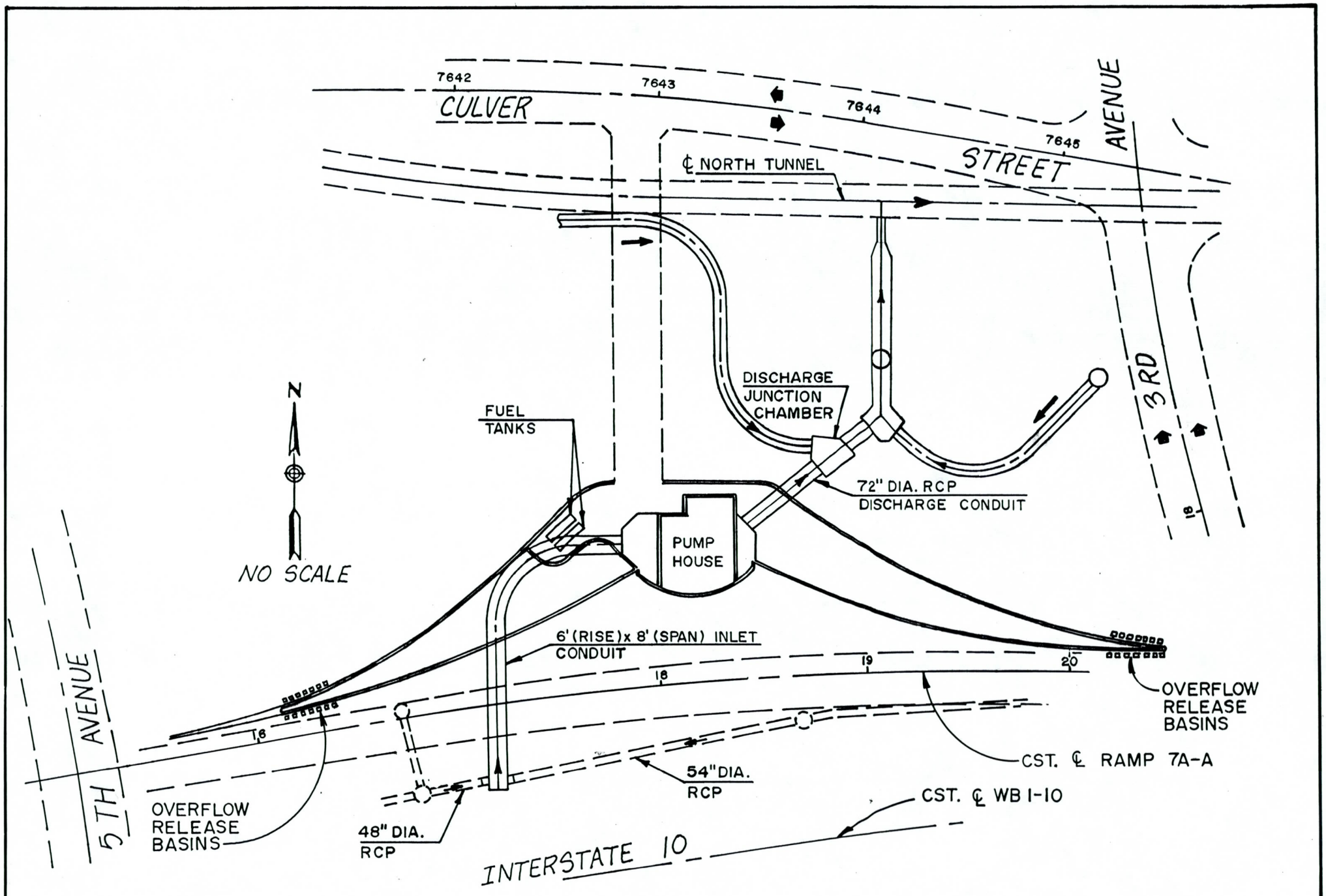


Figure 5-1

3rd Avenue Pump Station Site Plan

The inlet to the pump station is a junction chamber in the local freeway collector system at I-10 westbound station 7642+50, 48' Lt. The junction chamber routes flow from the collector system through a 6-foot rise by 8-foot span concrete box culvert inlet to the pump station.

Outflow from the Third Avenue pump station enters the North Tunnel through the Third Avenue drop shaft.

B. Third Street Pump Station

The Third Street pump station is located north of I-10 Ramp 7S-D, east of Third Street. Access is gained from an alley running east off Third Street (See Figure 5-2).

The inlet to the pump station is a 6-foot rise by 8-foot span box culvert which begins at a junction point located at I-10 median station 7677+19.50, 109' Lt. This junction point receives storm flow from the local freeway collector system.

The pump station discharges to the North Tunnel through the Third Street dropshaft.

C. Sixteenth Street Pump Station

The Sixteenth Street pump station is located north of the westbound I-10 traffic lane, east of Sixteenth Street. Access to the station is provided by a driveway on the south side of Culver Street (See Figure 5-3).

The inlet to the pump station accepts flow from the local freeway collector system at a junction point located at I-10 median station 7724+86.54, 133' Lt.

The Sixteenth Street pump station discharges to the Storm Water Interceptor through a connection just east of Sixteenth Street.

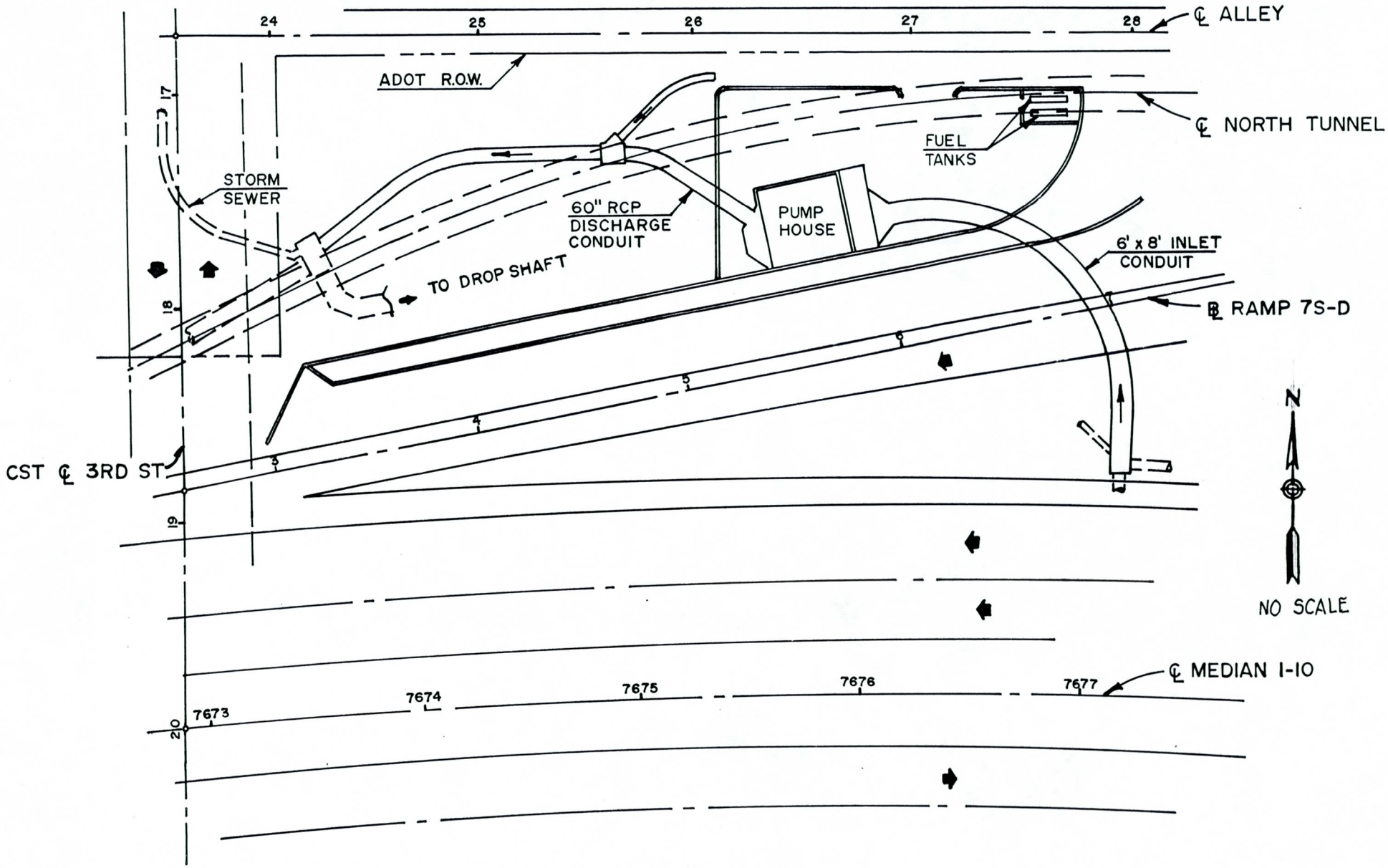


Figure 5-2

3rd Street Pump Station Site Plan

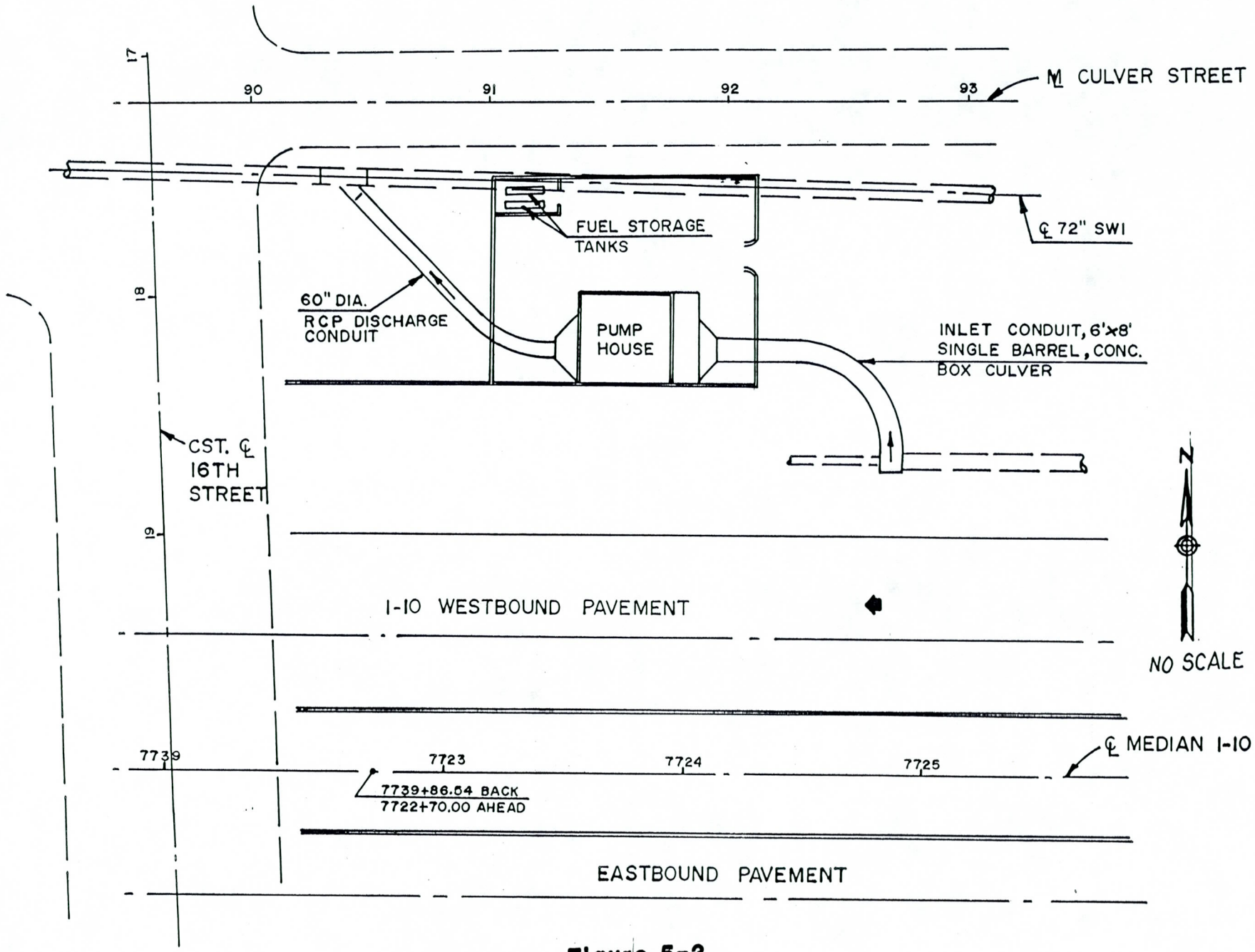


Figure 5-3

16th Street Pump Station Site Plan

D. Maricopa Traffic Intercharge Pump Station

The Maricopa Traffic Interchange (MTI) pump station is located in the triangular area enclosed by the westbound lane of I-17, the eastbound lane of I-10, and Ramp N-W at the Maricopa Interchange. The pump house itself is northwest of the depressed section of Ramp W-N. A gravel access road leads from Ramp N-W to the MTI pump station (See Figure 5-4).

A 36-inch diameter circular reinforced concrete pipe conducts flow from the storm water collection point at Ramp W-N station 222+24 to the pump station.

The MTI pump station discharges to Retention Basin A, which covers most of the remaining area enclosed by I-10, westbound I-17 and Ramp N-W.

III. HYDROLOGIC AND HYDRAULIC DESIGN CRITERIA

Generally, design of the pump stations follows the practices and procedures outlined in the FHWA publication "Manual for Highway Storm Water Pumping Stations", Vol. 1 and 2, dated October, 1982.

A. Flows

Peak flows were estimated using the Rational Method, outlined in previous chapters. Design inflow hydrographs were derived using the Baumgardner Method II. This method uses Rational Method peaks and times of concentration in combination with hydrograph shapes from the SCS TR-55 tabular discharge tables to produce the inflow hydrographs.

Utilizing these methods, the 50-year storm event peak inflows were calculated as 165.9 cfs at Third Avenue, 185 cfs at Third Street, 181.8 cfs at Sixteenth Street, and 20.8 cfs at the MTI station.

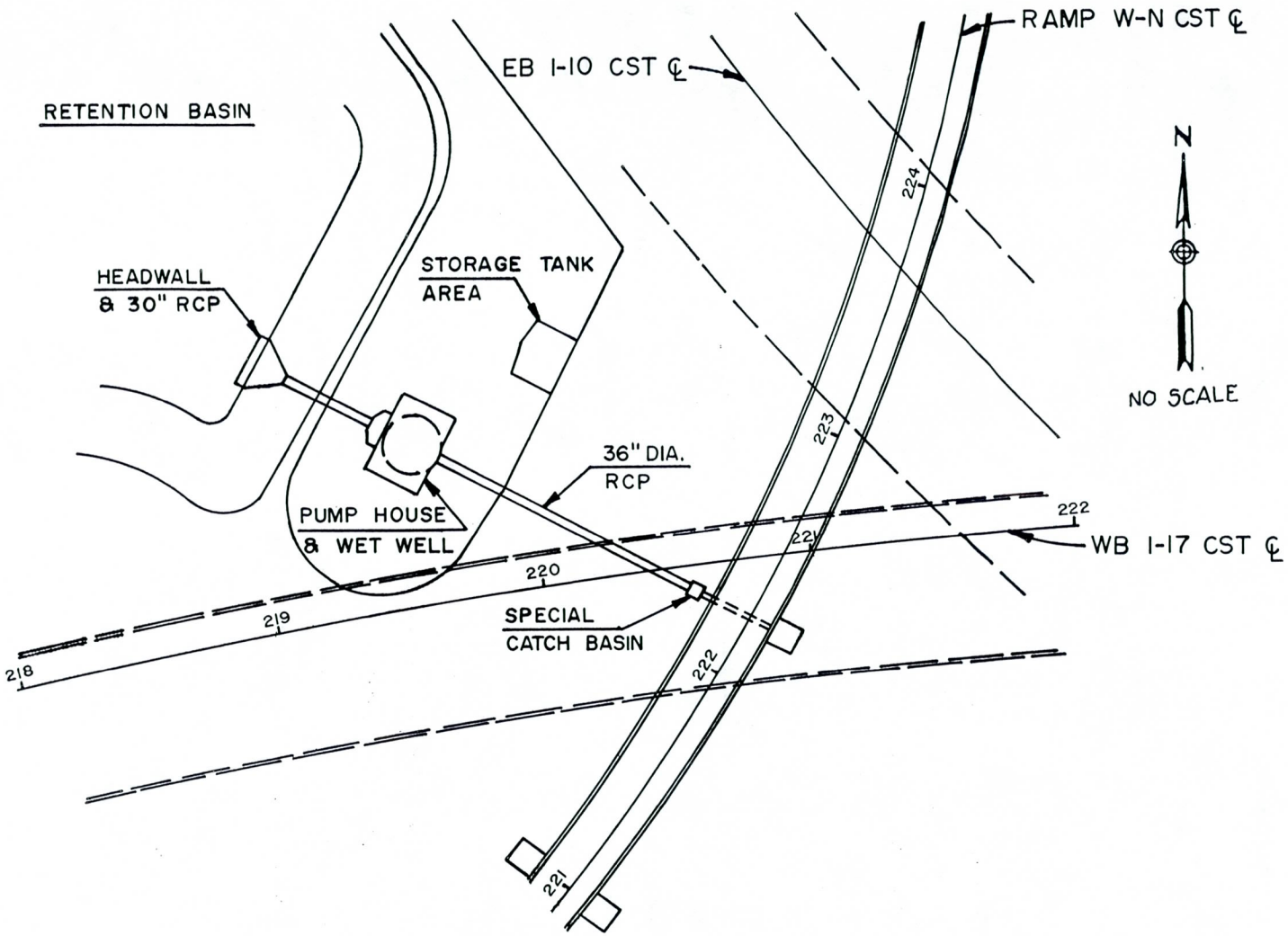


Figure 5-4

Maricopa Interchange Pump Station

Site Plan

The design storm for pump station capacity was the 50-year return period event. The design was checked against the 2-year and 10-year storms to insure that minimum performance levels were met.

B. Pumps and Drives

The number of units was determined on the basis of the minimum (2-year and 10-year) and maximum (50-year) flow distributions. At least two units were required to handle the 50-year flow. A standby unit was also provided.

The types of pumps were determined on the basis of performance requirements such as discharge, head, velocity, type of water and specified speed. Vertical, wet well, mixed-flow pumps were preferred. Operating speeds of the pumps were to be as low as possible, never exceeding 1800 RPM.

Drive engines were required to use natural gas if availability was confirmed by a letter of commitment from the utility company serving the area. Otherwise, liquid petroleum gas engines were specified. In either case, right-angle gear drives were required.

C. Sizing and Storage

In-line storage was considered when sizing the capacity of the pump stations. Storm water detention storage was also considered as a way to reduce pump station capacity. The storage and pumping rate were optimized for the most economical solution.

Pipes were sized to handle design flows adequately. Valves and gates were provided in accordance with pump operation and maintenance requirements.

D. Junction Structures

Junction structures were designed to route flows from freeway collector pipes to the pump station inlet conduits. Access to the junction structures was designed to facilitate cleaning the structures, the pump station inlet pipes, and to improve access to the pump station wet wells.

E. Wet Wells

Wet well design follows the recommendations from the Standards of the Hydraulic Institute and the pump manufacturers. These recommendations cover inflow arrangements, spacing between pumps, clearances from bottom and side walls and submergence. The wet wells are also designed to prevent vortexing as the liquid surface is pumped down to the main pump shut-off elevation.

IV. CALCULATIONS

A. Pump Calculations

Total Dynamic Head (TDH) and Net Positive Suction Head (NPSH) were determined in order to evaluate the hydraulic performance of the pump systems. Rating curves of TDH and NPSH vs. capacity were provided by pump manufacturers. Friction losses were calculated from coefficients used in the Hazen-Williams formula, $V=1.318C_H r^{0.63} s^{0.54}$. In this formula, V=flow velocity, r=hydraulic radius, s=hydraulic gradient, and C_H is a friction coefficient.

B. Pipe Calculations

Inlet and discharge conduit capacities were calculated using Manning's equation;

$$V = \frac{1.486\ r^{2/3}\ s^{1/2}}{n}$$

with V = Velocity (ft/sec)

r = Hydraulic radius

s = Hydraulic gradient

n = Manning's Friction Coefficient

n = 0.013 for concrete pipe

n = 0.014 for concrete box culverts

The volumetric flow rate can be expressed as Q=VA, with A=cross sectional area of the conduit ($ft.^2$) and Q=volumetric flow rate (ft^3/sec).

V. EVALUATION OF ALTERNATIVES

A. Life-Cycle Analysis

As part of their design work for the 43rd Avenue pump station, Harza Engineering performed a life-cycle cost analysis to identify the most cost-effective pump station configuration. This was a non-site specific study of various alternative arrangements, the results of which were applied to the three larger sites within the project area. Components studied included:

- Number of pumps
- Type of pumps
- Source and arrangement of pumping power
- Shape and configuration of the substructure

Life-cycle costs were based on a 50-year design life for the pump stations. Capital, operation and maintenance, replacement and electrical demand costs were used in the comparison. Non-economic factors considered in the analysis included

equipment reliability, ease of maintenance and safety in maintenance and operation.

1. Alternatives Considered

a. Pump Types and Numbers

The following types of pumps were considered in the life-cycle analysis:

- o Vertical column mixed flow pumps.
- o Submersible centrifugal pumps with submersible motors attached.
- o Screw pumps.

Other types of pumps were given initial consideration, but they were eliminated because of unsuitability or excessive cost. These include:

- o Submersible propeller pumps.
- o Vertical column propeller pumps.
- o Dry-pit centrifugal pumps.
- o Vertical turbine pumps.

The vertical column mixed-flow pumps were considered in 3-pump and 4-pump combinations. Submersible centrifugal pumps were considered in a 6-pump combination. Screw pumps were considered in a 5-pump combination. These combinations were set by the maximum size of individual pumps available and head requirements.

b. Station Shapes

Both circular and rectangular substructures were considered for the vertical column mixed-flow arrangements. The special requirements and numbers of pumps for the other pump types dictated the choice of a rectangular substructure to achieve good hydraulics and equalized flow to all pumps.

c. Power Sources

Alternative power sources considered for the vertical column mixed-flow pumps included direct gas engine drives, on-site power generation with two engine-generators, and utility electric power with or without standby power generation. The direct gas engine drive was not considered possible for the submersible pump arrangement, and impractical for the screw pump arrangement.

For the purposes of the life-cycle analysis all direct drive engines and engine-generators were priced as liquid petroleum gas (LPG) engines. LPG is the fuel used in most of the existing stations operated by ADOT. Natural gas was given consideration in the non-economic portion of the analysis, as was diesel fuel.

2. Conclusions

All alternatives using electrical power without standby engine-generators were dropped from further consideration because power failure during a rainfall event is too large a probability. The recommended arrangement, based on economic and non-economic factors, was as follows:

- Rectangular substructure.
- Four vertical column mixed-flow pumps.
- Direct engine drives through right angle gear reducers, using either gas or diesel engines.

B. Third Avenue, Third Street and Sixteenth Street Pump Stations

1. Initial Evaluation

The evaluation of the Third Avenue, Third Street and Sixteenth Street pump station alternatives was accomplished by analyzing the Third Avenue Station and extending the results to the other

two. The Third Avenue site was selected for this analysis because preliminary storm sewer plans were available for this site. It is reasonable to extend the Third Avenue results because the hydraulic and hydrologic design requirements are similar at all three sites. The results of the life-cycle analysis were considered in this evaluation as well.

The hydraulic analysis began by establishing the stage/storage relationship for each alternative. The 50-year design storm runoff was then routed through the station. For this routing, a set of pump sizes was chosen, with the third main pump designated inoperable. The design consultant for the pump stations, Harza Engineering, used a computer program based on the FHWA algorithm to develop these mass curve routings.

If the storage required for the alternative did not exceed the maximum available storage, the routing was considered successful. The lowest pumping rate per pump which produced a successful routing was further evaluated for cost effectiveness.

Four alternatives were considered in this initial evaluation. Alternative 1 included a 60-inch inlet pipe and the collector system as designed; Alternative 2 included a 96-inch inlet pipe and the collector system as designed; Alternative 3 included a 9-foot by 9-foot box culvert inlet, with the collector system as designed; Alternative 4 included a 96-inch inlet pipe and increased sizes in a portion of the collector system. Table 5-1 presents a summary of the hydraulic analysis for these alternatives.

The cost analysis of these options showed Alternative 1 has the lowest cost. Therefore, it was concluded that it would not be cost-effective to increase the size of the inlet conduit or the collector system. Later in the design process, when flows to the

stations were more clearly defined and higher than originally anticipated, it was determined that greater storage or larger capacity pumps would be required. It was decided to increase the storage capacity and the 6-foot (rise) by 8-foot (span) concrete box culvert design was selected as the most economical. It also can handle the 50-year design inflow without surcharging the collector system. This design was recommended for the Third Street and Sixteenth Street stations as well, since the 8-foot by 6-foot box culvert provides enough additional storage to handle their peak inflows without re-design of the stations or selection of additional pumps.

2. Additional Evaluation

Three different main pump alternatives were evaluated for cost-effectiveness. Alternative 1 consisted of three main pumps with engine drives. Alternative 2 consisted of four main pumps with engine drives. Both alternatives 1 and 2 utilized vertical-column, mixed-flow pumps. Alternative 3 utilized four submersible pumps with two engine-generator sets.

Alternative 1 was evaluated as having the least equipment and construction cost at each of the pump stations. This alternative was then compared to an option consisting of three submersible mixed-flow pumps, driven by submersible hydraulic motors. The hydraulic motors would in turn be powered by hydraulic pumps which are driven by turbine engines.

A cost comparison of these alternatives showed possible cost savings of 5.3% at Third Avenue and a 0.3% cost increase at Third Street. Both of these figures are smaller than the assumed accuracy of the cost estimates used. In view of the small cost savings achieved and the inexperience of the manufacturer in building pump systems of the required head and capacity, the hydraulic drive/turbine engine alternative was given no further consideration.

TABLE 5-1

SUMMARY OF HYDRAULIC ANALYSIS

Parameter	Alt. 1	Alt. 2	Alt. 3	Alt. 4
Size of Main Pumps Required (cfs) ea.	66	65	63	62
Max. Available Storage (cubic feet)	24,570	28,870	32,740	34,400
Max. Required Storage (computer output) (cubic feet)	23,825	28,779	30,910	34,132

3. Recommendations

Based on the evaluations above, the Third Avenue, Third Street and Sixteenth Street pump stations each consist of three main pumps and one nuisance pump. The main pumps are vertical column mixed-flow pumps with direct LPG engine drives through right-angle gear reducers. The nuisance pump is submersible with a submersible, explosion-proof electric motor.

Inlet conduits are 8-foot by 6-foot concrete box culverts at all three stations. Discharge conduits are 60-inch circular reinforced concrete pipe at Third Street and Sixteenth Street. The discharge conduit at the Third Avenue pump station is a 72-inch diameter reinforced concrete pipe.

4. Auxiliary Systems

a. Pump Controls

Pressure transducers sense the water level in the wet wells and bring each main pump into operation at pre-set water level control elevations. These elevations are derived from the hydraulics of the stations. An additional elevation point is set to activate a high water level alarm. Control elevations are shown in Figures 5-5, 5-6, and 5-7.

The pressure transducers are connected to a programmed pump controller, which activates the proper control circuit as each level is reached. The pump controller automatically changes the sequence of operation of the pumps each time all the pumps are stopped for 6 or more hours. The operation sequence will rotate as follows: 1-2-3, 3-1-2, 2-3-1, 1-2-3, etc. This rotation will tend to equalize wear on the pumps, drives and engines.

The high water level alarm triggers an audio alert and energizes a red light on the station roof. Alarm signals are also transmitted to the ADOT Computer Monitoring System. An additional alarm for engine starting failure is connected to the ADOT Computer Monitoring System. Failure of a pump engine to start is a critical condition and must be corrected, if possible, in the event of a major storm.

b. Fire and Hazardous Gas Systems

Fire protection for each station is provided by a foam-type fire extinguishing system with thermostatic heat detectors, an alarm signal generator, a foam concentrate tank, proportioning controls, and a fire pump and motor with associated piping and valves. The system is automatic with a manual operation capability. The system is sized to provide at least three foam injection periods of ten minutes each to the pump pit water surface. In addition, all electrical elements of the fire protection system are explosion-proof.

The hazardous gas detection system consists of monitoring elements located throughout each station. The monitoring elements activate a signal in the Motor Control Center upon the detection of flammable gas. This signal initiates the disconnection of power to the affected area (outside the electrical room), activates a special ventilation system to clear the gases from the station, and triggers an alarm.

The electrical room is isolated and sealed from the hazardous areas. It has a separate ventilation system. Portable external pumps must be used to remove the hazardous liquids from the pump pit.

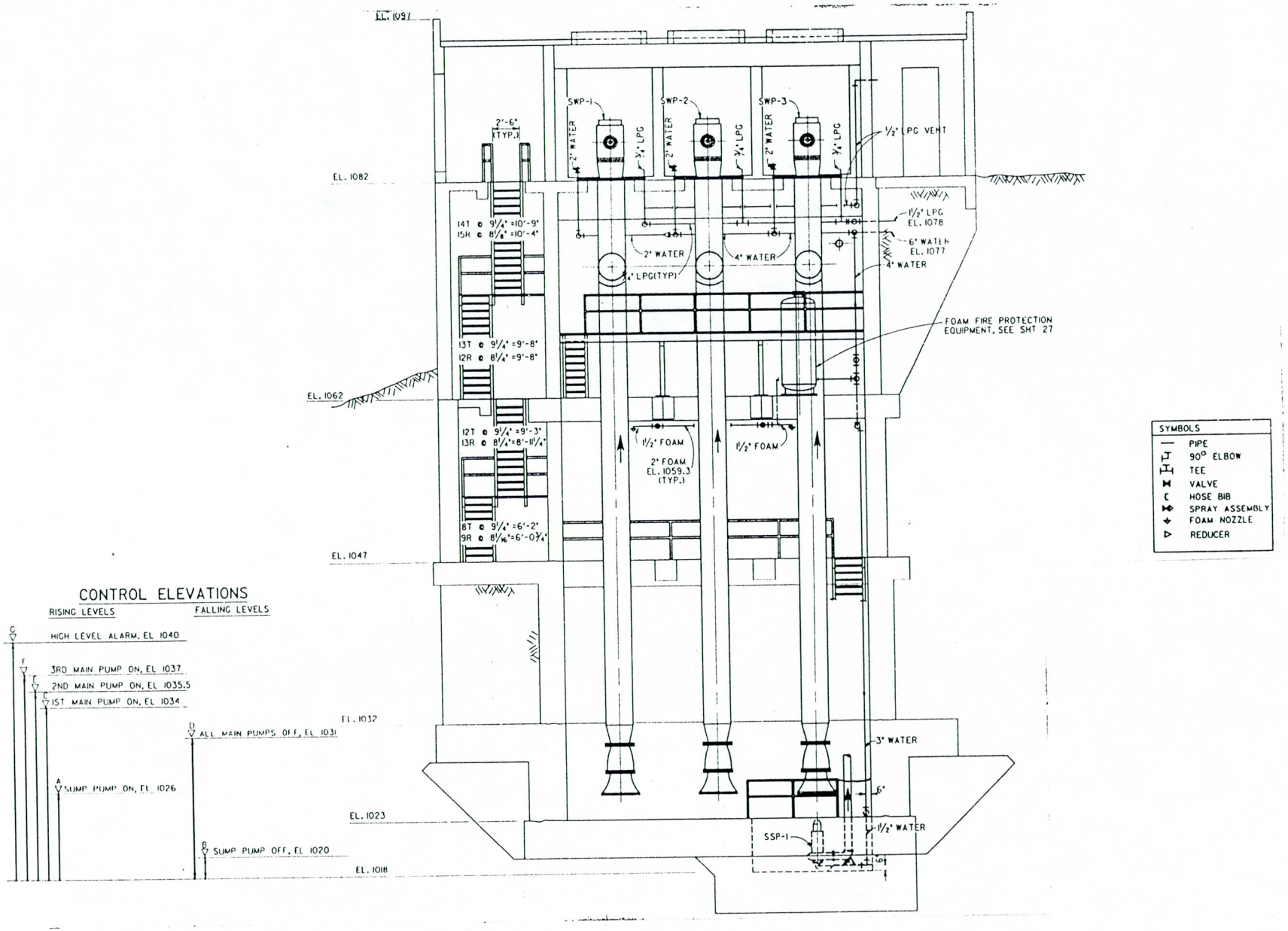


Figure 5-5

3rd Avenue Pump Station

Vertical Section

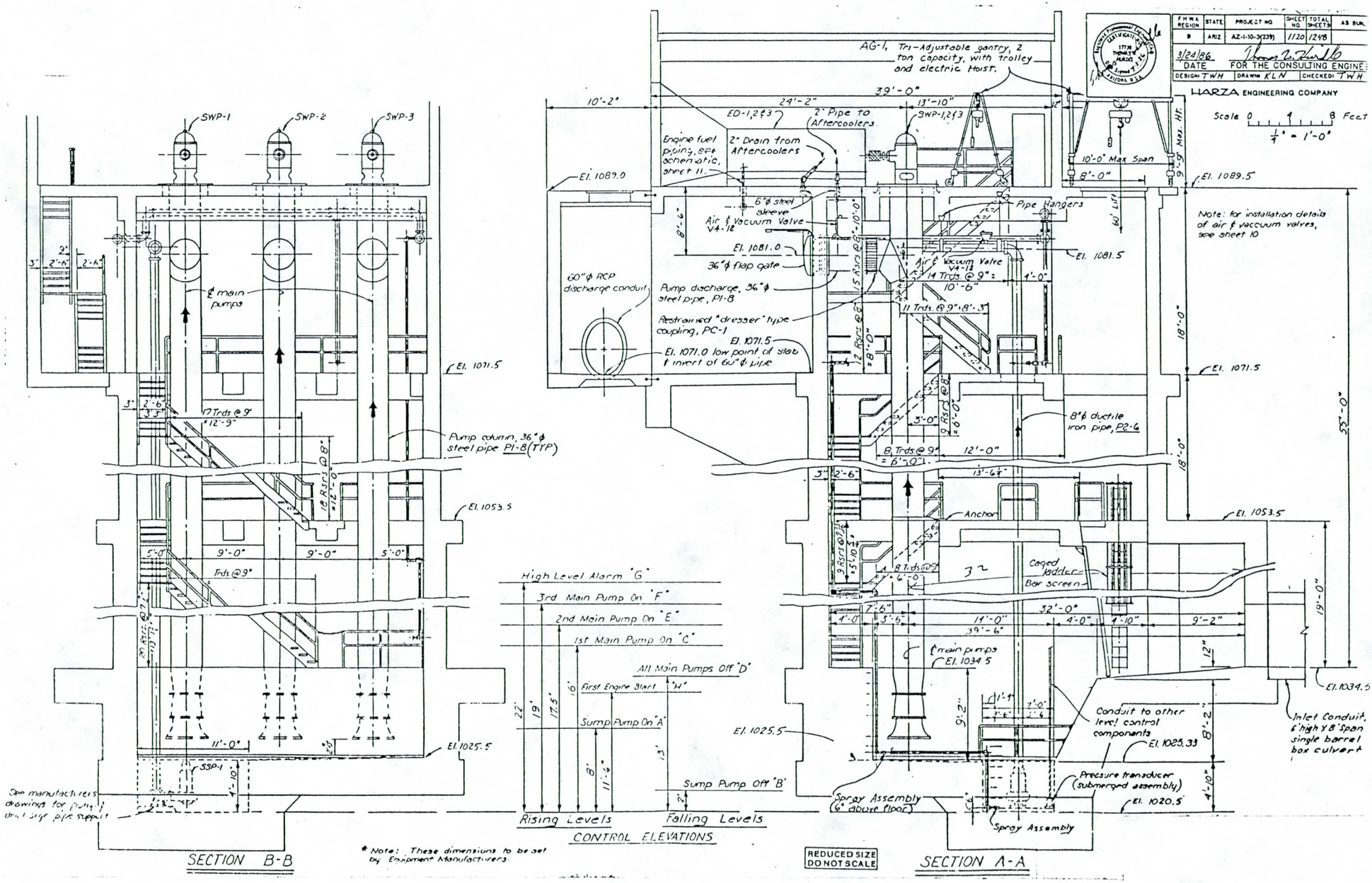


Figure 5-6

3rd Street Pump Station

Vertical Sections

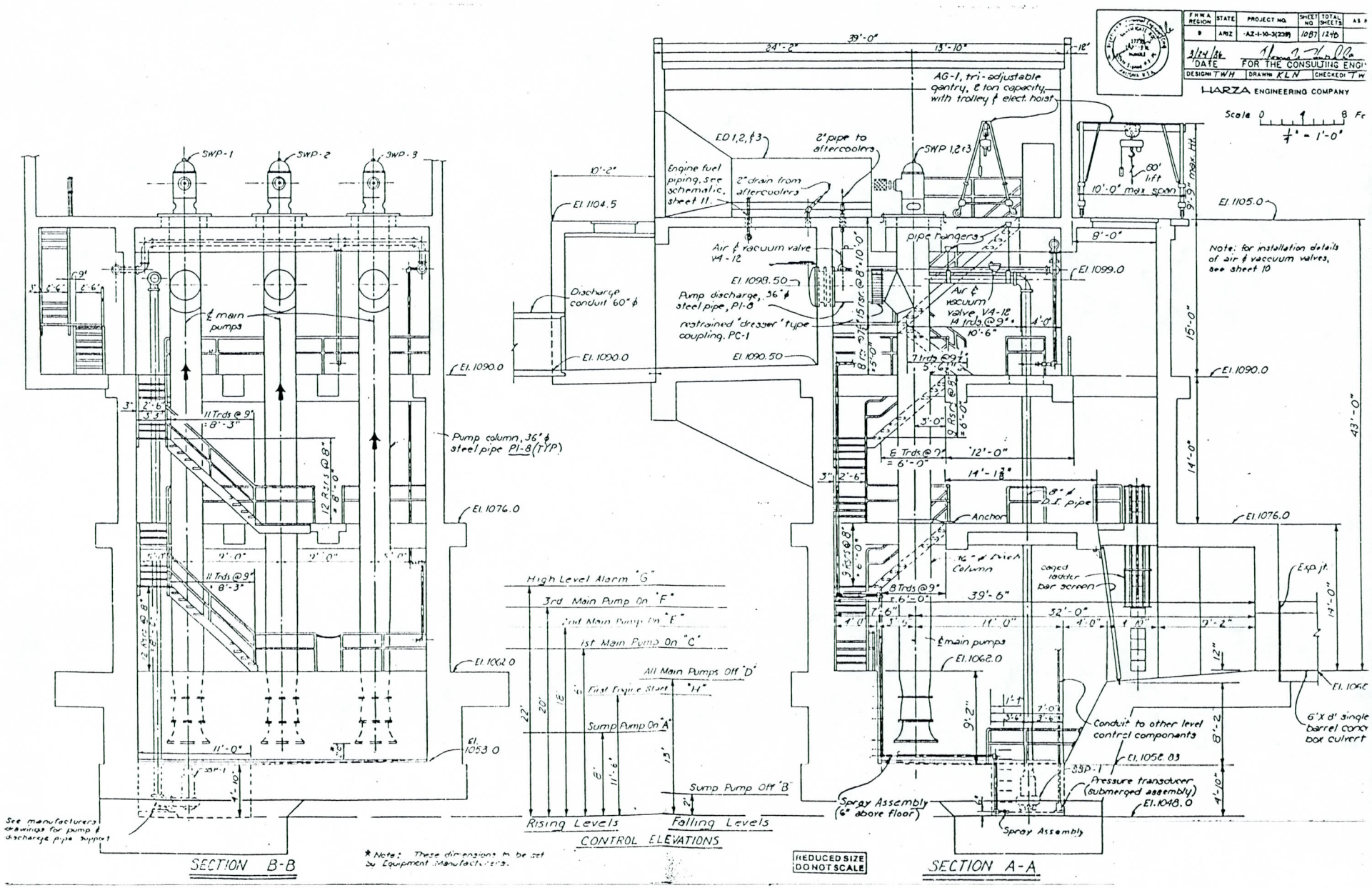


Figure 5-7

16th Street Pump Station

Vertical Sections

The following system interlocks are provided at each pump station:

- In case of hazardous gas detection, the pump pit ventilation system will be activated and all non-explosion-proof electrical circuits outside the electrical room will be disconnected. The hazardous gas alarm will be activated.

- In case of fire detection, the fire alarm will be activated; the fire extinguishing system will be activated; all other electrical systems outside the electrical room will be disconnected.

- In case of either hazardous gas or fire detection the main pumps and the auxiliary pump will be stopped, if running, or will be prevented from starting, if not running.

c. **Electrical System**

Each station receives electrical power from a single utility source. Battery systems are used for main pump starting, pump control, and emergency requirements. Utility power supply is 480 volts, three phase, at 60 Hertz.

Power distribution is through a central Motor Control Center (MCC) in each station. The MCC houses breakers, contactors, and combination motor starters for fans, the nuisance pump, and other miscellaneous motors and equipment. The MCC also includes a 3 phase, 120/208 volt lighting transformer and a lighting panel board.

General indoor lighting is of the fluorescent type, providing approximately 30 foot-candles of intensity. Interior lighting is explosion-proof with the exception of the electrical and engine rooms. Indoor emergency lights are explosion-proof, powered by the self-contained battery system.

Wall mounted sodium vapor lights provided outdoor illumination. These lights are controlled by photo-electric cells. Outdoor lights are located to illuminate the areas adjacent to building entrances and other critical areas.

Ground rods are driven into the soil near each pump station building, and connected to a bare copper ground wire loop around each building. The ground loop is connected to the MCC ground bus, motor frames, building reinforcement and non-current carrying metal parts of electrical equipment.

Control panels for the pumps are located in the electrical room of each station. Pump engines have a manual starting option in addition to the normal automatic operation.

d. Ventilation Systems

Each pump station has three separate ventilation systems. One system ventilates the pump pit, one system ventilates the electrical room, and one system ventilates the pump drive room.

Pump pit ventilation is accomplished by an exhaust duct system with an explosion-proof axial fan in the pump pit, just below the upper floor. The system takes in air through the inlet chamber gratings, and exhausts hazardous fumes through either one of two ducts. The first exhaust duct

system is located just above the normal high water level for the nuisance pump, and the second exhaust duct system is located just above the high water level for the main pumps.

Air flow through the two systems is controlled by spark-proof shut-off dampers at each level. This system only operates when combustible gases are detected in the pump pit. Exhaust is routed through the pump drive room.

Wall mounted fans ventilate the electrical room and the battery room. One fan is mounted in each room. These fans blow outside air into the rooms to maintain a positive pressure. This system precludes the possibility of hazardous gas entering these areas from either the pump drive room or the wet well.

The pump drive room in each station is ventilated by the natural circulation of air through open louvers on three sides of this room. Two oscillating fans with manual controls provide air movement for maintenance people when there is no circulation from the wind.

e. **Screening and Grit Removal**

The screen system is composed of a bar rack of flat steel bars, inclined at about 7.5 degrees from vertical. Spaces between the bars are 1 inch wide. Sections of the screen are a maximum of 2 feet wide, and removable for access and repairs.

Materials retained by the screen will fall, or can be raked, to the inlet chamber floor when the station is emptied. After a major storm, screened material can be removed by placing it in a bucket and hoisting it through the removable grating sections at the ground surface.

Grit removal consists of cleaning up all mud, sand and small gravel, as well as other material which passed through the screen but was not pumped out during station operation. The portion of this material which is not removed manually can be flushed into the nuisance pump sump. The nuisance pump can be started manually to remove the material. Low water level in the sump will automatically stop the pump.

Hose connections are located on both sides of the screen and on the walkway above the screen for the wash-down water. These connections accept 1-1/2 inch hose. Wash-down water is provided by the city, through a service pipe with curb-stop and meter. A "backflow preventer" eliminates any possibility of siphoning pump station storm water into the City of Phoenix water system.

C. Maricopa Traffic Interchange Pump Station

Four different methods were considered for disposal of storm runoff from depressed Ramp W-N at the Maricopa Interchange. Alternative 1, gravity drainage directly to the East Tunnel, is not possible because surcharged conditions in the tunnel prevent inflow during the design runoff event. Alternatives 2 and 3 both require pumping, either to a series of dry wells or to a retention/evaporation basin. The recommended alternative consists of a pump station which removes storm water from the ramp and discharges to a large retention/evaporation basin. The retention basin drains through a storm water outlet system to a 60-inch storm drain south of the interchange.

1. Evaluation

Three alternative combinations of wet well design and pump type were evaluated on the basis of criteria given in Figure 5-1 and

5-2 of the "Manual for Highway Storm Water Pumping Stations," Vol. 1 by the FHWA. Alternative 1 consisted of a rectangular wet well and two vertical column, mixed-flow main pumps with direct engine drives. Alternative 2 consisted of a circular wet well with two vertical column, mixed-flow, engine driven main pumps. Alternative 3 consisted of two submersible pumps.

According to the criteria in the FHWA manual, all three were acceptable choices. However, Alternative 2 had the best score and was therefore the recommended alternative.

A storage optimization analysis was performed to evaluate various combinations of pump capacity and storage volume. It was found that the entire range of reasonable pumping rates is obtainable by varying the impeller size and discharge pipe diameter on the selected pump. Operational cost differences between the combinations were insignificant.

Two wet well designs were considered. The first was a 20-foot x 20-foot square wet well. The second was a 22-foot diameter circular wet well. Both provide essentially the same storage capacity. The circular wet well option was found to be the least expensive to construct, and was therefore the selected design.

2. Recommendations

Based on the evaluations above, the MTI pump station consists of a circular, 22-foot diameter wet well, two vertical column mixed-flow main pumps and a submersible, solids-handling sump pump. No backup main pump is required because Ramp W-N is not a primary route. Wet well floor plan and sections are shown in Figure 5-8.

The main pumps are driven by liquid petroleum gas fueled engines through right angle gear drives. The sump pump is rated for 460-volt, 3 phase power delivered through a flexible, waterproof cable.

The inlet conduit is a 36-inch diameter reinforced concrete pipe. The discharge pipe is a 30-inch diameter reinforced concrete pipe which conveys flow to the retention basin.

3. **Auxiliary Systems**

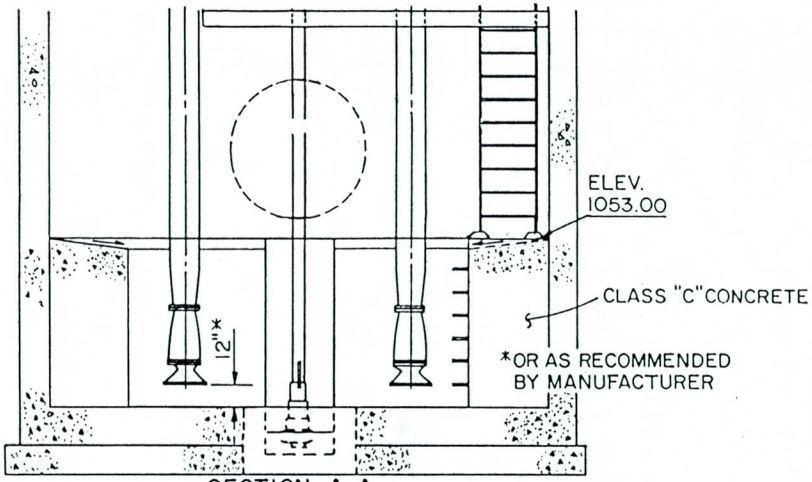
a. **Pump Controls**

The maximum allowable water level is set two feet below the elevation of the lowest ramp inlet grating. Pump settings were determined by storage requirements and minimum pump cycle times.

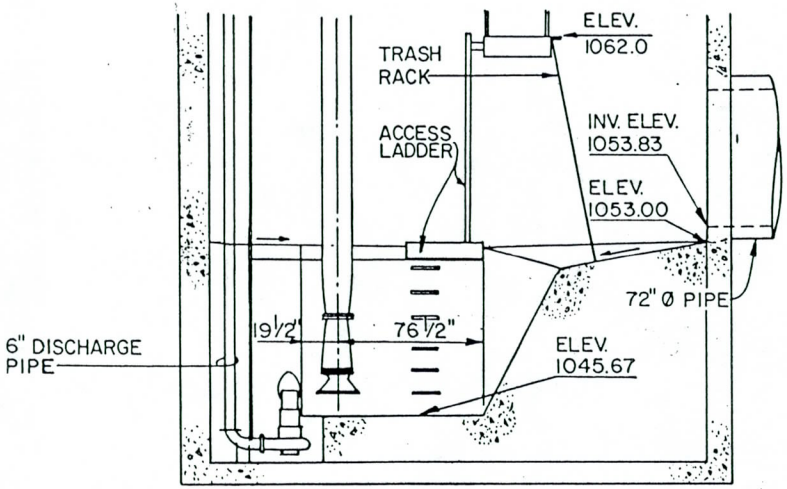
Main pumps are activated by a float-type mercury switch system, with one float for each elevation at which a pump is to be brought on-line. The floats are connected to a pump sequencing control panel which activates the proper circuit as each level is reached. The sequencing control panel alternates the main pump operating sequence in order to equalize wear on the pumps and minimize cycle times for the engines. An alarm light on the station roof is energized when the water level reaches the pre-set high-water elevation.

b. **Fire and Hazardous Gas Systems**

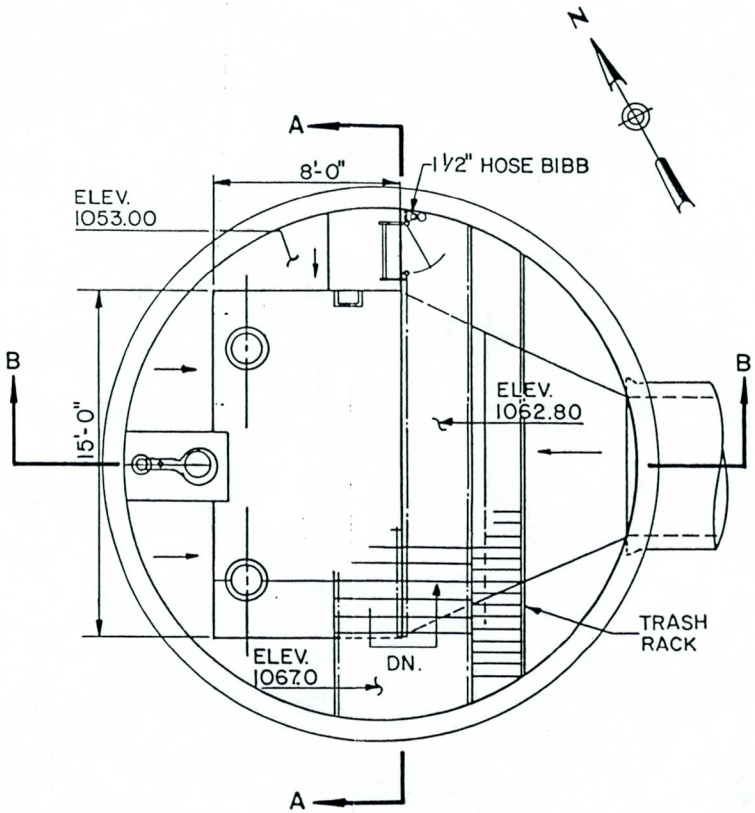
Fire protection is provided by five portable fire extinguishers, located at various points in the pump station. The fire extinguishers are carbon dioxide, fifteen pound capacity, rated for 10 B:C class fires.



SECTION-A-A



SECTION B-B



WET WELL-FLOOR PLAN

Figure 5-8
Maricopa Interchange Pump Station
Floor Plan and Sections

The hazardous gas detection system consists of two monitoring elements. These elements trigger an alarm and the roof mounted exhaust fan upon the presence of flammable gas. The sensors are factory calibrated at a 1000 ppm concentration level of propane gas.

Upon detection of flammable gases by the sensors, portable external pumps must be used to remove the hazardous liquids from the pump pit.

c. **Electrical System**

Electric power is provided by a single utility source, with battery systems for main pump starting, pump control, and for emergency lighting requirements. The utility power supply is 480 volts, three phase, 60 Hertz. Power distribution is through NEMA Type I enclosures, providing power to lighting transformers, the sump pump, and other incidental equipment.

Fluorescent light fixtures are used for general indoor lighting. These lights provide approximately 30 foot-candles of intensity in the station. Indoor emergency lights are supplied by an individual battery system.

Outdoor lighting is provided by wall mounted, sodium vapor units with photo-electric controls. These lights are mounted to provide lighting at building entrances and other critical points.

Battery systems provide power for the pump sequencing control panel and for cranking power for each main pump drive engine. Static battery chargers maintain all batteries in a fully charged condition.

d. Ventilation System

There is one ventilation system. The pump drive room and pump pit are ventilated by a roof mounted exhaust fan. Ductwork and dampers are located in the pump pit and drive room areas. The duct system is designed to take in air from the pump pit inlet chamber gratings and pump drive room air supply louvers. The exhaust fan is controlled by a thermostat or by a signal from the hazardous gas detection system.

e. Screening and Grit Removal

A trash rack, consisting of a bar screen with 1-1/2 inch spaces, is provided at the entrance to the wet well. Screened material can be hoisted to the pump house through an access hatch in the pump house floor.

Grit and fine material can be washed into the nuisance pump sump. Wash-down water is provided by a hose connection located at the lower stair landing level. The sump pump will discharge the material into the outlet junction box, which drains to the detention area through the outlet conduit.

Access to the outlet junction box is provided by a 3′ x 3′ covered opening. The junction box is flushed by high pressure potable water sprays. These sprays are automatically started at a pre-set water level in the sump, and continues for a pre-set time.

VI. PUMP STATION SUMMARY

Table 5-2 summarizes the I-10 Inner Loop pump station hydraulic design.

TABLE 5-2

PUMP STATION SUMMARY

Location:	THIRD AVE.	THIRD ST.	SIXTEENTH ST.	MARICOPA INTERCHANGE
Designer:	Harza Engineering	Harza Engineering	Harza Engineering	John Carollo Engineers
50-Year Peak Inflow:	165.9 cfs	185 cfs	181.8 cfs	20.8 cfs
Pumping Rates: Main Pumps:	3 Pumps @29,200 gpm 48.9 ft. Total Head	3 Pumps @35,000 gpm 49.8 ft. Total Head	3 Pumps @34,800 gpm 39.7 ft. Total Head	2 Pumps @4,000 gpm 52 Ft. Total Head
Nuisance Pumps:	1290 gpm 66.5 ft. Total Head	1300 gpm 66 ft. Total Head	1370 gpm 57 ft. Total Head	800 gpm 58 ft. Total Head
Pump Types: Main:	Vertical Column Mixed flow	Vertical Column Mixed flow	Vertical Column Mixed flow	Vertical Column Mixed flow
Nuisance:	Submersible	Submersible	Submersible	Submersible
Inlet Conduit:	8′ x 6′ Box Culvert	8′ x 6′ Box Culvert	8′ x 6′ Box Culvert	36" Diameter RCP
Discharge Conduit:	72" Diameter RCP	60" Diameter RCP	60" Diameter RCP	30" Diameter RCP
Discharge Point:	Third Avenue Drop Shaft (To North Tunnel)	Third Street Drop Shaft (To North Tunnel)	Storm Water Interceptor	Retention Basin

CHAPTER 6 - CONVEYANCE SYSTEM DESIGN

SYNOPSIS

This chapter summarizes the design of the Storm Water Interceptor (SWI) and the outfall tunnels. Initial recommendations, design refinements and final configurations are described.

Chapter 6 also includes a summary of the transient wave analysis which was performed to quantify the effects of surge phenomena in the tunnel system. The transient wave analysis resulted in recommendations for surge relief and air release facilities.

I. INITIAL RECOMMENDATIONS

As part of their "Hydrology and Hydraulic Design Concepts" report, Arthur Beard Engineers, Inc. (ABE) evaluated several different alternatives for a system which would convey storm water from the Inner Loop to the Salt River. The inital evaluation performed by ABE was expanded in the Value Engineering (VE) report prepared by HNTB. These two reports resulted in recommendations for the alignment and size of the SWI and tunnel system. The final design of the conveyance system components is described later in this chapter.

A. Alignment

1. ABE Report

Four different alignments were considered by ABE for the outfall system. In Alternative 1, all storm drainage collected along the Inner Loop was to be conveyed to the Salt River by box culverts located at 11th Street, 1st Avenue and the northbound frontage road of I-17. Alternative 2 utilized these same outfalls, plus one following the north-south leg of I-10. Alternative 3 used a single tunnel, located under 15th Avenue, to convey all storm

flow to the river. Alternative 4 involved construction of two tunnels, one under 15th Avenue and one under Central Avenue.

All four alternatives utilized an interceptor system along the north edge of the Inner Loop right-of-way to collect storm water and carry it to the outfall system. Sizing of the interceptor was dependent upon the outfall alternative under consideration. Drainage collection along the north-south leg of I-10 was not included in ABE's scope of work for this report.

The alternatives were ranked on the following criteria:

- Installed costs
- Operation and maintenance costs
- Right-of-way requirements
- Time of completion/traffic control
- Disposal of materials
- Utility conflicts
- Environmental effects
- Joint-use potential

Alternatives 3 and 4 were very close in the overall ranking. These two alternatives were directly compared on the basis of construction cost, operation and maintenance costs, additional right-of-way requirements, and joint-use potential. Based on this final evaluation, Alternative 4 was the recommended alternative in the ABE report.

2. VE Report

The Value Engineering Report prepared by HNTB expanded on the study done by ABE. Because of the reduction in peak flows which resulted from refinements in the hydrology portion of the VE Report, the culverts at 1st Avenue and 11th Street were dropped from further consideration. While the ABE Report only considered using the 15th Avenue tunnel in conjunction with the Central

Avenue tunnel, the VE Report considered various combinations of outfall alignments, including the proposed 21st Street tunnel.

The outfall alignments were combined with various east-west conveyance alignments, off-site collection systems and storage systems to produce a total of 26 different alternatives for the complete drainage system. These alternatives were evaluated using the criteria established in the ABE Report, as well as impact on historic landmarks and flood mitigation for properties south of the Inner Loop.

Three of the 26 alternatives were carried forward for further analysis. In Alternative 1, all off-site drainage was intercepted along the north edge of the Inner Loop right-of-way and conveyed to a single tunnel under Central Avenue. Alternative 2 carried existing storm sewers across the freeway alignment in inverted siphons. All surface flow and proposed storm sewers would be conveyed along the north edge of the freeway corridor to a single tunnel under Central Avenue. Alternative 3 consisted of inverted siphons which would carry the total 50-year surface flow and existing storm sewers across the freeway. Proposed storm sewer flow would be intercepted at the north edge of the corridor and conveyed to a tunnel at Central Avenue.

All three of these alternatives included the 21st Street tunnel (the East Tunnel), although the systems were not interconnected. The off-site collection system is described in Chapter 4 of this report, and will not be discussed here.

A Tunnel Feasibility Study was completed after the alternative analysis. It recommended a shift in the tunnel alignment from Central Avenue to 2nd Street to avoid the many high-rise buildings in the area. Modifications to the freeway alignment between 7th and Central Avenues resulted in very tight clearances at the Kenilworth School and the Ellis-Shackelford House. To

avoid potential damage to these historic sites, it was determined that the east-west collector between 7th Avenue and 2nd Street should be of tunneled rather than cut-and-cover construction.

The final analysis of the three alternatives, with the modifications recommended by the Tunnel Feasibility Study, was conducted as a series of independent studies. These studies included an analysis of the use of a storm other than the 50-year event as the design storm, the effects of modifications to the conveyance and outfall systems, a determination of flood damages to private property, and the application of hydraulic computer models to the complete drainage system.

Based on the final analysis, the alignment suggested in the VE Report followed the modified Alternative 1. It consisted of the following elements:

- An east-west conveyance system along the north edge of the Inner Loop corridor, intercepting all surface and storm sewer flow from 13th Avenue to 19th Street and conveying the storm flow to a Central Corridor tunnel. The section between 7th Avenue and 2nd Street should be tunneled.

- A Central Corridor tunnel which joins the east-west conveyance system at 2nd Street, conducting storm water to the Salt River. The tunnel alignment follows 2nd Street south to Sherman Street, where it shifts to Central Avenue, then south along Central to the Salt River.

- An outfall along the east side of the north-south leg of the Inner Loop corridor, conveying surface flow from 20th Street, flow from the proposed 20th Street storm sewer, and all flow reaching the north-south freeway leg south to the Salt River.

B. Size

Initial sizing of the conveyance system followed standard hydraulic practice using Manning's formula and the following guidelines:

- Manning's "n" = 0.012 for concrete pipe and 0.015 for box culverts and tunnels.
- Maximum velocity in pipes and culverts of 15 fps.
- Tunnel slope of 0.133%.
- Peak flows as described in previous sections of this report.

In order to determine the effects of flow routing on pipe size, the conveyance system was simulated with the SWMM3 hydraulic computer model. Two different portions of the program were used for modeling, the TRANSPORT block and the EXTRAN block.

The TRANSPORT section includes a subroutine which automatically increases pipe sizes to prevent surcharging. This subroutine was used to confirm the pipe sizes arrived at in the preliminary analysis. However, review of the results of the TRANSPORT modeling indicated that the tunnels were oversized at the upstream ends.

These results revealed two shortcomings of the TRANSPORT model. First, the program assumes free discharge conditions in all gravity conduits. Therefore it is not capable of accurately modeling the I-10 drainage tunnels, which operate under surcharged conditions. Second, because free discharge is assumed the program does not automatically allow for backwater effects or line storage. These were considered to be serious drawbacks to the model for this application.

The EXTRAN block was run to overcome the limitations of the TRANSPORT model. EXTRAN is capable of modeling surcharge and backwater effects, and line storage. The tunnel discharge was

modeled as an outfall with a tide gate in order to simulate its operation as an inverted siphon. The primary variable used in the EXTRAN runs was the tunnel diameter. Other conduit sizes were left as determined by the TRANSPORT model.

The results of the EXTRAN modeling showed that a 21-foot diameter tunnel in the 2nd Street system would prevent street flooding during the 50-year event, provided the Salt River was initially dry. With the river stage at the 10-year flood level, a 25-foot diameter tunnel would be required. For the 21st Street system, the tunnel diameters were 19.5 feet with a dry river, or 20.5 feet with the 10-year river stage.

Tables 6-1 and 6-2 summarize the initial conduit sizes obtained from the computer modeling of the I-10 Inner Loop drainage system.

Detailed analysis of the EXTRAN simulations showed that the street flooding was caused by a reflected wave traveling upstream while the tunnel fills. As the tunnel size increased, the fill time lengthened and the reflected wave was delayed past the peaks of the inflow hydrographs, which reduced the street flooding.

The EXTRAN model has some characteristics which cast some doubts on the reliability of the results. In particular, continuity errors were excessive and the model was very sensitive to minor changes in system configuration. Also, while the EXTRAN model was able to indicate the presence of a reflected wave, it was not able to predict the magnitude of the surge pressures. Although EXTRAN is probably the best generally available model for simulating complex pipe systems, its applicability to the I-10 drainage tunnels is limited.

Dr. Charles C.S. Song has developed a computer model to analyze the tunnel pressurization process and the pressure surges common to these types of systems. He was retained to conduct detailed

TABLE 6-1

2ND STREET SYSTEM
INITIAL CONDUIT SIZES

Street	Size	Slope (ft/lf)
19th Street	48"	0.00185
18th Street	60"	0.00186
17th Street	72"	0.00205
16th Street	90"	0.00214
15th Street	96"	0.00194
14th Street	96"	0.00227
13th Street	102"	0.00271
12th Street	12′ x 10′	0.00152
11th Street	12′ x 11′	0.00130
10th Street	12′ x 11′	0.00147
9th Street	12′ x 11′	0.00171
7th Street	(2) 10′ x 10′	0.00156
5th Street	(2) 12′ x 10′	0.00180
3rd Street	(2) 12′ x 10′	0.00180
2nd Street		
13th Avenue	84"	0.0009
11th Avenue	102"	0.0009
9th Avenue	10′ x 10′	0.0009
7th Avenue	10′ x 10′	0.0010
7th Avenue D.S.	14′	0.00245
5th Avenue D.S.	14′	0.00245

2ND STREET SYSTEM
INITIAL CONDUIT SIZES
(CONTINUED)

Street	Size	Slope (ft/lf)
1st Street D.S.	14′	0.00245
2nd Street D.S.	21′	0.00133
2nd/Van Buren	21′	0.00133
2nd/Grant	21′	0.00133
Central/Watkins	21′	0.00133
Riser Structure		

TABLE 6-2

21ST STREET SYSTEM
INITIAL CONDUIT SIZES

Street	Size	Slope (ft/lf)
20th/Belleview	96"	0.00150
21st/Belleview	(2) 12′ x 12′	0.00140
Moreland	(2) 12′ x 12′	0.00140
Portland	(2) 12′ x 12′	0.00140
Roosevelt	(2) 12′ x 12′	0.00150
Garfield	(2) 12′ x 12′	0.00150
McKinley	(2) 12′ x 12′	0.00150
Pierce	(2) 12′ x 12′	0.00150
Fillmore	(2) 12′ x 12′	0.00160
Taylor	(2) 12′ x 12′	0.00195
Polk	(2) 12′ x 12′	0.00300
Van Buren	21′	0.00133
Adams D.S.	21′	0.00133
Washington D.S.	21′	0.00133
Madison D.S.	21′	0.00133
Buckeye D.S.	21′	0.00133
Mohave D.S.	21′	0.00133
Riser Structure		

modeling of the I-10 tunnels to verify the design of the drainage system. His results are summarized in a later section of this chapter.

II. DESIGN REFINEMENTS

After the VE study was completed, several additional studies were performed to refine the initial design of the east-west collector system and the tunnel alignments. The studies analyzed construction alternatives for cost-effectiveness, impact on traffic, and hydraulic performance. These studies resulted in several modifications to the initial design.

A. Analysis of East Leg

The east leg of the east-west collector system (the Storm Water Interceptor) runs from the 2nd Street junction structure to 19th Street. The portion of the east leg from 2nd Street to 10th Street was analyzed to determine if bored tunnel construction would be more cost-effective than the cut-and-cover box culverts originally planned.

Hydraulic calculations for the Storm Water Interceptor indicated a need for a double 10′ x 10′ box culvert from 2nd Street to 7th Street, a 12′ x 10′ box culvert from 7th Street to 9th Street and a 10′ x 10′ box culvert from 9th Street to 12th Street. The tunneled alternative consisted of a 14-foot diameter tunnel from 2nd Street to 10th Street.

The tunnel alternative also included a collector system required to convey storm flow to the tunnel. The box culvert alternative would not need the collector system, since the conduits are shallower and direct inlets could be used.

In addition to being more efficient hydraulically, the tunnel alternative was less expensive and had minimal negative impact on traffic during construction. Therefore, the North Tunnel was extended to 10th Street, with a collector system conveying water into the tunnel through drop shafts at 3rd Street and 7th Street.

B. Analysis of 21st Street System

The 21st Street storm drain extends from the east outfall at the Salt River near 20th Street and University Drive northward through a curve-tangent-curve alignment to 21st Street north of the Maricopa Freeway. From there it runs north along 21st Street to Moreland Street, where the Storm Water Interceptor continues to the northwest.

1. Alternatives

Five combinations of bored tunnel and box culvert were considered for this alignment. The tunneled portion of the alternatives was 21 feet in diameter, at a nominal depth of 35 feet from the ground surface to the tunnel crown. The box culvert portion was 12′ x 12′ box culverts of reinforced concrete, constructed with conventional open-cut methods. Depths ranged from 10 to 33 feet from the ground surface to the top of the culvert.

The number of box culverts required to conduct runoff from the 50-year design storm was calculated using the SCS TR-20 hydrology program. Two culverts were required north of Washington Street, three culverts from Washington Street to Pima Street, and four culverts from Pima Street to Mohave Street where the 21-foot diameter outfall tunnel would extend to the outfall in all alternatives.

Each alternative included the outfall to the Salt River and a drop structure connecting the box culverts to the tunnel at its

upstream end. Other than those common features, the alternatives are described as follows:

Alternative 1 - Box culverts extending from 22nd Street to Mohave Street, and bored tunnel from Mohave Street to the outfall.

Alternative 2 - Box culverts extending from 22nd Street to Washington Street, and bored tunnel from Washington Street to the outfall.

Alternative 3 - Box culverts extending from 22nd Street to a point 200 feet north of Van Buren Street, and bored tunnel extending from that point to the outfall.

Alternative 4 - Box culverts extending from 22nd Street to Fillmore Street and bored tunnel from Fillmore Street to the outfall. This was the recommended alignment in the VE Report.

Alternative 5 - Box culverts extending from 22nd Street to a point 120 feet south of Moreland Street, and bored tunnel from that point south to the outfall.

2. Evaluation

The evaluation of the five combinations was based on cost. Tunnel costs included costs of tunnel construction, access structures and protection of existing structures and utilities from ground movement induced by tunneling operations. Drop shafts required to conduct storm runoff into the tunnel were also included.

Box culvert costs were based on the use of 12′ x 12′ reinforced cast-in-place or precast concrete, constructed by conventional open-trench methods. Costs included the conduit, excavation, backfill, and manholes required to convey runoff into the culverts.

Alternative 5, construction of the 21-foot diameter storm tunnel from the Salt River outfall to Moreland Street, was recommended. This was the least expensive alternative. It also had the advantages of minimizing scheduling problems and avoiding traffic congestion, air and noise pollution, and safety hazards associated with conventional open-cut construction.

III. TRANSIENT WAVE ANALYSIS

A. Transient Wave Phenomenon

As described in earlier sections of this report, the Inner Loop drainage system includes two main tunnels which run parallel to each other from the Inner Loop to the Salt River. The Salt River is usually dry but will occasionally carry flood flows of various magnitudes. The inverts of the drainage tunnels at the exits are approximately 30 feet below the river bed. This means, at the beginning of a storm event, the tunnels may or may not be empty depending on the river water level. During a storm condition, discharge to the river is possible only after a significant portion of the system is pressurized. That is, for every significant storm, the system will undergo a pressurization process during which a large pressure head may be generated.

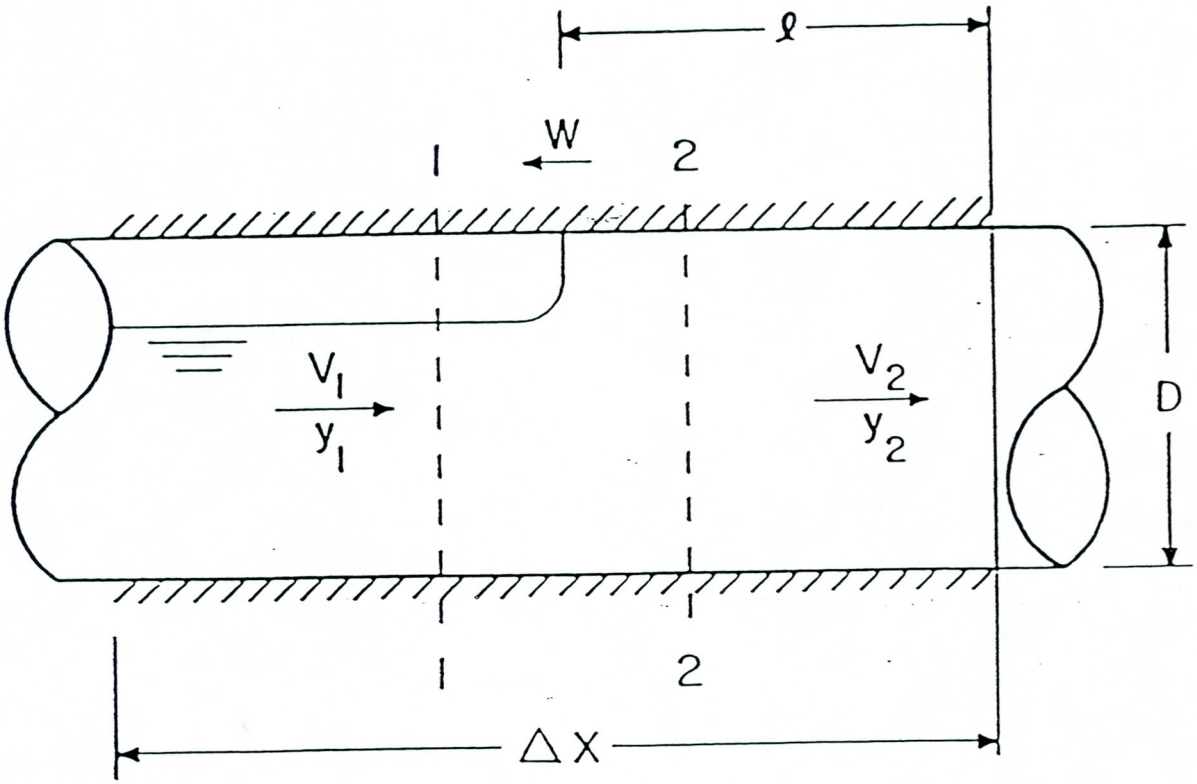
It has been demonstrated in previous works that the most critical period for the design of a storm sewer is the period of pressurization. The spreading of the pressurized zone is usually accomplished by one or more moving positive surges (interfaces). These positive surges may move in either an upstream or

downstream direction and appear as a moving hydraulic jump. These moving jumps may collide with a structure or each other and create large transient pressures. They may also cause temporary but large backflows which can produce flooding problems.

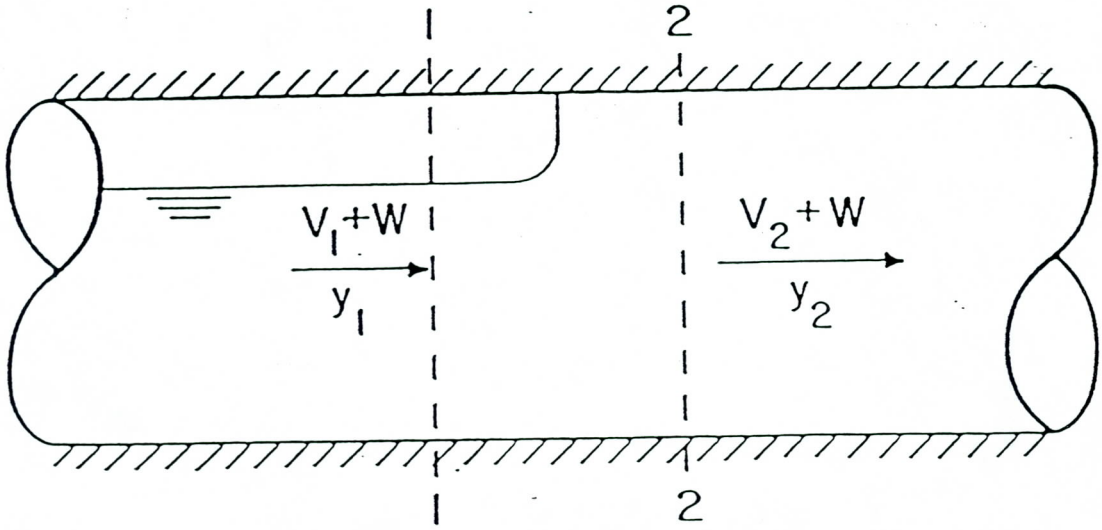
Accurate modeling of the flow during the pressurization period requires the accurate and continuous calculation and accounting of the moving interfaces. The only model known to handle this is the Mixed-flow Hydraulic Transient Model developed at the St. Anthony Falls Hydraulic Laboratory by Dr. Charles C.S. Song.

The way that the model computes the positive interface is briefly described as follows. A positive interface moving upstream, usually as a result of restricted outflow, is sketched in Figure 6-1. A positive interface moving downstream, usually as a result of too large an inflow, is sketched in Figure 6-2. In either case, there is a shock wave across which the pressure and the velocity changes abruptly. There are five variables (the wave speed W and the velocity and head at each side of the shock, V_1, Y_1, V_2, Y_2) to be computed at the interface. According to the x-t diagram shown in Figure 6-3 there are three characteristic equations (C_1+, C_1-, and C_2-) that have physical meaning and can be used for the computations. In addition, there are two shock boundary conditions (continuity and momentum conditions) that can be used. Thus, there are five equations that can be used to solve for the five unknowns.

Dr. Song's model was applied to the Inner Loop drainage system throughout the development of the detailed design. The following sections summarize the results of Dr. Song's work and the effects of some of the changes in the system configuration. Also summarized are Dr. Song's recommendations for surge protection facilities, which were carried forward into the final design. Revisions to these facilities proposed by the design consultants were reviewed by Dr. Song and often were checked by simulating the revisions with the computer model.

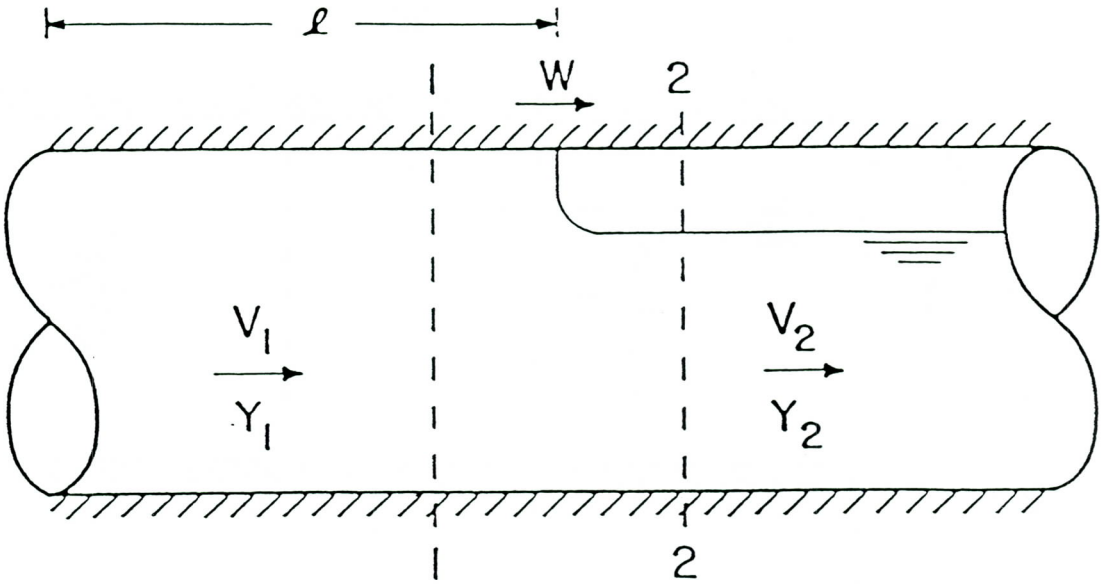


(a) POSITIVE PRESSURIZATION WAVE MOVING UPSTREAM

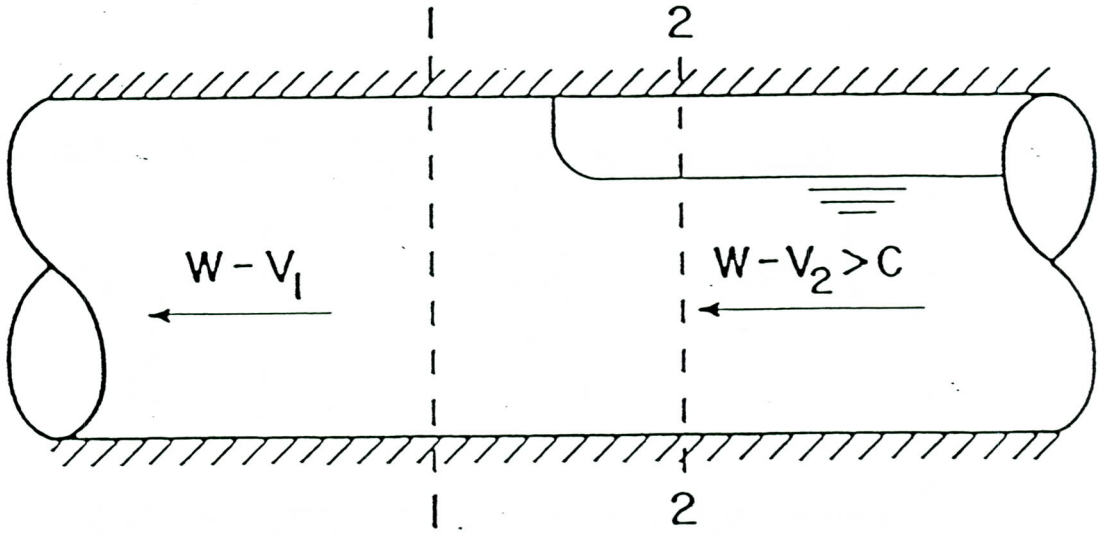


(b) EQUIVALENT STATIONARY INTERNAL HYDRAULIC JUMP

Fig. 6-1 Upstream moving pressurization interface (Positive wave).



(a) PRESSURIZATION WAVE MOVING DOWNSTREAM



(b) EQUIVALENT STATIONARY INTERNAL HYDRAULIC JUMP

Fig. 6-2 Downstream moving pressurization interface (Positive wave).

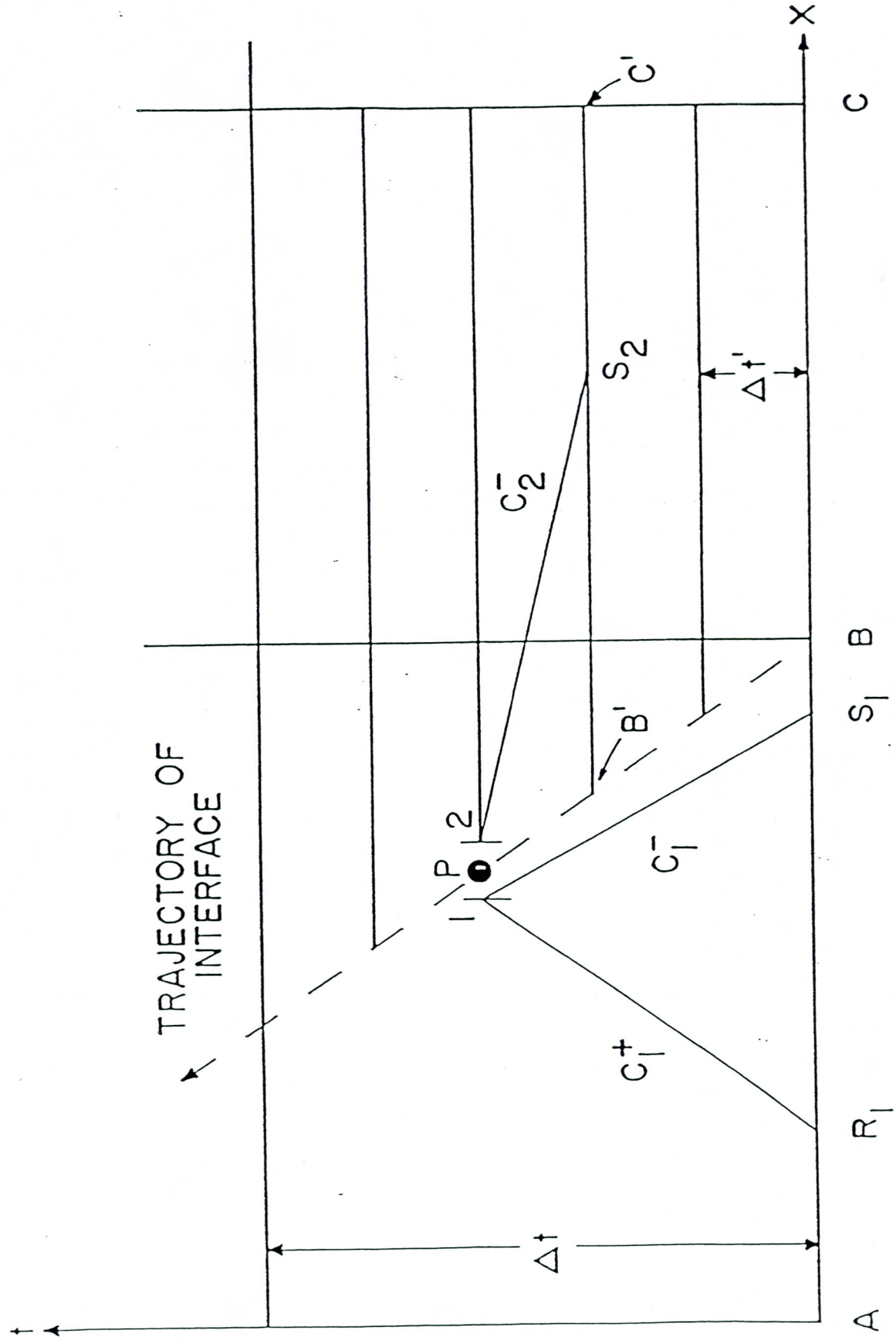


Fig. 6.3 x-t diagram for pressurization interface.

Input hydrographs for all of Dr. Song's modeling work were provided by HNTB, and were based on the output from the TR-20 hydrology model previously discussed. Revisions to these hydrographs were issued periodically, as dictated by modifications to the hydrology and changes in drop shaft locations and design flows.

B. Original System Modeling

Initially, it was assumed that the East and West Tunnel drainage systems were hydraulically independent.

The separate 2nd Street drainage system had a T-shaped configuration. It consisted of the east and west branches and the tunnel that runs from the junction at 2nd Street southward to the river. The entire east branch was at a relatively higher elevation and dropped about 42 feet before it joins the tunnel. According to the mathematical modeling work conducted by HNTB using EXTRAN, it appeared that the east branch would not pressurize under the design storm runoff condition. For this reason, the east branch was initially not modeled in detail but the outflow from the east branch as computed by EXTRAN was used as an inflow at the junction. The east branch was later included in the model because the part of this branch from 2nd to 10th Streets was lowered to an elevation equivalent to that of the trunk sewer as a result of the extension of the North Tunnel.

The separate 21st Street drainage system had no branches and ran essentially in the north-south direction. The initial design called for a drop of approximately 40 feet at Fillmore. This drop was later shifted to Moreland. Finally, the two systems were interconnected at the upstream ends (between 19th and 20th Streets) and the combined system was modeled as a unit. As will be described later, the combined system requires less surge relief structures.

For each run of the model, a set of inflow hydrographs are selected as the input to the model, always matching the time origin of the hydrographs and the initiation of the computer simulation. At the beginning of a simulation, the drainage system may be assumed empty or filled with water from the previous rainfall.

1. Simplified 2nd Street System

For the reason previously stated, the east branch was not included in this model. The flow from the east branch as computed by HNTB was used as an input to this simplified model. Therefore, the results of this model are valid only if the elevation of the east branch is so high in comparison with the rest of the system that the east branch is hydraulically independent. Five complete simulation runs were made. The main purpose of these five runs was to study the general transient flow characteristics of the system rather than to determine the maximum design pressure. A schematic of the system is shown in Figure 6-4. The distance between two neighboring stations is assumed to be 350 feet for modeling purposes.

Run number A-1 modeled the 2nd Street system with the inital conduit sizes listed in Table 6-1 and inflow hydrographs provided by HNTB. At the downstream end of the system, there was a 40 ft. x 40 ft. reservoir and a trapezoidal channel with a 10-percent adverse slope leading to the river. The river was assumed to be dry and the tunnel empty at the beginning of the simulated storm inflow.

At about t=115 min. the tunnel began to pressurize at the downstream end and produced a pressurization shock wave that moved upstream at a speed ranging from 10 fps to 20 fps. This shock wave intensified when it reached the junction at t=132.5 min. During this period the flow between the pressurization wave and Grant Street was in the reverse direction (backflow). Flow

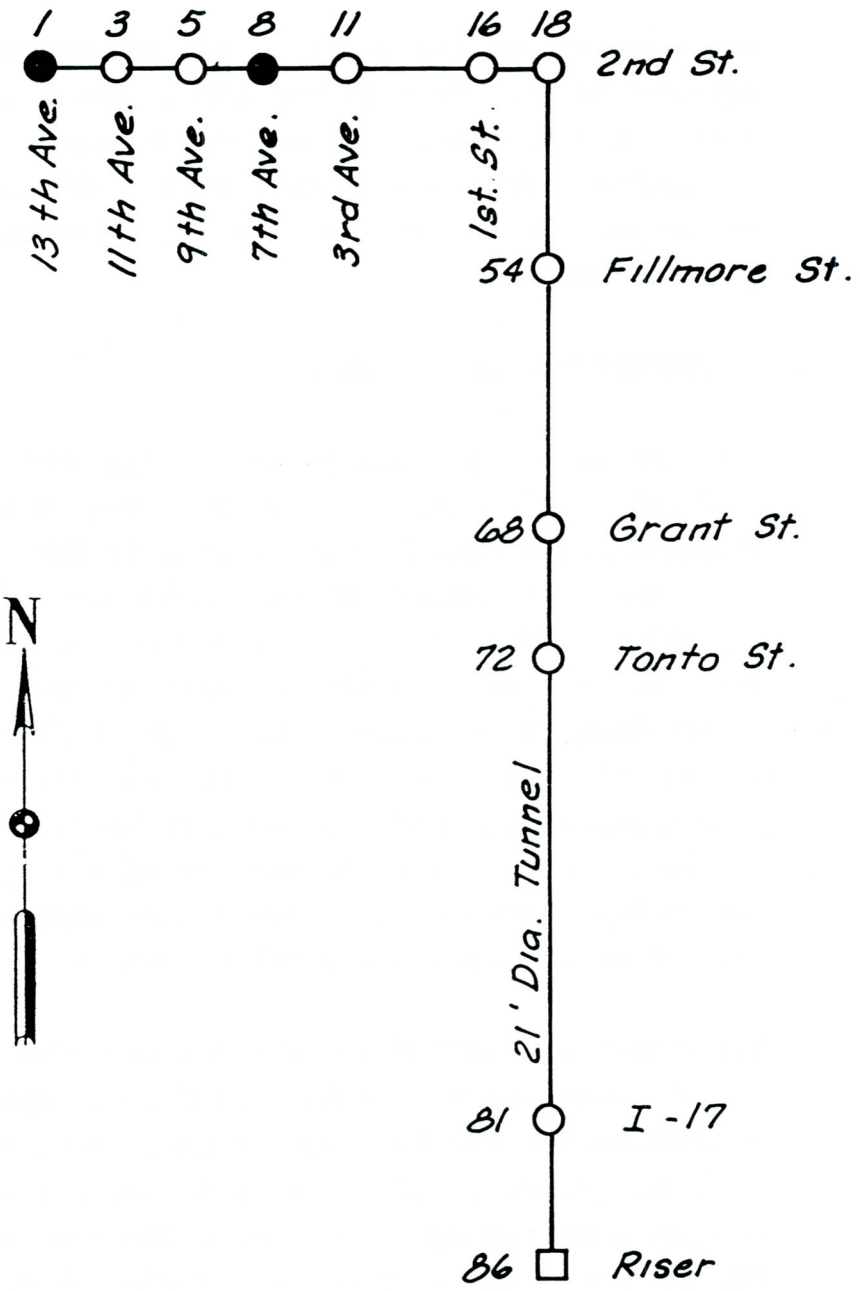


Fig. 6-4 2nd St. System Configuration
(Separate System - Original Model)

○ Inflow Points

● Surge Relief Structure

out of the system to the river commenced almost simultaneously with the pressurization of the junction at t=132.5 min. The pressurization wave reached the upstream end at about t=137.5 minutes.

Collision of the pressurization wave with the upstream end caused temporary high head and overflow at 7th, 9th, 11th and 13th Avenues. The overflow was largest at 13th Avenue (about 200 cfs) and lasted for about 4 minutes. A few minutes later, the west branch depressurized briefly and repressurized once more at t=170.7 minutes. This again caused overflow at 9th, 11th, and 13th Avenues for 11 minutes at reduced rates. About 10 minutes after the time of peak inflows, the west branch depressurized permanently.

This run clearly showed that the system requires some surge relief facilities in order to avoid excessive transient pressure and backflows. Some fundamental reasons for the problem are listed below.

- Relatively high river level as compared with the tunnel level removed the pressure relief mechanism of outflow during the pressurization period.

- Lack of storage space at the riser shaft between elevation 21 (crown of tunnel) and elevation 35, as measured from the invert, resulted in a rapid rise of backwater level during pressurization. This was responsible for the relatively large backflow (640 cfs maximum) occurring in the main channel during the pressurization period.

- Lack of storage space and large inflow at the 2nd Street junction caused even larger backflow (1400 cfs maximum) in the west branch during the pressurization period.

- Lack of storage space at 13th Avenue and a large backflow was responsible for overflows from the west branch.

- Large inflows during the pressurization period made the pressurization process very dynamic.

Remedies to the transient problems, therefore, can be found through elimination of one or more of the items listed above.

In run A-2, the physical dimensions of the drainage system were identical to those of the first run. However, the Salt River was modeled at its 10-year flood stage. The West Tunnel and the 14-foot diameter North Tunnel were assumed to be full at the beginning of the simulated inflow.

The flow was established much more smoothly in this case. No severe surge was predicted. The upstream portion of the system between 7th and 13th Avenues did pressurize, but no overflow occurred in any portion of the system. Because of these results, all subsequent runs assumed the worst case initial conditions, with the tunnel empty at the start of the storm.

A simulation run (A-3) using the 10-year inflow hydrographs showed no pressurization in the branch between 7th Avenue and 13th Avenue. No overflows occurred in this case. The main tunnel did pressurize and some backflow was predicted. It appeared that the main tunnel would have to withstand approximately 100 feet of transient pressure head.

Run A-5 was made without the storage reservoir at the downstream end of the main tunnel. Approximately 300 feet of closed conduit with adverse slope was located at the downstream end. 10-year inflow hydrographs and initially empty tunnels were assumed.

The timing and intensity of pressurization was comparable to run A-3 which included the reservoir. The lack of a reservoir

appeared to have little effect on the flow due to the 10-year storm. The same conclusion was assumed to hold for the 50-year event.

2. 21st Street System

Run number B-1 was for the 21st Street system with the original physical dimensions (with the Fillmore drop shaft) and the 50-year inflow hydrographs provided by HNTB. A layout of the system is shown in Figure 6-5. The distance between two neighboring stations was 300 ft. for this system. The riser shaft and the river conditions were similar to that of run A-1. The river was assumed to be dry and the tunnel empty at the beginning of the storm.

The tunnel started to pressurize at the downstream end at about t=125 minutes. Thereafter, the flow in the tunnel behaved very similar to that of run A-1 for the 2nd Street system except that the intensity of the pressurization wave was slightly lower. Outflow to the river started at t=133.7 minutes. Substantial amounts of pressure surges occurred in the 21-foot diameter tunnel downstream of Fillmore but the conduit upstream of Fillmore did not pressurize and there was no overflow.

Because of the surges, the maximum design pressure without a surge relief structure at Fillmore would be substantially above the steady flow pressure.

Run B-2 was made for the same system using the hydrographs generated by the 100-year storm as the input. The results were very similar to those of run B-1 except that the culvert upstream of the drop pressurized starting at t=179 minutes. Only a small amount of overflow occurred at the upstream end. Three additional runs were made for slightly modified physical parameters including the shifting of the drop from Fillmore to Van Buren. The hydraulic characteristics remained essentially unchanged.

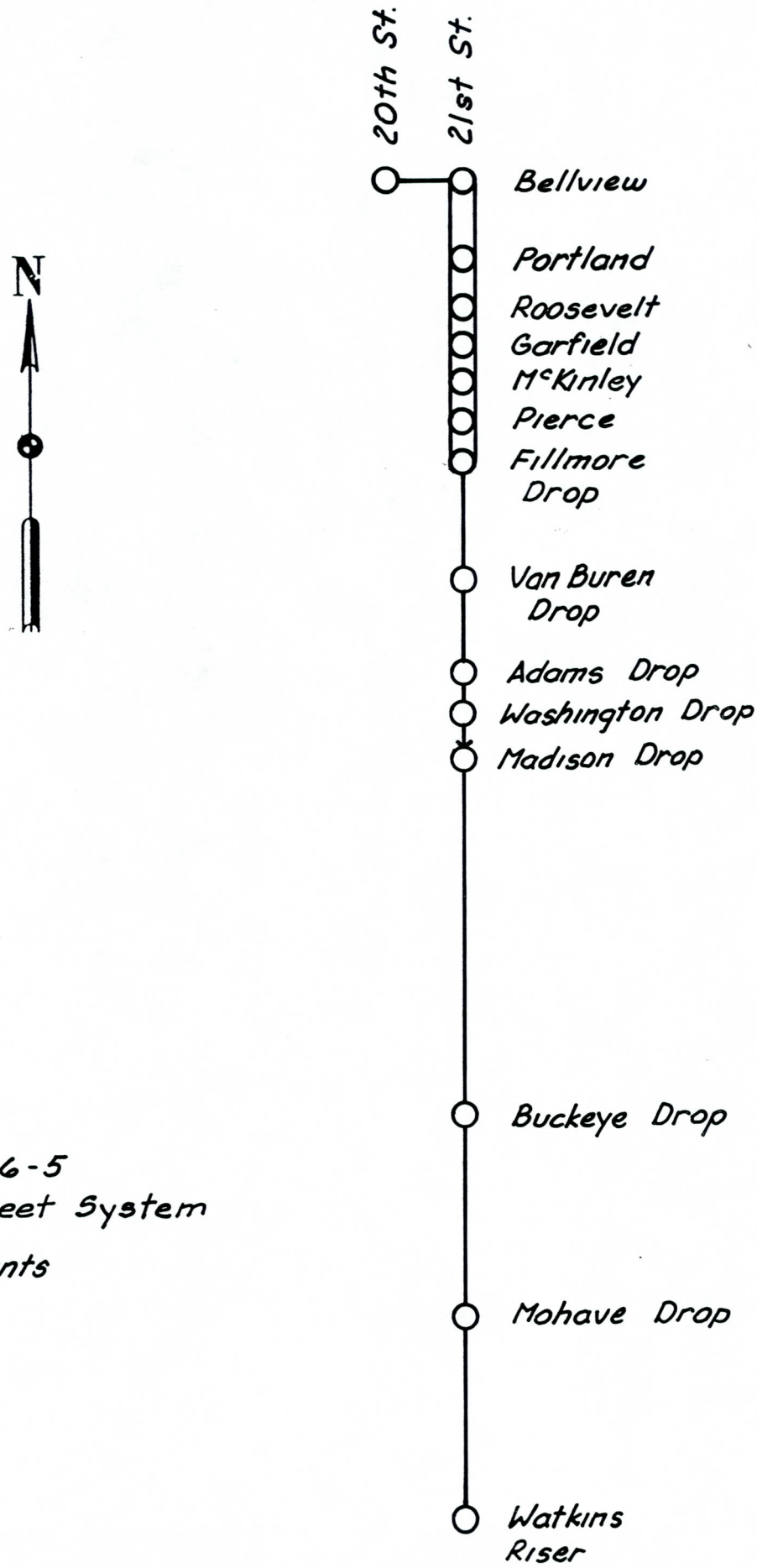


Fig. 6-5
21st Street System

○ Inflow Points

The mathematical modeling results showed the system to have satisfactory hydraulic characteristics that need no major modification. Since the entire system pressurizes for the 100-year storm, the system should end upstream with a vertical shaft of 8 ft. or greater diameter. The design maximum head should be 100 ft.

C. Refined System Modeling

1. Addition of East Leg

After numerous revisions and modifications it was decided to incorporate the "East Leg" from 2nd Street to 19th Street as part of the 2nd Street system for modeling purposes. A schematic of the system is shown in Figure 6-6. There were a number of design changes during the modeling process and a total of 40 simulation runs were made. Because there were so many runs made, and because it would serve no purpose to report the details of each run, only some highlights and typical features of the modeling results will be described herein.

a. Surge Front Modeling

The pipe sizes, invert elevations and inflow hydrographs were provided by HNTB. HNTB provided no drop shaft sizes. For this reason a rough estimate was made according to the inflow quantity without considering possible transient effects. These estimated drop shaft sizes, called the "initial drop shaft size" are listed in Table 6-3.

The overflow level for each drop shaft was assumed to be equal to the ground surface elevation. Outflow from the riser shaft to the river would occur when the water level is 35 feet above the invert.

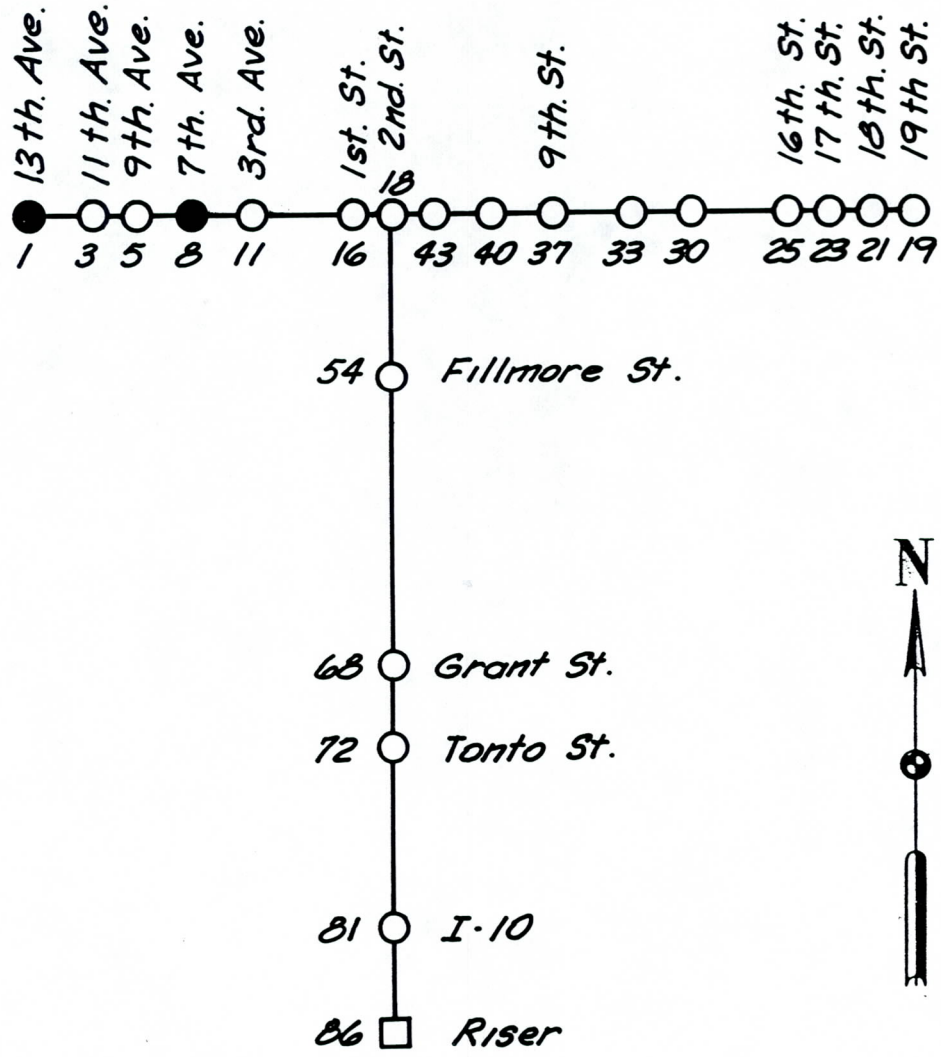


Fig. 6-6 2nd St. System Configuration
(Separate System - Original Model)

○ Inflow Points

● Surge Relief Structures

TABLE 6-3

INITIAL DROP SHAFT SIZE

Location	Station	Diameter (ft)
13th Ave.	1	7
11th Ave.	3	8.5
9th Ave.	5	15
7th Ave.	8	14
3rd Ave.	11	20
1st St.	16	14
3rd St.	43	14
7th St.	40	14
9th St.	37	14
13th St.	33	20
15th St.	30	10
16th St.	25	14
17th St.	23	6
18th St.	21	6
19th St.	19	6
Fillmore	54	14
Grant	68	4
Tonto	72	10
Maricopa Freeway	81	6

When the 50-year hydrographs were applied to the system initially empty, the downstream end began to pressurize at about t=124.7 minutes. As the tunnel filled a pressurization surge gradually moved upstream. Disregarding the oscillatory disturbances, the surge amplitude and the backflow behind the surge front increased each time the surge front passed a drop shaft. By the time t=136.5 min., when the surge front arrived at the junction at 2nd Street, the backflow increased to about 500 cfs. When this backflow combined with the incoming flow of about 2400 cfs, there was a change in the flow rate of 2900 cfs across the surge front. The surge split at the 2nd Street junction and continued to proceed upstream in each branch. The magnitude of the surges at this time in the west branch and the east branch were 1300 cfs and 1000 cfs, respectively. At about the time the surge reached the junction, the outflow to the river at the downstream end also began.

The difference in the discharge at the surge front increased to 2100 cfs when the surge front passed the 3rd Avenue drop shaft. The backflow behind the surge front at this time was 1700 cfs. This large increase in the backflow is due to the large inflow at the 3rd Avenue drop shaft. The backflow moved up the drop structure at 7th Avenue and entered the 10-foot square conduit at about t=140.0 minutes. It finally reached the upstream end at 13th Avenue at t=141.3 minutes and started to cause overflow at 13th Avenue, 11th Avenue, 9th Avenue, and 7th Avenue. The overflows lasted for about 2 minutes and did not occur again until the peak inflow period.

No overflow was indicated from the east branch due to its relatively high elevation. The outflow to the river, at the beginning of overflow, was 2300 cfs and rising very rapidly.

For this initial separate system design, a maximum head of 500 feet was predicted to occur between the riser structure

and the Maricopa Freeway drop shaft. This was an oscillatory waterhammer pressure occurring during the period of overflow and persists for more than 2 minutes. The maximum head dropped to 200 feet between Maricopa Freeway and Tonto Street. It declined to about 100 ft. further upstream. A maximum head of 200 feet was also predicted to occur between 11th Avenue and 9th Avenue. This head occurred immediately after the surge hit the upstream end and lasted for only a few seconds.

b. Alternative Methods for Elimination of Street Flooding

The majority of the 40 runs were made to find the optimum design which will reduce the surge sufficiently to eliminate the street flooding. Four effective methods were identified and studied in detail. Three of the four alternatives are described below. The fourth alternative method is the optimum method described in Section C.

- Increase the drop shaft and manhole height.

An obvious solution to street flooding is to continue the drop shaft above the ground surface high enough to prevent overflow. Modeling indicated that extending the drop shaft 55 feet above the ground surface would be required to prevent the overflow at 13th Avenue if the drop shaft sizes are as shown in Table 6-3. The required height decreases as the drop shaft sizes are increased.

This method was not recommended because it would also increase the maximum transient heads beyond reasonable amounts, 500 ft. or more.

- Lower the outflow elevation

A major factor affecting the surge is the timing of the outflow with respect to pressurization. In the initial design, outflow did not occur until the surge arrived at the 2nd Street junction and the 21-foot diameter tunnel was completely pressurized. Mathematical modeling runs predict that the overflow will be eliminated if the outflow elevation is reduced by 5 feet. In this case the outflow to the river starts at t=132.2 minutes when the surge front is still 7,000 feet away from the 2nd Street junction. The outflow has reached 700 cfs by the time the surge hits the 2nd Street junction, and 1600 cfs when the surge arrives at the 7th Avenue drop structure. This increased outflow reduces the surge intensity enough to prevent the overflows in the west branch.

If the outflow elevation cannot be reduced because of the river bed elevation constraint, then raising the tunnel elevation should have a beneficial effect. No modeling runs were made to study the effectiveness of raising the tunnel elevation. But, it is expected that the elevation would have to be increased by more than 5 feet to prevent street flooding because the relative elevation of the west collector line will also decrease if the tunnel is raised. This alternative was not feasible because of geological constraints.

- Surge tanks

Effective surge tank locations for eliminating the overflows were identified at 13th Avenue, 7th Avenue, 3rd Avenue, and 9th Street. Assuming the pipe sizes are the same as those given by the preliminary design and listed in Table 6-1, the surge tank areas required to eliminate the overflows were determined by trials. The results are listed in Table 6-4.

Surge tanks are assumed to consist of cylindrical tanks of constant cross-sectional area extending from the invert of the tunnel to the ground surface. Other types of surge tanks may also be used but would require separate analysis.

TABLE 6-4

SURGE TANK REQUIREMENT WITH ORIGINAL PIPE DIAMETER

Location	13th Ave.	7th Ave.	3rd Ave.	9th St.
Size, ft^2	180	7,000	1,250	1,250

When the inflow hydrograph at 3rd Avenue was reduced by relocating the proposed storm drain connection, the surge tank requirement there was eliminated.

c. Optimum Design for Prevention of Overflow

As stated previously, the cause of overflow is the backflow behind the pressurization surge. When this backflow is obstructed by a reduction in pipe diameter or stopped by the end of a conduit, pressure builds up and overflow occurs. To relieve this pressure build up, there must be sufficient storage capacity at upstream ends and points where significant change in pipe size occur.

However, a surge tank located in the middle of a line, such as the 7th Avenue drop, is not very efficient because much of the backflow may simply bypass it. To increase the pipe diameter upstream and concentrate the storage reservoir at the upstream end would be more efficient. After some trial runs, the following combination was determined to be the optimum.

a. Increase the conduit size between 13th Avenue and 7th Ave. to 10 ft x 12 ft.

b. Use a reservoir of 4,000 sq. ft. at 13th Avenue and a 700 sq. ft. surge shaft at 7th Avenue.

No surge tanks are needed at 3rd Avenue and 9th Street.

Increasing the conduit size between 13th Avenue and 7th Avenue prevents the first surge from pressurizing the conduit. Instead, the backflow will be carried in a form of a moving hydraulic jump which is considerably weaker than the pressurization surge that would take place in the smaller conduit. Only long after the bore reaches the upstream end does a new pressurization surge occur in the duct. This two-step change makes the system much less dynamic and prevents the overflow.

d. Design Pressure Head

The transient pressure generated in the 2nd Street system was found to be closely related to the pressure build up at the final stage of pressurization. For this reason, the transient pressure head is lowered when the overflow is reduced. Under the recommended conditions previously described, the design pressure head of 100 ft. over the whole system is recommended except in the reach upstream of 9th Street where much smaller values can be used because the pipe will not pressurize there.

e. System Capacity

With the recommended pipe sizes, small amounts of overflow (less than 70 cfs) would occur for about 15 min. during the peak inflow period of the 50-year storm at the 13th Avenue, 11th Avenue, 9th Avenue, and 7th Avenue inflow points if

Manning's "n" is taken to be 0.013. Since the amount of overflow is so small compared with the total flow rate at the time, the system capacity matches the 50-year storm almost perfectly. In fact, no overflow occurs when the Manning's "n" is reduced to 0.012. Here, the overflow consists mostly of the runoff that fails to enter the drainage tunnel and is not necessarily equivalent to the backflow. Since the runoff prediction cannot be that accurate and since the Manning's "n" should be closer to 0.012 when the tunnel is new, this system should be considered to have sufficient capacity to carry the 50-year storm runoff.

2. Revised 21st Street System

The physical dimensions of the system were revised by HNTB in December 1983. The drop originally located at Fillmore was shifted to Moreland, lengthening the 21-ft. diameter tunnel by approximately 3300 feet. The 50-year and 100-year hydrographs were updated to include the effects of the Papago East storm sewer. A total of 16 runs were made.

a. Hydraulic Characteristics of Original Design

The first run was made with the 50-year hydrographs with no provision for surge relief. Unlike the previous system, this revised system completely pressurized and caused overflow at 20th Street, Pierce, Van Buren, Washington, Madison, and Buckeye Streets. The overflow started at t=141.2 minutes and lasted for about 2 minutes.

For the 100-year hydrographs larger amounts of overflow occur, once at the time of pressurization and a smaller amount at the time of maximum inflow. The first period of overflow starts at t=142.0 minutes and lasts for about 6.5 minutes at Van Buren and Washington Streets.

b. Elimination of Street Flooding

After several trial runs, it was determined that the independent 21st Street system required a combined surge tank area of 10,000 sq. ft., about equally divided between the Moreland Street drop structure and the 20th Street drop shaft.

c. Design Pressure Head

The design maximum pressure head of 100 ft. may be used under the condition that the surge structures recommended above are provided.

3. Interconnected (Combined) System

a. Description of the Combined System

The primary purpose of the interconnection between 19th and 20th Streets was to provide an outlet for the existing 16th Street storm sewer during construction of segment C of the Inner Loop Freeway, prior to completion of the West Tunnel and Storm Water Interceptor system. The interconnection was incorporated into the transient wave model to determine if the size of the surge protection facilities might be reduced if one system could be relieved into the other. This was confirmed by the initial runs. By enlarging the pipe between 16th and 20th streets from 72-inch to 90-inch diameter the need for a 5000 sq. ft. surge chamber at 20th Street was eliminated without affecting the 2nd Street system. Figure 6-7 shows the configuration of the combined system.

The initial runs simulating the combined system ignored the double 12'x12'x500' culverts connected to the Moreland Street drop structure. These culverts were later included as a surge relief structure. The system was modeled using the

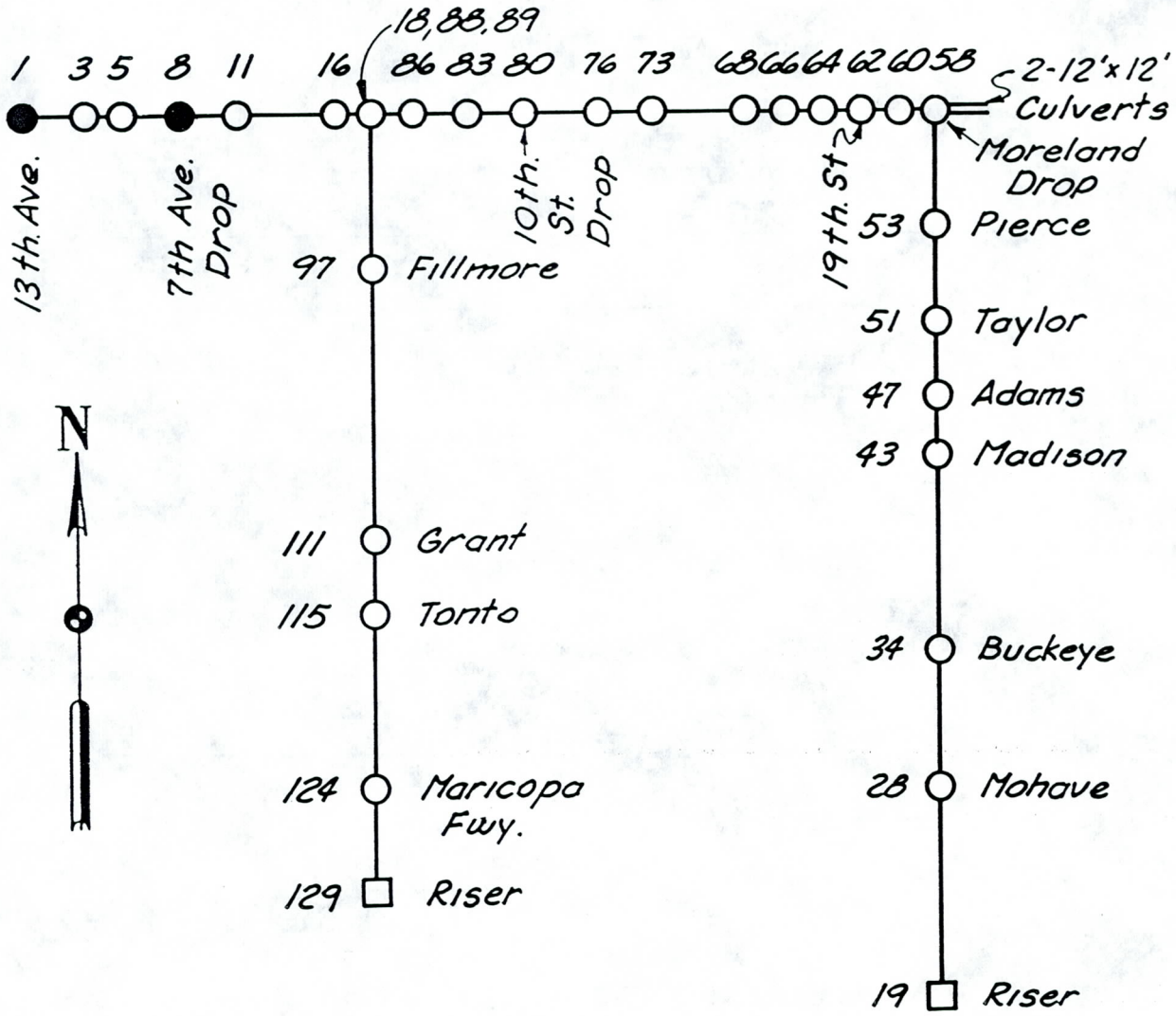


Fig. 6-7 Combined System Configuration
(Final Model)

○ Inflow Points

● Surge Relief Structure

50-year, 25-year and 10-year inflow hydrographs, both with and without the Papago East storm drain. The results of run CD-9 were judged to be the most satisfactory. Table 6-5 summarizes the pipe size distribution used in the final design runs.

TABLE 6-5

PIPE SIZE DISTRIBUTION USED FOR THE FINAL DESIGN RUNS

FROM	TO	PIPE SIZE
13th Avenue	7th Avenue	10′ x 12′
7th Avenue	2nd St. Junction	14′D
2nd St. Junction	West Riser	21′D
2nd St. Junction	10th Street Drop	14′D
10th Street Drop	12th Street	10′ x 10′
12th Street	15th Street	8′D
15th Street	20th Street	7.5′D
20th Street	Moreland Drop	8′D
Moreland Drop	East Riser	21′D

The conditions for run CD-9 are listed below:

- Pipe size distribution as listed in Table 6-5.

- Each riser shaft is connected to two inclined pipes which drain into an open channel.

- Surge relief structures: a pond at 13th Avenue which has a cross-sectional area of 5000 sq. ft. at the invert elevation and 43,000 sq. ft. at 14 ft. above invert; a 30 ft. diameter drop structure at 7th Avenue; the double 12′ x 12′ x 500′ culverts at Moreland Street used as a surge relief structure.

- Hydrographs: 50-year storm with the Papago East storm sewer in place.

- Initial Condition: Tunnel system empty and no flow in the Salt River.

Unless otherwise noted, the following description of the simulation results is based on run CD-9.

b. **Hydraulic Characteristics**

The two 21 ft. diameter tunnels complete their pressurization at almost the same time under the condition of run CD-9 (t=139 min. for the 2nd Street tunnel and t=140 min. for the 21st Street tunnel). The flow in the combined system is nearly identical to that of the independent systems until about t=142 min. when the backflow from the 21st Street tunnel arrives at the east branch of the 2nd Street system. Initially, this backflow may have the intensity of 600 cfs and pressurizes the east branch from upstream. The intensity of this positive surge will gradually decrease as it travels along the east branch and becomes insignificant by the time the surge front reaches 13th Street.

As described above, the east branch of the 2nd Street system plays the role of surge relief for the 21st Street system. The increased flow to the 2nd Street system is not enough to alter the characteristics of the 2nd Street system. Thus, by interconnecting the two systems, the surge structure at 20th Street required for the independent system can be eliminated.

The combined system would still require a surge relief structure of about 5000 sq. ft. at Moreland to avoid overflows at Buckeye, Madison, and Adams if the double 12'x12'x500' culverts at Moreland are ignored. Subsequent runs indicate that the culverts can be used as the surge

relief structure. However, in order for the culverts to be effective, the following conditions must be met.

- The culverts must be nearly empty when the surge front arrives at the Moreland Street drop structure. This means the culverts should not carry much flow during pressurization. This condition is satisfied by the hydrographs for the 50-year storm without Papago East, but the surge relief capability of the culverts needs to be re-evaluated in the future should they carry significant amounts of flow.

- The culverts must be ventilated at the upstream end to evacuate air at the rate of 600 cfs each.

- The elevation of the culverts at Moreland should be about the same as that of the 8 ft. pipe.

The combined drainage system responds to the 50-year storm with Papago East roughly the same way as it responds to the 50-year storm without Papago East, with the exception noted above.

c. Hydraulic Grade Line

Because the flow is very dynamic, only instantaneous hydraulic grade lines can be plotted. By plotting the hydraulic grade line at different times, it is possible to observe the movement of the surge front as the tunnel is being filled from the downstream end. The hydraulic grade lines along the continuous line joining the 21st Street tunnel and the 2nd Street tunnel are plotted in Figure 6-8. The vertical lines shown in these figures indicate the inlet structures and the tops of these lines correspond to the ground surface. Overflow takes place when a hydraulic grade line moves above the top of a vertical line. Station numbers correspond to those shown in the system layouts.

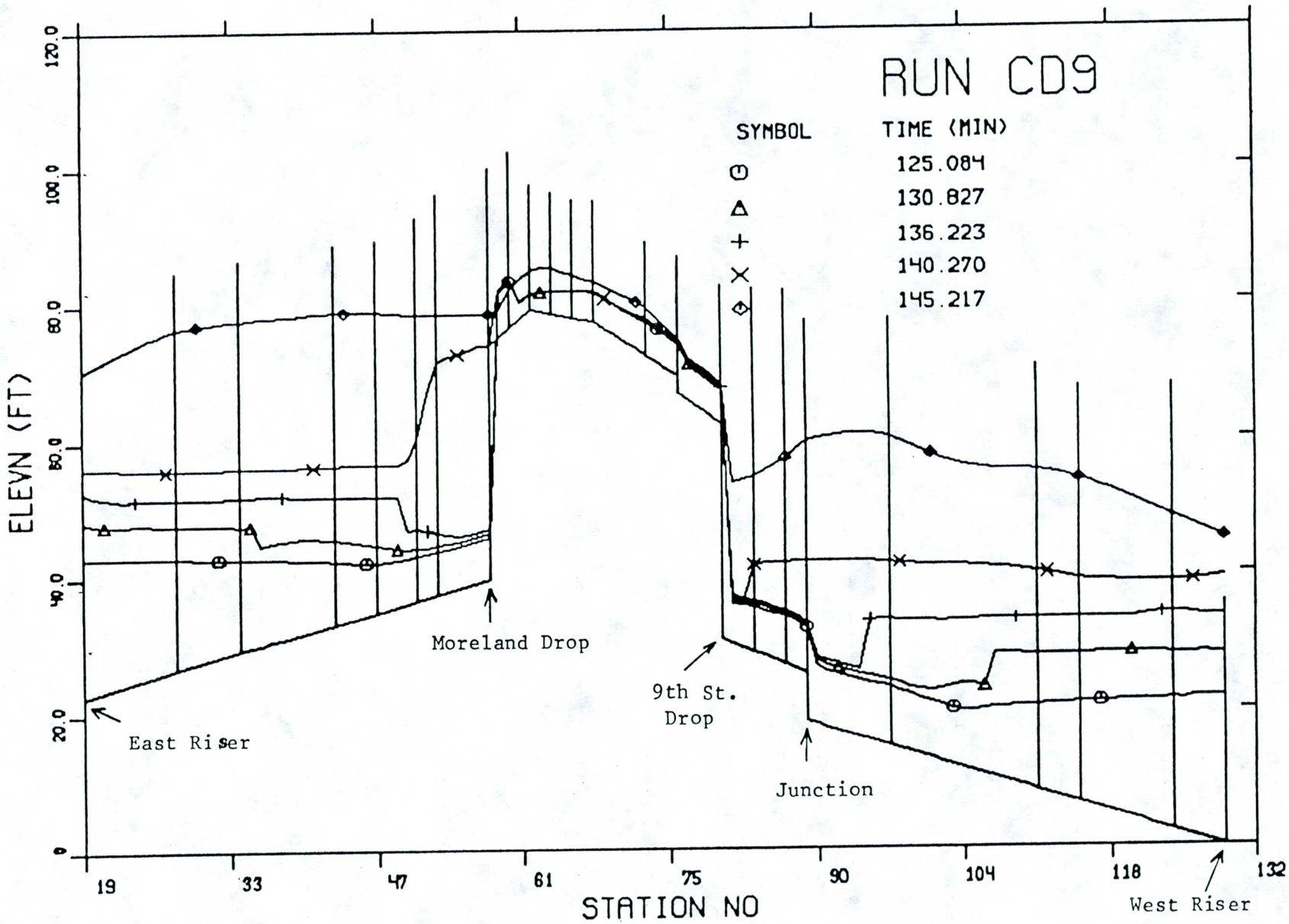


Figure 6-8 Hydraulic Grade Lines During Pressurization
East Tunnel - West Tunnel Connected

Figure 6-9 shows hydraulic grade lines at five instances shortly after the system has been pressurized and the inflows are still increasing. The flow is not very dynamic during this period and the hydraulic grade lines rise slowly as the inflows continue to increase. There is a slight negative pressure in the east branch of the 2nd Street tunnel system near 171 minutes due to siphon action. Note that the hydraulic grade line in the 21st Street tunnel is already receding, but it is still rising in the 2nd Street tunnel at 171 minutes.

Figure 6-10 shows hydraulic grade lines at three instances near the peak inflow period. There are many other hydraulic grade lines for this and many other runs but they are not included in this report to avoid confusion.

d. **Hydrographs**

The effects of the surge and backflow can also be seen in the hydrographs plotted at various points along the tunnel system. Several of these are included as examples.

Figure 6-11 shows the hydrograph at 13th Avenue as computed by run CD-9. A brief period of negative discharge between 140 minutes and 150 minutes is due to the backflow produced by the surge. The other sharp oscillation occurring between 180 minutes and 190 minutes is due to pressurization starting at 13th Avenue at the time of peak inflow. This last pressurization at the west branch causes the hydraulic grade line shown in Figure 6-10 to rise in the 2nd Street system while the hydraulic grade line in the 21st Street system is already receding.

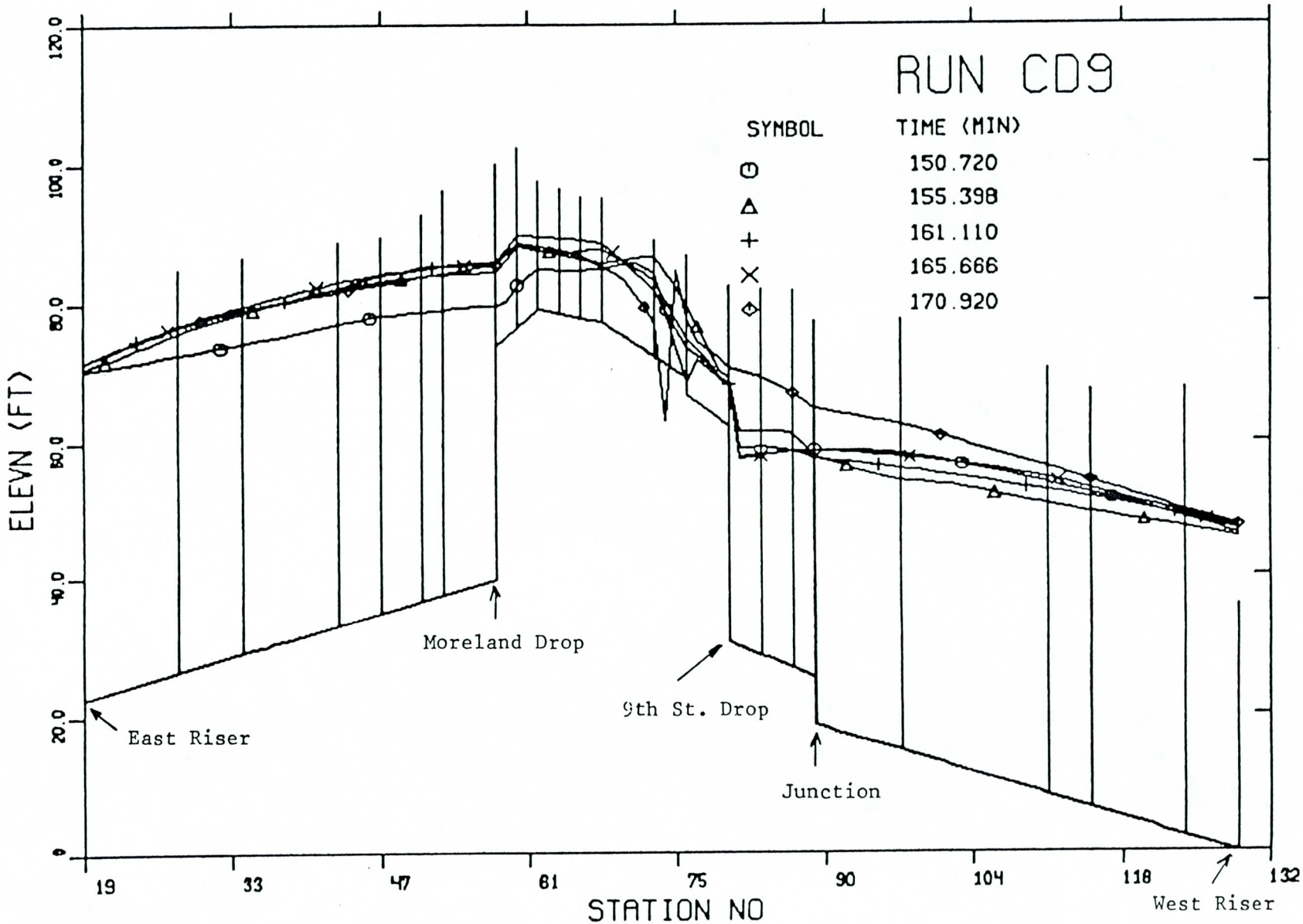


Figure 6-9 Hydraulic Grade Lines After Pressurization
East Tunnel - West Tunnel Connected

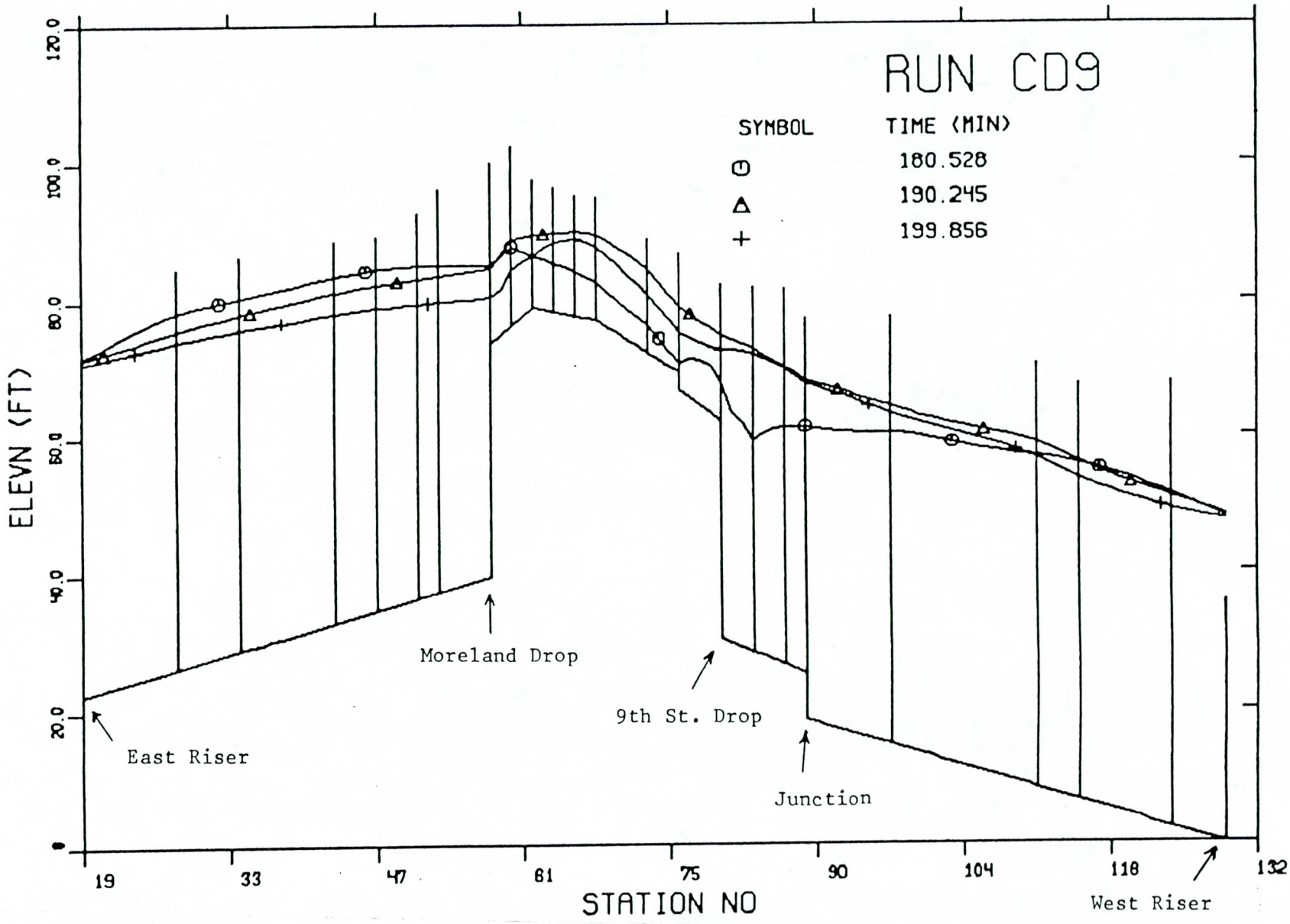


Figure 6-10 Hydraulic Grade Lines After Peak Inflows
East Tunnel - West Tunnel Connected

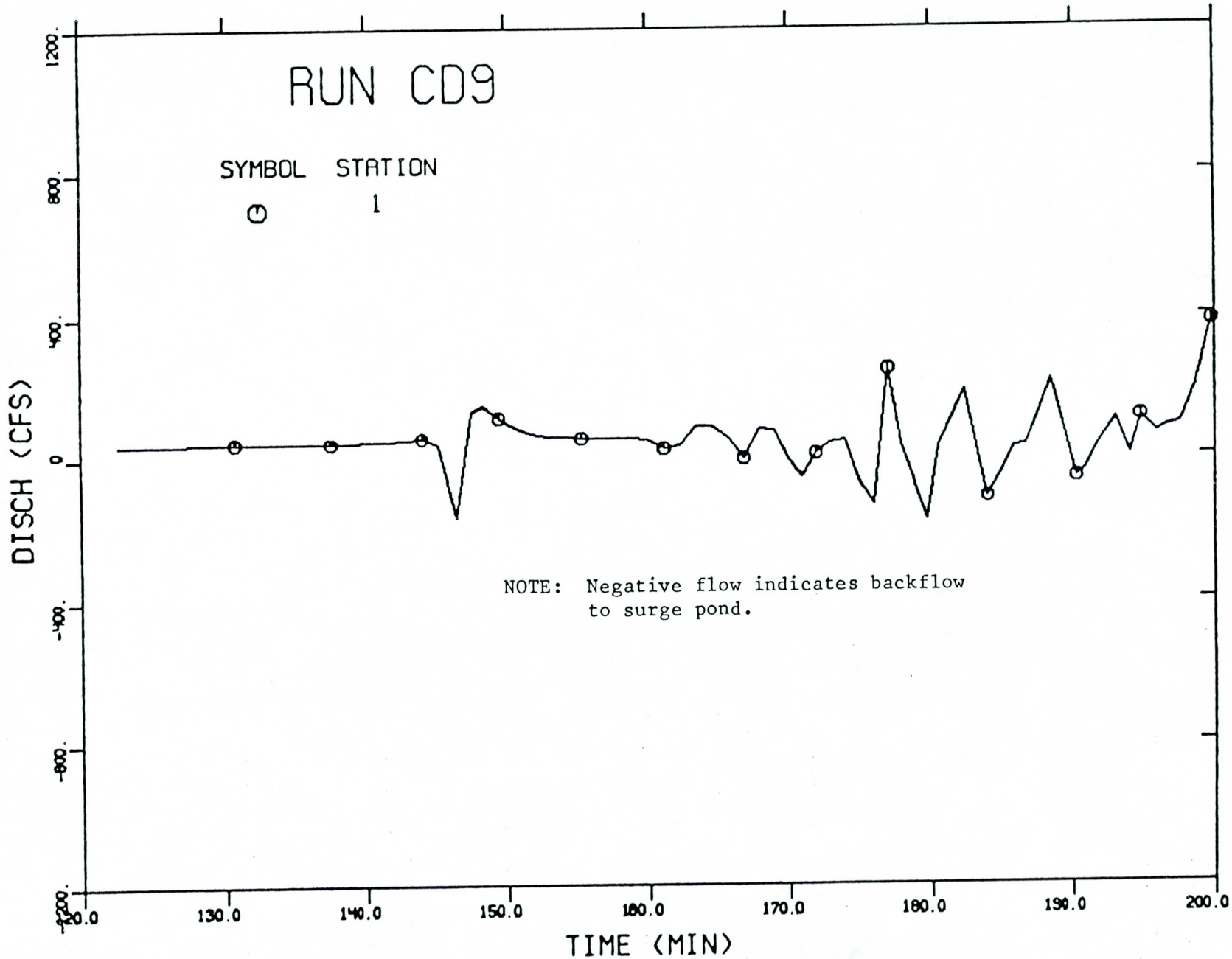


Figure 6-11 Hydrograph at 13th Avenue

Figure 6-12 shows the hydrograph in the west branch at the 2nd Street junction. Note that the backflow here lasts longer than that of Figure 6-11 because this point is further downstream (backflow starts with the arrival of the surge and ends shortly after the surge reaches the upstream end).

Figure 6-13 shows the hydrograph in the east branch at the 2nd Street junction. Because the initial flow in the east branch is somewhat greater than that of the west branch, the surge reduced but did not reverse the flow in the east branch.

Figure 6-14 shows the hydrograph just upstream of the Moreland Street drop structure in the 21st Street system. Note that the station numbering system is such that the normal flow direction in the 21st Street tunnel is regarded as negative but backflow is regarded as positive. For this reason, negative discharge in Figure 6-14 represents flow in a normal direction. With the double 12'x12' culverts acting as surge relief, the surge intensity at the Moreland Street drop is substantially reduced.

Figure 6-15 shows the hydrograph at a station between 19th Street and 20th Street. It is interesting to note that the flow is directed to the east branch before the surge arrives due to the large inflow at 20th Street, but the pressure in the 21st Street tunnel pushes some flow towards the 2nd Street system permanently after pressurization.

Figure 6-16 shows the outflows to the river from the two risers and some overflows due to the 50-year storm. Although overflows are indicated at four stations, these overflows are actually only very small amounts of runoff denied entrance to the drainage tunnel rather than backflows.

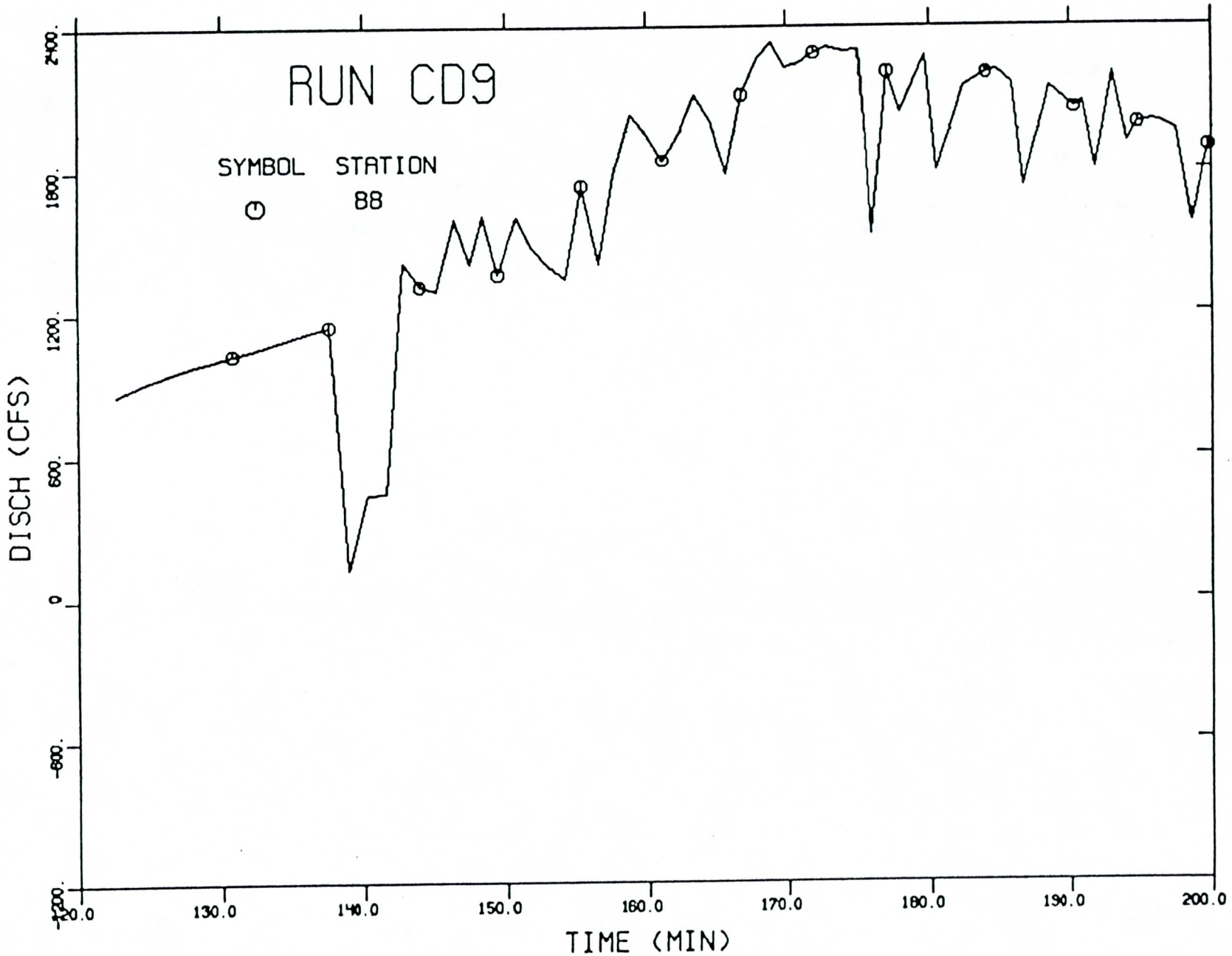


Figure 6-13 Hydrograph at 2nd Street Junction, East Branch

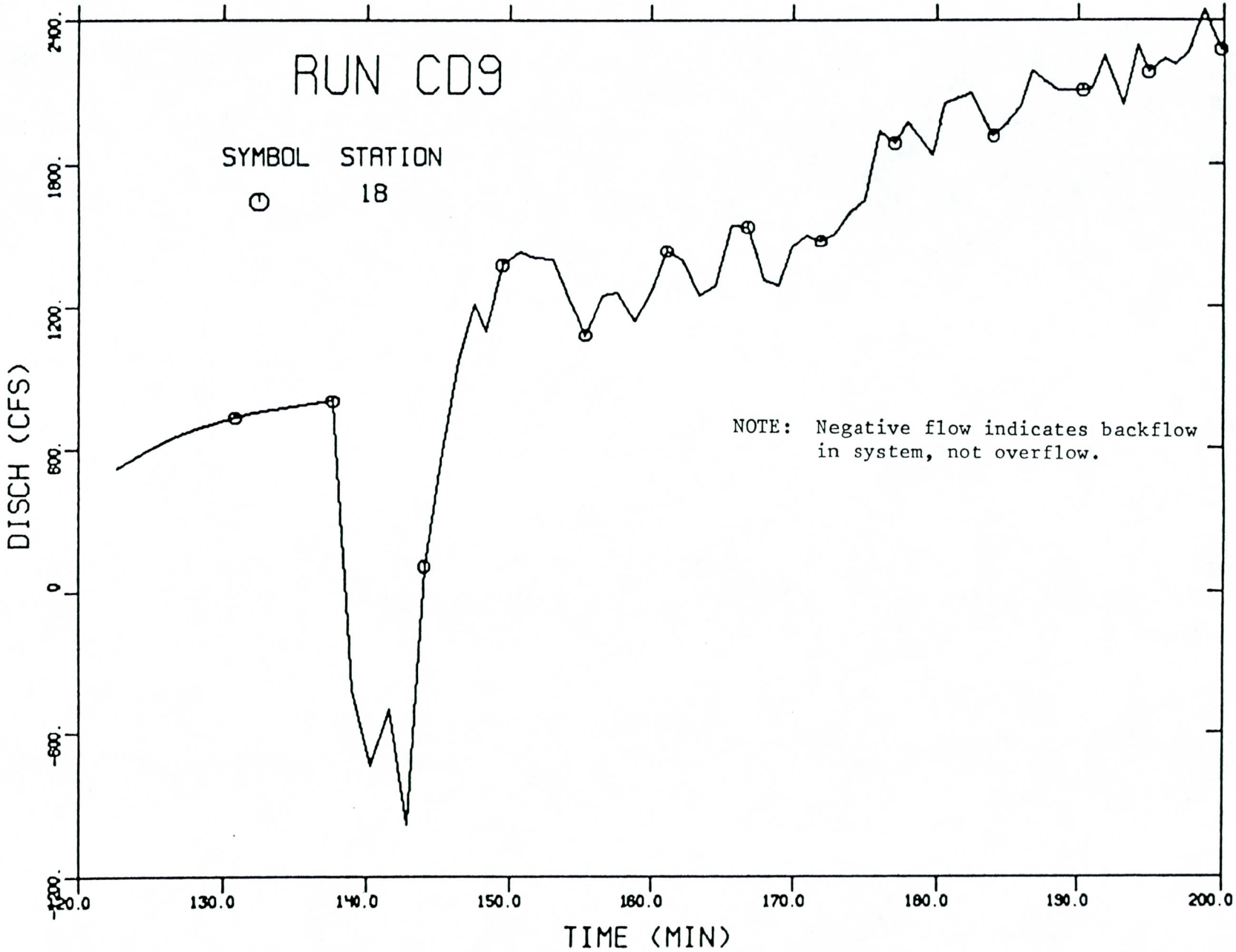


Figure 6-12 Hydrograph at 2nd Street Junction, West Branch

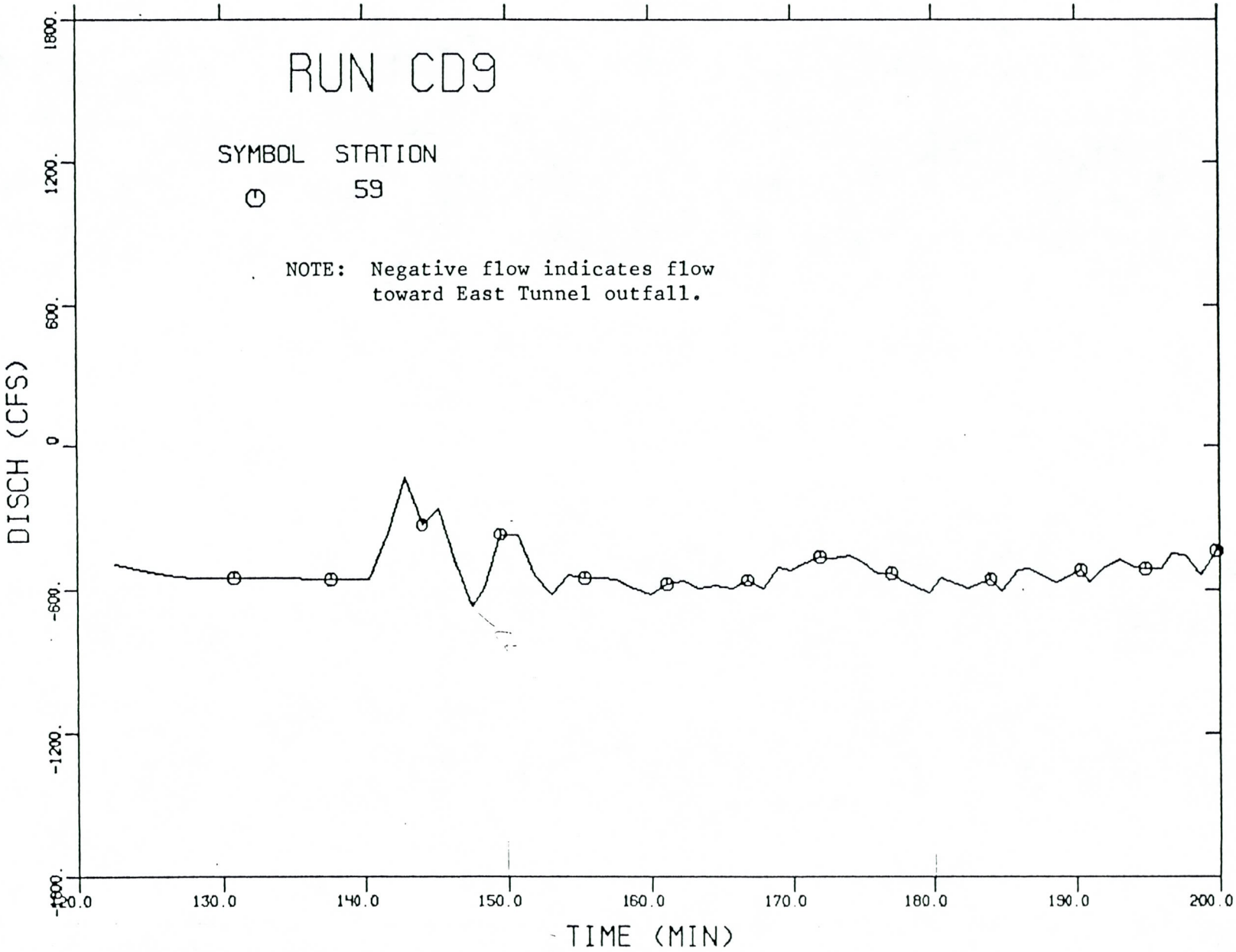


Figure 6-14 Hydrograph Above Moreland Street Drop

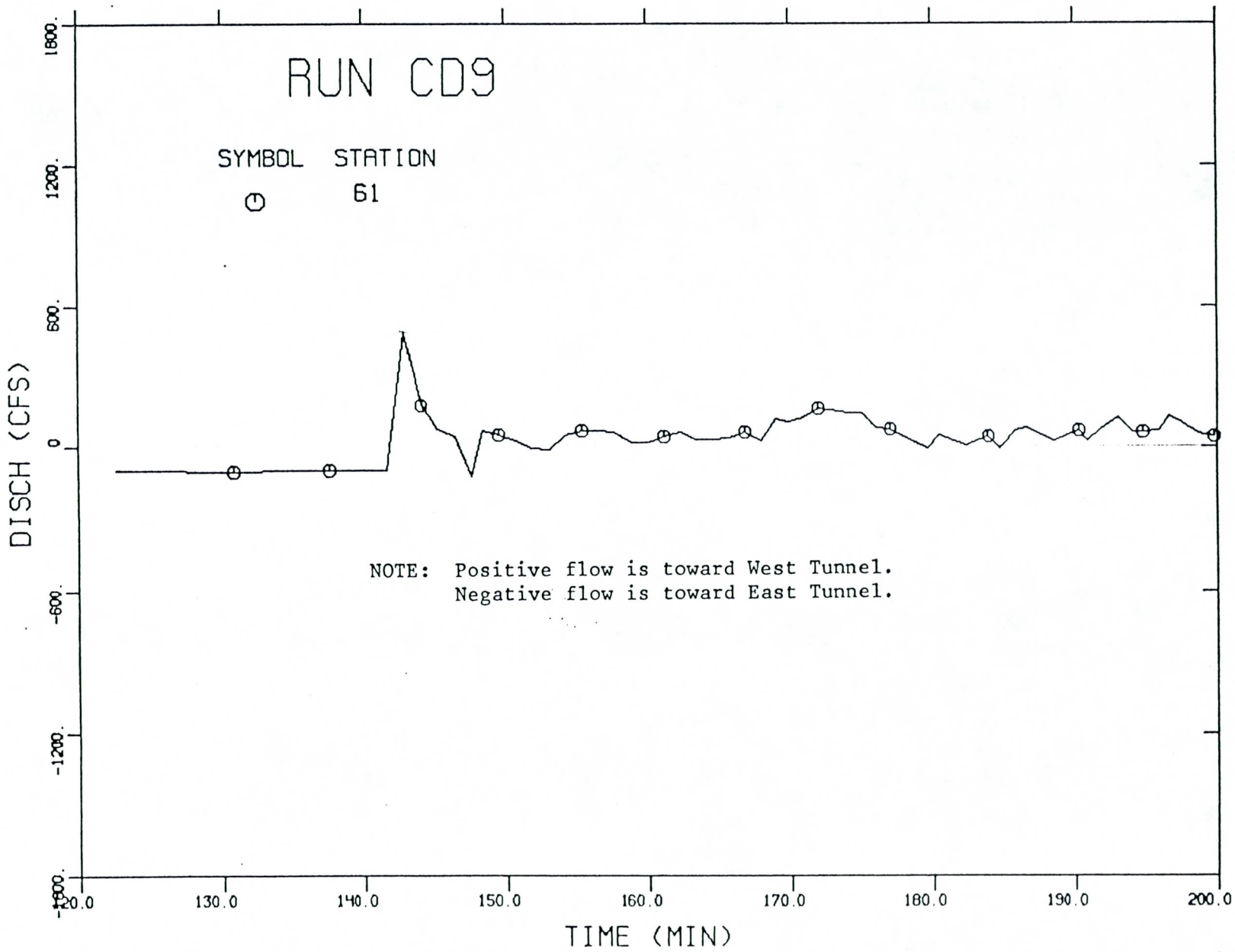


Figure 6-15 Hydrograph Between 19th and 20th Streets

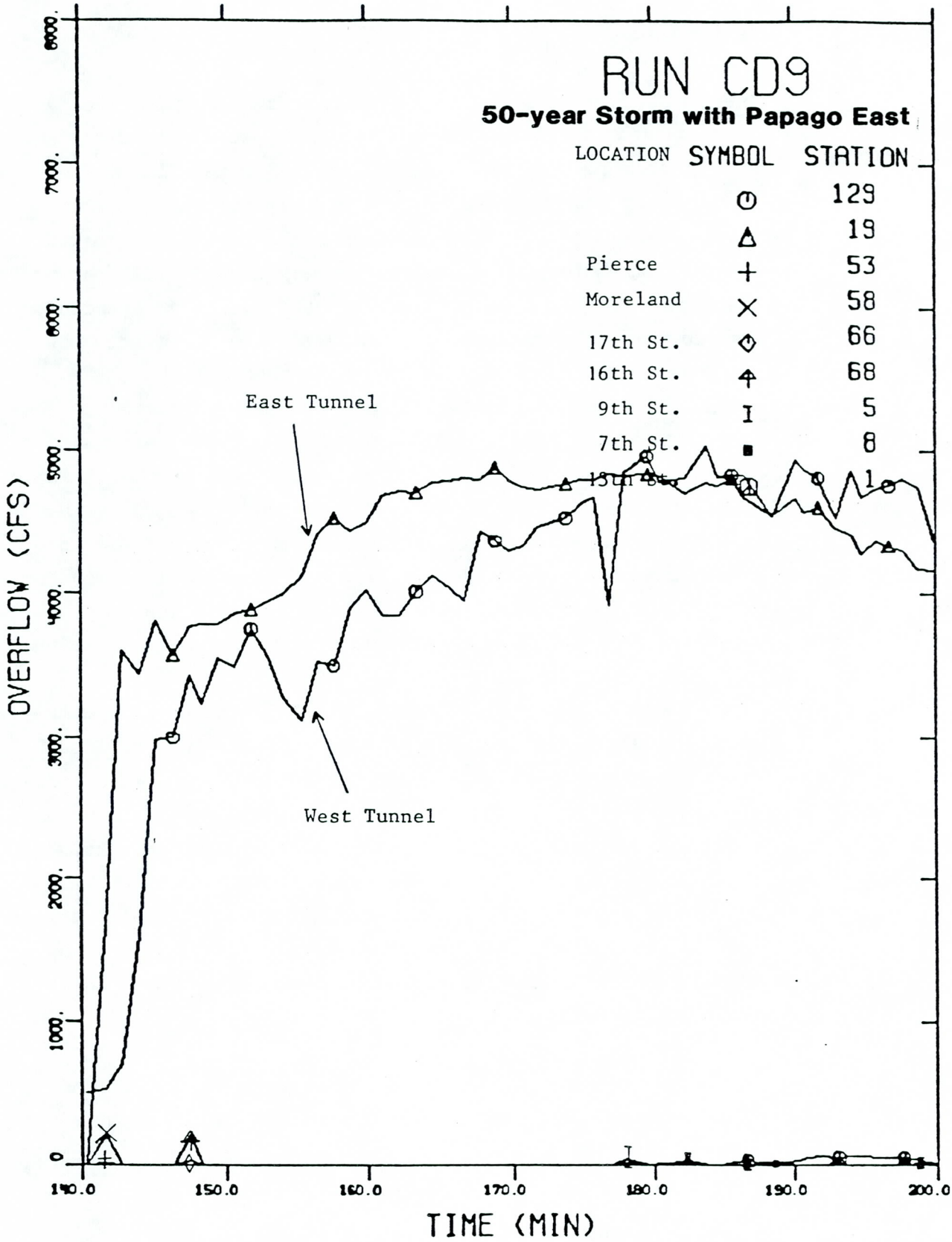


Figure 6-16 Outflow and Overflow Hydrographs

e. Design Head

The design head consists of two components, the quasi-steady head and the transient head. The quasi-steady head is a slowly changing head typically lasting for several minutes or longer. It can be obtained by measuring the water surface level in drop structures or manholes. The transient head is a rapidly changing (acceleration related) pressure head which lasts for only a few seconds or even a fraction of a second depending on the location. It is also known as waterhammer pressure which can be measured by a pressure transducer.

The quasi-steady head is proportional to the surge intensity but limited by overflow elevations. The transient head is also related to the surge intensity but not limited by overflow elevations. It is directly proportional to the pressure wave speed which is strongly affected by the amount of air bubbles contained in the water. For this reason, the transient head is a highly variable quantity.

In order to save the computational costs which are directly proportional to the pressure wave speed, artificially large pressure wave speeds (between 200 fps and 1000 fps) were used in the simulation runs. The computer transient heads were scaled up according to the pressure wave speed. The design head is obtained by adding the quasi-steady head and the adjusted transient head. It may be interesting to note that, for deeper tunnels, larger portions of the design head consist of the quasi-steady head, but for shallower tunnels the opposite is true.

During certain high flow periods, the pipe between 21st and 10th Streets acts as a siphon with pressurization and depressurization processes taking place. Flow under this condition is quite unstable and difficult to model

accurately. Consequently, the maximum head computed in this region may contain some error. This effect of instability usually results in over-estimation of maximum pressure.

f. **Effect of Storm Size**

There were different degrees of transient effects occurring in the drain system when different size storms were used for simulation. Run CD-9 indicates that the entire drain system will pressurize under the 50-year storm with the Papago East storm sewer, even with a pond and other surge relief structures. This is the worst condition which governs the design.

For the 25-year storm with Papago East, the west branch of the 2nd Street system will remain unpressurized for the whole storm period. However, the pond (may be a smaller size) is still needed because there is still a moving hydraulic jump and a backflow during the critical period of pressurization. About half of the east branch of the 2nd Street system will be pressurized for a short period of time as a result of the backflow from the 21st Street tunnel. This pressurization is rather weak and will be relieved shortly after its occurrence.

The 10-year storm will not pressurize either the west branch or the east branch of the 2nd Street tunnel. There will still be a mild moving hydraulic jump formed in the west branch and a small surge relief pond or tank is needed at 13th Avenue. For storm size equal to or smaller than the 10-year storm, the combined system will act like two independent systems.

g. Effect of Initial Storage

Generally speaking, the water initially stored in the tunnel causes the tunnel to pressurize earlier, at a time when the inflow rate is smaller. This will tend to reduce the surge intensity, backflows, and the related transient pressure head.

If the tunnel is initially full, as is the case when the Salt River is conveying the 10-year flood, even a storm as large as the 50-year storm will not cause a transient problem. The flow in the tunnel will respond to the gradually rising inflows smoothly. The tunnels have sufficient capacity to carry the peak flow without causing any backflow or overflow. It is estimated that the 50-year storm will cause no transient problems if the tunnel is more than 3/4 full at the beginning of the storm.

The amount of initial storage required to eliminate the transient problem decreases with decreasing storm size. No quantitative determination has been made for the storage requirement except for the 50-year storm.

h. Partial Surge Protection

The availability of space at 13th Avenue and the twin culverts of the future Papago East sewer led to the use of a pond at 13th Avenue and the possible elimination of the surge structure at Moreland Street. This use of the pond and the Papago East sewer as relief structures actually gave the system extra protection against the transient flow. The system will function almost as well if these surge relief structures are decreased by 25 percent. Even if the surge structure sizes are reduced by 50 percent, the maximum design head for the 50-year storm will only increase to 150 feet. A complete elimination of surge structures would result in

prohibitively high design heads. As an additional safety factor, it was decided to retain the Moreland structure in the system.

The surge structure sizes could be reduced for storm sizes less than the 50-year storm. There is a substantial excess storage capacity at the 13th Avenue pond for 25-year and 10-year storms. The pond size should not be reduced by more than 25 percent of its original design size.

For a storm less than the 10-year storm, there will be no backflow from the 21st Street tunnel to the 2nd Street tunnel. This means interconnection serves to relieve the surge in the 21st Street tunnel only for storms larger than the 10-year storm.

4. Squaw Peak Parkway Storm Drain

The model was revised to include the Squaw Peak Parkway storm drain, in order to determine the effect of surges in the I-10 tunnel system on the performance of the Squaw Peak storm drain and the need for surge protection facilities.

Model runs made with this revision indicated that the Moreland Street surge relief facilities absorb enough of the energy of the pressurization surge to protect the Squaw Peak storm drain. By the time the surge arrives at the Squaw Peak storm drain, the intensity of the surge has been reduced enough to prevent backflow in the Squaw Peak system. The surge will temporarily reduce Squaw Peak flow from 735 cfs to 570 cfs. This flow reduction will cause a temporary mild pressurization. The maximum head in this branch of the Squaw Peak storm drain is about 40 feet.

The inclusion of the Squaw Peak storm drain also reduced the impact of surges on the 72-inch connecting pipe. There should be no unusually large pressures occurring in this pipe.

5. Interim Condition

The interim condition model represented the system which was anticipated to be in operation prior to completion of the entire interconnected drainage system. The interim system model consisted of the East Tunnel, the SWI from 16th Street to the Moreland Street drop structure, and about 1600 feet of the Squaw Peak storm drain. A 4 ft. x 4 ft. vertical shaft was assumed at the upstream end (16th Street) to provide ventilation and pressure relief.

The most significant effect of disconnecting the East Tunnel system from the West Tunnel system is an increase in overflow at the time of pressurization. Some increase in transient pressure in the 72-inch SWI pipe was also predicted, but inclusion of the Squaw Peak storm drain reduced the magnitude of the pressure increase. As a result of this modeling, the design pressure head for the SWI between 16th Street and 19th Street was reduced to 60 feet.

Unlike the final system, the interim system will pressurize at the upstream end first, at about t=120 min. The downstream end of the East Tunnel will pressurize at t=125 min., with the surge front reaching the Moreland Street drop structure at t=145 min. The entire interim system will be pressurized by t=148 min. The maximum head in the East Tunnel remained about the same as predicted in previous model runs.

D. Sensitivity Analysis

1. Moving Storms

Additional modeling was performed to evaluate the response of the I-10 Inner Loop drainage system to moving storms. Hydrographs were supplied by HNTB for runoff due to 50-year storms moving from east to west and moving from west to east. The hydrographs were used as inputs to the existing combined system model that was previously used to obtain run CD-9.

a. Storm Moving East to West

Computer run CD-20 modeled the 50-year storm moving from east to west across the drainage area. In this case, outflow from the East Tunnel started about one hour earlier than outflow from the West Tunnel.

The maximum hydraulic heads reached in the East Tunnel were about the same as the corresponding values computed for the stationary storm. Maximum heads in the West Tunnel were somewhat lower than values for the stationary storm. The culvert between 13th Avenue and 7th Avenue remained unpressurized during the entire simulation.

b. Storm Moving West to East

Run CD-21 modeled the 50-year storm moving from west to east. Outflow from the West Tunnel began about 80 minutes before outflow from the East Tunnel. The maximum heads computed in both tunnels were similar to those in the stationary storm. The culvert from 13th Avenue to 7th Avenue was pressurized, and the surge relief pond at 13th Avenue was fully utilized.

2. Localized (Concentrated) Storm

The inflow hydrograph for the concentrated storm is shown in Figure 6-17. This hydrograph was input to the model at the Moreland Street drop structure. In this case, only the East Tunnel will carry storm water.

Only the East Tunnel between the Moreland drop and the exit was pressurized. The pressurization occurred very gradually resulting in no pressure overshoot or surge pressure. Only hydrostatic pressure is generated in this case.

In summary, the system's response to each of the two moving storms is milder than its response to the corresponding stationary storm. Therefore, the previously recommended surge relief structures are adequate for moving and stationary storms. The total inflow from the localized storm does not cause significant transient problems.

E. Recommended System

1. 13th Avenue Surge Pond

The pond at 13th Avenue should have a minimum surface area of 5000 sq. ft. The culvert should have a sufficient opening to this pond in order to pass backflow of 500 cfs without significant head loss. The capacity of the pond is estimated at 25,000 cu. ft.

2. 7th Avenue Drop Structure

An adequately ventilated surge relief structure of at least 700 sq. ft. is required at the 7th Avenue drop structure. The culvert between 13th Avenue and 7th Avenue should be of uniform size and not less than 10 ft. by 12 ft.

Drainage Area = 6.5 sq. mi.; 32nd Street to Old Cross-Cut Canal, Arizona Canal to Papago East

Hour	Q (cfs)
2.0	0
2.1	0
2.2	0
2.3	18
2.4	153
2.5	436
2.6	744
2.7	1,035
2.8	1,282
2.9	1,454
3.0	1,534
3.1	1,528
3.2	1,455
3.3	1,335
3.4	1,186
3.5	1,026
3.6	864
3.7	700
3.8	523
3.9	337
4.0	169
4.1	67
4.2	32
4.3	15
4.4	5
4.5	1
5.0	0

Figure 6-17 Hydrograph for Concentrated Storm

3. Moreland Street Drop Structure/Surge Basin

A portion of the interceptor east of the 21st Street drop structure may be utilized as the East Tunnel surge tank. The interceptor structure is composed of two 12 ft. by 12 ft. RCB culverts. The double RCB culverts should be ventilated at the upstream ends to evacuate air at the rate of 600 cfs each. The drop structure should not be smaller than the connecting tunnel.

4. Additional Recommendations

a. The 2nd Street and 21st Street systems should be interconnected by a pipe not less than 7.5 feet in diameter.

b. Since the tunnel system must carry backflows as well as normal flows, it should be designed to do so.

c. The tunnel system should be provided with adequate air release devices. The amount and location of air releases recommended are reported in the next section.

d. The design head varies from location to location and, to some extent, depends on the storm size.

e. When Papago East is fully used in the future to convey water, its effectiveness as a surge relief structure should be reevaluated.

F. Air Release Requirements

An air pocket in a pipe line is known to cause undesirable hydraulic conditions. For this reason, most recently constructed drop structures for sewer systems are designed to automatically remove the entrained air that enters the sewer. The drop structures designed for the I-10 Inner Loop drain tunnels are all assumed to have the standard air removal feature mentioned above.

This drainage system is somewhat unique in that the outlets will be pressurized and sealed off to air flow before any outflow can occur. This means air must travel against the flow of water and be released somewhere upstream in the system while the tunnel is being filled. Special air release (and to a lesser extent, air supply) features are needed for this system because regular drop structures are not designed to handle this type of air release.

The air release rate requirement was computed in runs CD-9 through CD-14. These runs were made in conjunction with other studies using the previously determined design tunnel profile and dimensions. Each run uses a particular set of design inflow hydrographs. These design hydrographs were based on the conditions listed in Table 6-6 and were furnished by HNTB.

TABLE 6-6

HYDROGRAPHS USED FOR AIR FLOW COMPUTATION

Run No.	Storm, years	Papago East
CD- 9	50	With
CD-10	25	With
CD-11	10	With
CD-12	10	Without
CD-13	25	Without
CD-14	50	Without

The need for air release starts when the downstream end of either tunnel pressurizes. The positive pressurization surge pushes air from the downstream end towards the upstream end at a rate that sometimes exceeds 2,000 cfs. In addition, the increasing flow depth in the remaining open channel flow zone will also tend to push the air out.

The geometry of the drain tunnels and the manner in which the system pressurizes under design storm conditions suggests five suitable locations for air release. The recommended locations and the maximum outflow (release) and inflow (ventilation) are listed in Table 6-7.

TABLE 6-7

AIR RELASE AND SUPPLY REQUIREMENT

(Negative Value Means Supply)

Station No.	Location	Air Flow (cfs)
1	13th Ave.	1,800 (-500)
8	7th Ave.	1,800
58	Moreland	3,000
80	10th St. drop	1,300
89	2nd St. junction	2,600

The open inlet areas provided at the 7th Avenue, 10th Street and Moreland Street drop structures should be more than adequate to handle required air releases. Air release velocities would be on the order of a maximum of 7.4 fps (5.1 miles/hr) at 10th Street, to a minimum of 0.6 fps (0.4 miles/hr) at Moreland Street.

The 7th Avenue and 10th Street structures will have a deaeration chamber and air vent as modeled in the SAFHL studies. Surge tests at SAFHL showed a tendency for a surge to push air and water through a horizontal chamber above the false crown, up the vertical air vent, and out the top of the model. This problem in the model was corrected by cutting off the air divider wall below

the top of the structure and providing a hood at the top to deflect surges back into the shaft. These recommended features have been included in the design of the 7th Avenue and 10th Street drop structures.

The preliminary recommendation for air release control of the 2nd Street junction chamber was to provide three openings, with a total area of 90 square feet, on top of the junction chamber.

To preclude freeway inundation during intermittent air releases in the form of air-water spouts, the three openings are to be extended to the finished ground surface.

This recommendation was implemented and protective frame with grates is provided to cover the openings at the top of the shafts.

IV. STORM WATER INTERCEPTOR DESIGN

A. Description

The Storm Water Interceptor (SWI) is a system of reinforced concrete pipes and reinforced concrete box culverts conveying runoff from the off-site surface drainage collection systems to the outfall tunnels. The SWI also collects the flows in existing and proposed City of Phoenix storm drains intersecting the freeway corridor, and receives the discharges from the pump station at 16th Street.

The SWI is divided into three segments, each terminating at a drop structure discharging into the tunnel system. Segment 1 consists of a 10 ft. by 12 ft. box culvert running from 13th Avenue east to 7th Avenue, then north to the 7th Avenue drop structure just south of Culver Street. Segment 2 includes 72-inch pipe flowing west from 19th Street to 16th Street, 90-inch pipe from 16th to 15th Streets, 96-inch pipe from 15th

Street to 12th Street, and 10 ft. by 10 ft. box culvert from there to the 10th Street drop structure. The 7th Avenue and 10th Street drop structures discharge into the 14 ft. diameter North Tunnel. Segment 3 consists of 72-inch pipe from 19th Street east to the junction with the Squaw Peak Parkway storm drain and 10 ft. by 10 ft. box culvert from that junction to the Moreland Street drop structure, which discharges into the 21 ft. diameter East Tunnel. Segment 3 also includes a portion of the Squaw Peak Parkway and Papago East Freeway storm drains.

Segments 2 and 3 of the SWI are connected at 19th Street. The connection is a design criterion resulting from the Transient Wave Study Report. The West Tunnel system acts as a surge relief structure for the East Tunnel system. Hence, surges resulting from transient waves are reduced without construction of a large surge relief structure. It was decided to construct the Moreland drop structure as an added measure of safety. As recommended by Dr. Song in March of 1985, minimum size for the interconnecting pipes is 72 inches in diameter.

B. Design Criteria

The following criteria were used in the design and sizing of the SWI. Minimum sizes recommended by Dr. Song for the SWI are as follows:

- Segment 1: 10 feet X 12 feet reinforced concrete box (RCB) culvert.
- Segment 2 and 3: 72-inch diameter reinforced concrete pipe (RCP).

Minimum design hydrostatic pressures for the SWI resulting from transient conditions, are as follows:

Segment 1	42 feet of water
Segments 2 and 3	
10th Street to 12th Street	52 feet of water
12th Street to Papago East Surge Basin	60 feet of water

The following equations were used in the computation of head loss in the conduit.

- Conduit Friction Loss - Manning's equation with Manning's "n" of 0.012 for RCP and 0.015 for RCB.

- Transition Loss:

For velocities which increase in direction of flow,

$$h_t = 0.1 \left[\frac{V_2^{\,2} - V_1^{\,2}}{2g} \right]$$

For velocities which decreases in direction of flow,

$$h_t = 0.2 \left[\frac{V_1^{\,2} - V_2^{\,2}}{2g} \right]$$

- Rate of convergence in the transition shall not exceed 5° 45′. Subscripts 1 and 2 designate upstream and downstream reaches, respectively.

Junction Loss:

a. Junction loss with pressure flow where $A_1 = A_2$ and friction loss is negligible:

$$h_j = \frac{V_2^2 - V_1^2}{2g} - \frac{2A_3}{A_2} \cdot \frac{V_3^2}{2g} \cos \theta$$

Subscripts 1 and 2 designate upstream and downstream reaches from the junction, respectively. Subscript 3 designates the side entrance conduit. Θ designates side entrance conduit angle.

b. Junction loss with pressure flow where A_1 does not equal A_2 is estimated using either A_1 or A_2.

Curve Bend Loss:

$$h_{cb} = 0.20 \frac{V^2}{2g} \sqrt{\frac{\Delta}{90°}}$$

Δ designates the degree of curvature of the bend. This equation is valid for Δ not exceeding 90°.

Angle Bend Loss:

$$h_{gb} = 0.0033\ \theta \frac{V^2}{2g}$$

Θ designates the degree of deflection of the bend. Maximum Θ was 6°.

The hydraulic grade line under the steady-state design flow condition (50-year storm frequency) is generally below the crown of the conduit in the Storm Water Interceptor. In a few sections, the hydraulic grade line is slightly above the crown of the conduit. Maximum distance between manholes along the SWI is 1,000 feet.

C. Horizontal Alignment

The alignment of the SWI system is dictated by the following elements:

- Proposed highways and ramps
- Proposed surface roadway improvements
- ADOT right-of-way limits
- Proposed highway structures, i.e., bridges, piers, and pump stations
- Proposed off-site storm water collector inlets
- Existing utilities

1. **Segment #1 Alignment**

The SWI Segment 1 begins at the surge pond west of 11th Avenue. From 11th Avenue to 7th Avenue, the alignment of Segment 1 parallels the monument line of Moreland Street. The 10′ x 12′ RCB is underneath the south sidewalk on Moreland Street with an offset of 14.50 feet from the monument line. A series of twelve side inlet catch basins along the street curb above the RCB direct street flows into the SWI. The alignment of the SWI along 7th Avenue parallels the monument line on 7th Avenue with a west offset of 68 feet. The 68-foot offset was determined on the basis of the available space between the street curb and the right-of-way. The outlet from the 7th Avenue drop structure curves to the east to match the North Tunnel alignment.

2. **Segment #2 Alignment**

Between 10th and 12th Street, the SWI Segment 2 alignment is parallel to the monument line of Culver Street with a south offset of 46.33 feet. In this reach, the proposed 10′ x 10′ RCB also serves as a retaining wall along freeway ramp 7S-A. From 12th Street to a point approximately 220 feet west of 14th Street, the offset distance changes from 46.33 feet to 21.50 feet due to the location of freeway ramp 16-A.

Beginning at 14th Street, the monument line of Culver Street forms a reverse curve and the SWI alignment follows the reverse curve accordingly. At SWI Station 39+39.96 where the reverse curve ends, the alignment is parallel to the monument line again with a south offset of 28.5 feet until reaching 16th Street. From 17th Street to 19th Street, the offset distance is a constant 45 feet south of the monument line. The offset distances between the monument line and the SWI alignment were determined during various progress meetings in the course of the preliminary design.

3. **Segment #3 Alignment**

Five alternative alignments were studied for this segment of the SWI. The following criteria were considered in developing the alternatives:

1. The northbound and southbound ramps to McDowell Road, which are 20-foot deep depressed sections.

2. A conceptual layout of the fully directional interchange structures.

3. A total of ten local and freeway collector systems are to be connected from the proposed ramps to the storm water system.

4. The double 9′ x 12′ box culverts in the Papago East are to be extended to the north side of the corridor in the future, which will require an additional 320 feet of the double box culvert. These box culvert sizes were changed to 9′ x 10′ in later designs of the Papago East storm drain. The facility was constructed as an 11′ x 10′ RCB to the north side of the corridor then transitioned to a 10′ x 10′ RCB.

5. The Squaw Peak storm drain was assumed to be 96-inch diameter pipe as determined by the City of Phoenix. The 50-year flow rate based on a hydraulic grade line parallel to the ground slope was estimated to be 530 cfs. The City of Phoenix indicated later that the Squaw Peak storm drain would be 108-inch diameter RCP.

The evaluation of the five alternatives considered ease of construction in relation to existing and proposed highway structures, interference with the proposed interchange features, hydraulic performance and construction cost.

Based on these factors, the selected configuration for the SWI through the Squaw Peak area is as follows: The Storm Water Interceptor connects to the Squaw Peak Parkway storm drain at a junction structure located east of the future S-N ramp to McDowell Road. A 10 foot by 10 foot RCB leads from the junction structure to the Moreland Street drop structure junction chamber. The Papago East Freeway storm drain extends from the Moreland Street drop structure to the east along the south side of the Papago East corridor.

D. Vertical Alignment

Vertical alignment of the SWI system is dictated by the following elements:

- Outfall tunnel invert elevations
- Roadway grades
- Storm water collection system elevations
- Future roads and ramps
- Existing storm sewers
- Existing utilities

1. **Segment #1 Alignment**

The invert elevations for Segment 1 were set based on Dr. Song's recommendations. The depth of cover for the 10' x 12' RCB varies from 1.5 to 6 feet. Utility conflicts are resolved in this segment by rerouting telephone and gas lines.

2. **Segment #2 Alignment**

Invert elevations of the SWI at some locations are governed by the connections to the storm water collection systems. The design elevation for a proposed 96-inch storm water collector in 12th Street requires the SWI to be lowered from the minimum elevation by an additional 2 feet. The depth of the cover in this segment varies from 6 to 12 feet. The extra depth over the SWI in Segment 2 is caused by the following:

- Connection of local collector pipes.
- Avoidance of the existing sewer lines in 10th Street
- Maintenance of a minimum cover of 3 feet over the RCB at the face of the freeway cut slope.

No utility conflict was expected for the design grade of the SWI in this segment, however during construction a 48 inch water main was found to be in conflict at 19th Street. The cross-section was transitioned to a "squashed" box culvert under the water main then transitioned back to the circular cross-section. This modified cross-section will not adversely affect the system's performance.

3. **Segment #3 Alignment**

Depth of the SWI in Segment 3 is governed by the elevation of freeway ramp S-N and clearance between pier footings supporting the future structures N-E, W-N and HOV.

The invert elevation at the north end of the Squaw Peak Parkway storm drain section included in the SWI design was provided by the City of Phoenix. The invert elevation of the Papago East Freeway storm drain is controlled by the clearance at an existing sanitary sewer along 24th Street.

E. Interference with Structures

Five locations along the SWI alignment were identified as having potential conflicts with planned highway structures. They are the proposed 9th Avenue pedestrian overpass, the 12th Street and 16th Street vehicular overpasses, the 18th Street pedestrian underpass, and the bridges at the I-10/Squaw Peak/E. Papago T.I.

The 9th Avenue pedestrian overpass is a proposed four-span pre-stressed concrete box girder bridge passing over the I-10 freeway and two 7th Avenue ramps, A & D. The 10′ x 12′ RCB conflicted with the location of the north abutment of the bridge structure. Through additional study, it was concluded that the bridge's north abutment could rest on top of the SWI.

The 12th Street overpass is a two-span concrete box girder bridge. Two 30-foot long wing walls are planned at each end of and parallel to the bridge. The 96-inch RCP in Segment 2 is located approximately 25 feet north of the north abutment. The pipe is, therefore, located underneath the two wing walls. To avoid structural interference, the wing walls above the RCP will have to span horizontally without a footing. The footing underneath the wall will resume outside the RCP interference range.

The 16th Street vehicular overpass and the 18th Street pedestrian overpass were checked for potential clearance problems. No interference with the SWI is anticipated at either of these locations.

A curvilinear alignment is planned for Segment 3 of the SWI and the Squaw Peak Parkway storm drain. This will maximize the clearance at bridge abutments and provide necessary clearance at Lanes W-E and E-W.

F. Conduit Sizing

Conduits for all three segments of the SWI were sized based on the design criteria and parameters previously described. 108 inches was the maximum diameter considered for reinforced concrete pipe. Where larger capacities were required, an appropriately sized box culvert was specified. In some cases, conduit sizes in excess of hydraulic capacity requirements were specified in order to effectively control surge pressures.

Initial design of the SWI produced the following conduit sizes, which were used in the final design:

- Segment 1

11th Avenue to 7th Avenue Drop Structure	10′ X 12′ RCB

- Segment 2

10th Street Drop Structure to 12th Street	10′x10′ RCB
12th Street to 15th Street	96-inch RCP
15th Street to 16th Street	90-inch RCP
16th Street to 19th Street	72-inch RCP

- Segment 3

19th Street to Squaw Peak Junction	72-inch RCP
Squaw Peak Junction to Moreland Street Drop Structure	10′x10′ RCB

In segments 2 and 3, the planned RCP conduit was compared to an alternative consisting of RCB with an equal conveyance factor. The Manning roughness coefficient (n) was assumed to be 0.012 for both the RCP and the RCB in this comparison. The equivalent sizes were determined to be 8-feet x 6-feet RCB for 90-inch RCP and 8-feet x 6-feet 8 inches RCB for 96-inch RCP. Both the RCP and RCB were designed to withstand anticipated surge pressures of 100 feet of water.

The analysis was based on construction cost, ease of maintenance and flow conveyance factors. Construction cost for the RCB were estimated to be over twice that of the RCP alternative. RCP will have a higher velocity than RCB at a given flow rate, due to the smaller hydraulic radius of the RCP. This gives the RCP better self-cleaning characteristics than the RCB. The RCB's flat bottom would allow better access for cleaning vehicles, but maintenance activities requiring vehicles would be rare. For these reasons, RCP was recommended as the backbone of the SWI in Segments 2 and 3.

G. Local Collector System

PRC Engineering included local collector systems with the design of the SWI at two locations. Catch basins were detailed along the south side of Moreland Street between 11th Avenue and 7th Avenue. Each catch basin has a capacity of 40 cfs without flooding the street based on information provided by BRW, Inc.

The second local collector system designed with the SWI runs along the Inner Loop corridor between Roosevelt Street and Fillmore Street. Concrete catch basins are located in six depressions which collect storm water from the street. Flows from the catch basins are routed through pipes which range in

size from 24 to 60 inches to a junction structure. Discharge from the junction structure enters the East Tunnel at the Pierce Street drop shaft.

V. TUNNEL DESIGN AND CONSTRUCTION

Many of the design parameters for the three drainage tunnels were determined during the computer modeling described in previous sections of this chapter. These parameters include alignment, hydraulic gradient, design flows and design pressures. This section covers changes and additions made to the design criteria during the final design process. Tunnel construction is also summarized.

A. Final Design Criteria

1. Hydraulic Design

Modeling done for the VE Report was based on a Manning's "n" of 0.015 and a hydraulic gradient of 0.133%. Manning's "n" value was subsequently reduced to 0.013 to reflect the increased hydraulic efficiency of the cast-in-place final linings designed for the tunnels. Design slopes for the tunnels were finalized at the following values:

East Tunnel: 0.1226%
West Tunnel: 0.1337%
North Tunnel -
West Branch: 0.2450%
East Branch: 0.1596%

Design hydrostatic pressure head is 100 feet of water as determined in the modeling process. This is adequate to withstand transient surge pressures as well as pressures

encountered during normal tunnel operation. These pressures were determined in the computer modeling performed by Dr. Song, described in previous sections of this chapter.

2. Structural Design

A flexible lining concept was used in the tunnel design. This design procedure consists of four separate steps:

a. Provide for the expected ring load.
b. Provide for the anticpated distortions due to bending.
c. Give consideration to the possibility of buckling.
d. Make allowance for significant external conditions not covered in a-c above.

a. Ring Load

The ring load in the lining of a tunnel is generally smaller than that corresponding to the overburden pressure. Therefore, the design ring load was taken to be γz, with γ = total unit weight of the soil and z = depth to the axis of the tunnel. This design load provides a satisfactory allowance for the influence of adjacent foundations or construction.

b. Bending

In order to anticipate distortions due to bending, an estimate was made of the change in diameter likely to occur if a perfectly flexible lining of the same shape as the tunnel were installed in similar soil conditions. Field data shows that the change in diameter of a tunnel lining rarely exceeds 0.5 percent. This change in diameter is acceptable with respect to non-structural requirements. A cast-in-place

lining can change shape from circular to elliptical by an amount corresponding to this deformation without overstressing. This type of inner lining was used in all 3 tunnels.

c. Buckling

Buckling has been noted in tunnels where supports twisted or were irregularly blocked. However, there is no report of a failure due to buckling of a tunnel lining when the lining was everywhere in contact with soil or grout. Therefore, provision was made to fill the overmine space between the outer tunnel lining and the surrounding soil with either pea gravel or grout. This also helps reduce land subsidence due to the tunneling operation.

d. Additional External Conditions

Due to the nature of the tunneling operation, the outer lining was designed with ample reserve strength for shield jacking loads and for asymmetrical loads which were deemed likely at the heading itself. Provisions were made to minimize subsidence effects at ground suface.

B. Tunnel Construction

The Inner Loop drainage tunnel system is one of the largest in the nation constructed in soft ground. A study entitled "Geotechnical and Engineering Assessment of Tunnel Feasibility" was completed by HNTB in March, 1983 to identify and isolate geologic and engineering parameters pertinent to tunnel construction in the Phoenix area.

Tunnels were constructed entirely within the sand-gravel-cobble (SGC) deposits which are predominant in the Phoenix area. These deposits exhibit great variation in cementation, ranging from

cohesionless (running ground) to well-cemented sands and gravels (firm ground). As in any soft-ground tunneling operation, surface settling was a possibility. Therefore, a complete film and video record was made of ground conditions along the tunnel alignments before construction commenced.

Drop shaft locations were determined during the tunnel design process. Tunnel blockouts were provided for drop shaft connections. Drop shafts were designed and built under contracts separate from the tunnel contracts. Drop shafts are covered in Chapter 7 of this report.

1. Construction Methods

The "shield" method was used for tunnel construction. It was chosen because it is adaptable to the variety of ground conditions in the area, presents minimum risk for surface ground settlement, and is economical. A tunnel boring machine, the shield, was chosen by the contractor. The shield temporarily braces the surrounding material while excavating ahead. Outer lining segments are placed between the tail section and the excavation wall. The machine is then jacked ahead, and the outer lining is exposed to ground loads. Each of these cycles covers approximately 4 ft. of advance.

The inner tunnel lining was built with fixed-form, cast-in-place techniques. This produced a smooth finish, capable of withstanding the design internal hydrostatic pressures without degradation. The inner lining has an 11-inch wall thickness. It is continuous, except at locations to provide for drop shaft connections.

2. East Tunnel Slope

The design slope of the East Tunnel was finalized at 0.1226%. However, during construction the slope at the East Tunnel was inadvertently increased between segment Station 125+10 and segment Station 129+02. In order to correct this problem, the tunnel was constructed at a slightly shallower slope for approximately 300 feet. The tunnel was restored to the design level after Station 132+26.

The result of this variance from the design slope was a sag of as much as 18-21 inches in the tunnel. This depression was filled in order to maintain a uniform slope throughout the length of the tunnel. A slight decrease of cross-sectional area occurs due to the fill in this portion of the East Tunnel.

Dr. Song analyzed the effect of this variance from design conditions with the same hydraulic transient model previously used to design the tunnel system. As a result of his analysis, a 12-inch diameter air vent was recommended and installed at Station 125+10 to relieve trapped air. This vent reaches the ground surface in the east embankment fill of Ramp WJ-D, just south of the gore area between the ramp and Van Buren Street, at approximately ramp Station 13+75. No significant increase in transient pressure is anticipated as a result of the variance from design slope and subsequent corrections.

CHAPTER 7 – APPURTENANT STRUCTURE DESIGN

SYNOPSIS

Chapter 7 describes the development of drop shaft and drop structure design. The 2nd Street junction structure is also included.

I. Drop Structure Design

Three drop structures are required for the I-10 Inner Loop drainage system. Drop structures are required at 7th Avenue and 10th Street to convey flows from SWI segments 1 and 2 into the east and west branches of the North Tunnel. The third drop structure is required at Moreland Street to carry the flow from SWI segment 3 into the East Tunnel. It also conveys flows from the East Papago Storm Drain to the East Tunnel.

The drop structures convey the flows from the collector system near the ground surface to the outfall tunnels which are approximately 60 feet below the ground surface. The structures must dissipate the kinetic energy of the falling water in a manner that will result in a relatively smooth flow into the tunnel. If the kinetic energy is not adequately dissipated within the drop structure, serious damage to the structure and the outfall tunnels could occur. High velocity flow and/or high impact forces could cause severe erosion of the drop structure concrete. Furthermore, uncontrolled flow without energy dissipation could result in large-scale surges within the outfall tunnel. Air-entraining characteristics of the drop structures must also be considered. If large amounts of entrained air enter the outfall tunnel, hydrodynamic forces may cause damage to the tunnel. Entrained air may also escape through the drop shafts and cause loud noise and water spouts at the ground surface.

The system is unpressurized during the early and late periods of the design storm and pressurized during the middle period of the design storm. The major considerations in the design for the unpressurized

case are energy dissipation and the amount of air entrainment. Under pressurized flow, the major considerations are the head loss through the structure and its ability to convey the peak flow. Peak flow conveyance should not pose a problem since the 7th Avenue drop structure is oversized to accommodate surge pressures and the diameters of the other two drop structures are the same as the diameters of their respective connecting outfall tunnels.

The peak design flow during pressurized flow is the peak flow of the design inflow hydrograph. However, the peak design flow during open-channel flow can only be established on the basis of engineering judgment, since the time at which flow in the drop structures changes from unpressurized to pressurized cannot be accurately calculated. In the case of the I-10 drainage system, computer simulations calculated that time as accurately as possible; however, the results must be considered approximate. More importantly, the simulations only consider the 50-year and 10-year storm events. A more frequent, localized, high intensity rainfall could result in an inflow hydrograph with a peak open-channel flow greater than the peak open-channel flow associated with the 50-year event. With these uncertainties it was considered reasonable to assume peak inflow hydrograph flow to be peak unpressurized flow into the drop structures.

A. Design Criteria

The following are the minimum requirements for the three drop structures:

o <u>7th Avenue Drop</u>

- Minimum diameter (to accommodate surge)	30 ft.
- Peak discharge during 50-year storm	1,032 cfs
- Drop height	20.18 ft.

- **10th Street Drop**

- Minimum diameter (equal to tunnel diameter)	14 ft.
- Peak discharge during 50-year storm	1,326 cfs
- Drop height	30.30 ft.

- **Moreland Street Drop**

- Minimum diameter (equal to tunnel diameter)	21 ft.
- Peak discharge during 50-year storm	2,573 cfs
- Drop height	29.98 ft.

The hydraulic design criteria were established as follows:

- The structure should operate adequately under unpressurized conditions over a range of flows from zero to the peak unpressurized flow.

- The energy associated with the drop should be effectively dissipated upstream of the tunnel inlet and within the drop structure.

- Head loss through the drop structure should not adversely affect the system's flow carrying capacity.

- Control of air entrainment and adequate venting should be accomplished within the drop structure.

- The structure must safely accommodate the surges identified by computerized modeling of the system.

- The structure should be cost effective for both construction and maintenance.

- The structure should be constructed within available land area.

B. Drop Structure Configuration

1. Alternatives

Six alternative types of drop structures were considered: simple drop structure, refined drop structure, morning-glory spillway, baffled chute spillway, ogee spillway and a cascade type of structure. A preliminary screening eliminated the obviously unsuitable alternatives. The following is a brief summary of the results:

Simple Drop Structure. This structure consists of a straight, reinforced concrete shaft connected directly to the collector and the tunnel. It is very attractive from the point of view of constructability and cost. However, the hydraulics of such a structure are unacceptable. The flow into the tunnel would be unstable and require that a relatively long length of the tunnel lining be more heavily reinforced to take the loading which could exist. This type of structure would also introduce large amounts of entrained air into the tunnel flow which could possibly result in reduced flow capacity. No further consideration was given to this alternative.

Refined Drop Structure. This structure consists of a shaft connected directly to the collector similar to the simple drop structure, but with a deaeration chamber between the shaft and the tunnel. The deaeration chamber minimizes entrained air from entering the tunnel and provides a transition section in which the flow can stabilize prior to entering the tunnel. This alternative was considered to be an acceptable solution and was retained for further study.

Morning-Glory Spillway. This structure consists of a circular inlet weir, a vertical shaft and a 90° elbow connecting the shaft to the tunnel. This type of structure does not have good energy dissipating characteristics and could possibly result in siphon flow within the tunnel. This alternative was discarded.

Baffled Chute Spillway. This type of structure consists of an inclined invert slab with a series of large baffle blocks to retard flow velocity and dissipate energy. It would be an effective energy dissipater during unpressurized flow; however, it would also have a high head loss during pressurized flow. Therefore, this alternative was discarded.

Ogee Spillway. This type of structure consists of a conventional ogee spillway with a chute and stilling basin. The hydraulics of such a scheme are considered to be acceptable. This type of structure was retained for further consideration.

Cascade Type Structure. This type of structure consists of a series of steps which dissipate energy. It was found that this type of structure would require regular maintenance and would have high head loss under pressurized flow. This alternative was discarded.

2. **Evaluation of Final Alternatives**

The two retained alternatives (refined drop structure and ogee spillway) were further evaluated to determine the best possible configuration for the three drop structures.

From the standpoint of hydraulic operation, the ogee spillway structure is considered to be acceptable over the full range of flows expected as a result of the 50-year inflow hydrograph. This is true of both the unpressurized and pressurized flow regimes. The only concern with the hydraulics of this structure is the velocity during unpressurized flow, which will approach 50 fps. While this velocity does not present a serious concern with regard to cavitation potential for a well-constructed structure, it will require special consideration with regard to the surface finish to minimize the potential for erosion and/or cavitation. Close tolerances must be specified to keep offsets, protrusions

and other surface irregularities within acceptable limits. Such a requirement will increase the cost of the structure slightly.

The refined drop structure is relatively new compared to an ogee spillway structure. There has been a significant amount of model testing by the St. Anthony Falls Hydraulic Laboratory (SAFL) on this type of structure.

Previous SAFL studies have shown that one advantage the refined drop structure alternative has over the ogee spillway alternative is the freedom for the interceptor system to enter the drop shaft at almost any angle without requiring extensive modifications to the structure. For example, accommodating the interceptor system entrance to the drop shaft at 90° to the outfall tunnel versus 180° would merely require rotating the diaphragm wall 90°. The air vent would remain in the same position. This modification would not adversely affect the hydraulic performance of the drop structure.

Based on the model tests and theoretical considerations, it was concluded that the refined drop structure with the addition of a sump has excellent hydraulic operating characteristics over a wide range of flows, even for flows higher than the design flow. This should be a very low maintenance structure except for dewatering the sump.

The refined drop structure without a sump has good hydraulic operating characteristics over a wide range of flows without the need for dewatering. The only concerns with this structure are the long-term integrity of the weir, which is considered to be a minor concern, and the possibility of sediment deposition upstream of the weir, which could adversely affect the performance of the weir as an energy dissipater if the turbulent flows do not scour the deposits during the early stages of a flood. Such sediment deposits would also change the effective height of the weir and modify its hydraulic characteristics.

Based on the initial design studies, the following conclusions were reached:

- Three types of drop structures were considered to be reasonable alternatives for this project. These were the refined drop structure with sump, the refined drop structure without sump, and the ogee spillway structure.

- Based on hydraulic performance, the refined drop structure with sump was ranked best with the other two ranked a very close second and third.

- Based on risk to the structure and to the tunnel, the refined drop structure with sump was ranked best with the other two ranked a very close second and third.

- The drop structure without sump was ranked best based on anticipated maintenance, assuming the sumps of the other two alternatives will require post-storm dewatering. However, the need for post-storm dewatering of sumps to maintain them in a normally dry state is questionable and should be studied further.

- Any of the three alternatives will function adequately assuming that sufficient attention is given to details during final design and construction.

The following recommendations were made as a result of the initial design phase investigations.

- The refined drop structure with sump was recommended for the three drop structures. While there is no great difference in

cost, performance or risk between the alternatives, the drop structure with sump has been shown to be technically superior in meeting the design criteria. Its cost is considered equal to the other two alternatives considering the level of confidence in the present cost estimates. Figure 7-1 shows a sketch of the refined drop structure layout.

- The need to maintain sumps normally dry should be studied further to confirm the need for such a criterion.

3. Summary of Design Requirements

Arrangement and size of the recommended drop structure configurations have been affected by the computer studies performed by Dr. Song, presented in the report entitled "Mathematical Modeling of I-10 Inner Loop Drainage System." Those studies indicate that, for the conditions of tailwater at a level below the invert of tunnel outlets and inflow to the system resulting from a storm, the combined system of north, east and west drainage tunnels will be pressurized and sealed off to air flow before any outflow of water can occur. These studies of the design storm resulted in recommendations for pressure control by surge relief structures as follows:

1. **7th Avenue Drop Structure:** shaft with an area of at least 700 square feet, adequately ventilated.

2. **Drop Structures at 10th Street and Moreland Street:** shaft areas equal to the area of the connecting outfall tunnel, 14 and 21 feet diameter, respectively.

3. **Storm Water Interceptor:** between 11th Avenue and 7th Avenue and at East Tunnel for the future Papago East interceptor.

4. **Pond:** at 11th Avenue.

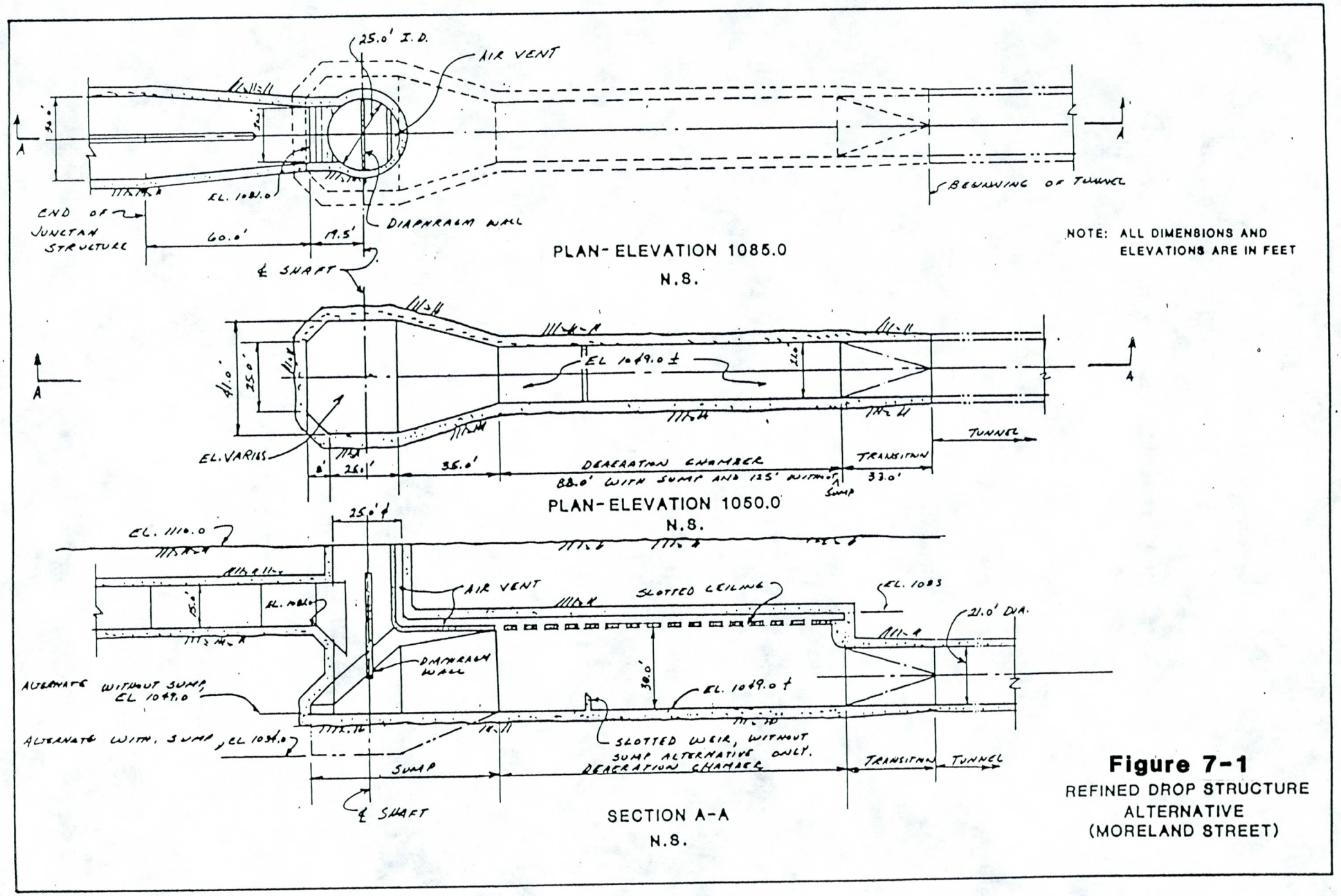


Figure 7-1
REFINED DROP STRUCTURE ALTERNATIVE (MORELAND STREET)

Based on the above recommendations by Dr. Song, the following provisions should be included in the drop structure design:

1. An interceptor box outlet leading to a free flow jet trajectory to the shaft.

2. A vertical diaphragm wall within the shaft for the drop structures at 7th Avenue and Moreland Street to absorb and deflect the impact of the jet. A diaphragm wall is not required for the 10th Street drop structure because the diameter of the shaft is only 15 feet. Therefore, the jet entering the shaft will hit the backwall of the shaft, allowing thorough dispersion-aeration of the falling jet through the shaft.

3. Vertical shaft diameters of 30 feet at 7th Avenue, 15 feet at 10th Street and 25 feet at Moreland Street.

4. Deaeration chamber connecting the sump and drop structure outlet transition.

5. Transition from the outlet of the drop structure to the main tunnel.

6. An air vent within the shaft, separated by a slotted divider wall.

7. Suitable slot deflectors located along the drop shaft side of the divider wall to deflect the flow of the falling jet.

8. An air chamber along the soffit of the deaeration chamber separated by a slotted air diffuser.

9. Suitable slot openings in the air diffuser to draw air from the deaeration chamber into the shaft air vent, for continuous air circulation.

The principal purpose of the sump is to contain the entrained air and direct it to the deaeration chamber or air vent shaft. Air removal in the sump is the most critical feature protecting against damages and ensuring the safe operation of the drop structure system. The sump area, which has either a flat or depressed floor, also dissipates the kinetic energy of the falling water and directs the flow at reduced velocities to the outfall tunnel.

If the sump must be drained, a small pump would be required at each of the drop structures. It may be possible to mitigate environmental concerns by adding chemicals to the sump periodically. In that case, no sump pumps would be needed.

The deaeration chambers are provided with an air chamber separated by an air-slotted false ceiling. Slot openings along the center part of the ceiling draw air from the chamber into the shaft air vent. Downstream of the slots, the air chamber will fill with trapped air and act as a surge damper. The air chamber will also control air bubbles rising to the crown of the sump.

C. Moreland Street Drop Structure

Based on the initial design review, PRC Engineering prepared an alternative drop structure design for Moreland Street prior to commencement of preliminary design. The alternative structure provided a permanent surge tank at the Moreland Street drop structure, as opposed to the original concept of using the 500-foot double 12′ x 12′ box culvert in the Papago East Freeway drainage system as a temporary surge basin. The alternative drop structure was sized to provide a surface area of 5,000 square feet for surge relief as recommended by Dr. Song in his mathematical modeling report. This required that the walls of the drop structure be opened up to the ground surface and that the structure be lengthened.

Preliminary design for the combined surge tank-drop structure was based on the above considerations. In addition, it was specified that the top of the walls for the Moreland Street Drop Structure be lowered to three feet below the ground surface and that the site be properly graded to provide storage for overflow from the structure during the surge condition. Access was to be provided to the bottom floor and the top of the structure was to be provided with permanent grates.

1. Preliminary Layout

Preliminary layouts and alternative designs were evaluated for the surge tank drop structure considering the surge relief requirement, hydraulic performance and construction alternatives. The surge relief requirement was met by maintaining an open structure with vertical walls from the floor to the ground surface. Evaluations of hydraulic performance were based on the results of the SAFL model study.

2. Final Layout

A flat floor structure has been adopted for the Moreland Street drop structure based on the SAFL test results for Models DS3 R9 and DS3 R10. The walls of the structure have been opened to the surface and the deaeration chamber has been lengthened to 159 feet in order to meet the requirements for surge relief. The total surface area provided is 5,335 square feet.

Below elevation 1075, the structure is similar to SAFL Model DS3 R10 except for a larger boot area and longer deaeration chamber. The boot has been retained because SAFL highly recommends it and a review of the report and video tapes indicate that it is very

effective. The area of the boot has been increased over that which was modeled. Dr. Nicolau of PRC Engineering and SAFL were of the opinion that increasing the boot area would be desirable with the sump at Elevation 1049. However, this was not done during testing since no funds were available to revise the model.

Above elevation 1075, the inlet area is open up to the ground surface. The bottom width of 41 feet is maintained for the full height of the structure. The diaphragm wall has been widened to 41 feet and acts structurally to brace the side walls. Two guide walls are provided extending from the box culvert outlet to the diaphragm. Hydraulically, they will provide almost identical inflow conditions as tested in the model. Structurally, they will serve to stiffen and brace the diaphragm wall.

D. 7th Avenue Drop Structure

The 7th Avenue drop structure is designed for a maximum discharge of 1,032 cubic feet per second. Based on Dr. Song's studies and recommendations, a minimum surface area of 700 square feet shall be provided for surge relief at this location. Site constraints are severe at the location and they have a significant effect on the layout of the structure.

1. Preliminary Layout

The location and length of the drop structure is constrained by the following:

- The alignments of the SWI and North Tunnel intersect at an angle of 82° 52′ 34.5". A curved segment is required to connect the drop structure to the tunnel.

- For best hydraulic performance, it is undesirable to curve any portion of the drop structure. Therefore, a linear alignment must be maintained from the drop structure inlet to the end of the rectangular-to-round transition.

- A minimum open surface area of 700 square feet must be maintained for surge relief at the inlet.

- The open inlet area should not encroach on Moreland alley.

The curved segment required to connect the end of the drop structure to the end of the North Tunnel will be a circular section having a diameter of 14 feet. The minimum acceptable radius of curvature for this segment, based on head loss considerations, is twice the diameter or 28 feet. Thus the length of the curved segment required will be 40.99 feet. Therefore, the length available for linear alignment of the drop structure, without encroaching on Moreland Alley, is limited to a total of 125 feet.

The SAFL model studies showed that a drop structure with or without a sump is hydraulically acceptable. Therefore, both types of structures were evaluated. The required length from the centerline of shaft to the end of the transition for a drop structure without a sump is 130 feet. The controlling dimension is the length required for the hydraulic jump downstream of the slotted weir. The required length for a drop structure with a sump is 119 feet. Initial site constraints made a drop structure with a sump necessary at 7th Avenue. As shown in the SAFL model studies, this type of structure also provides the best hydraulic performance.

2. Final Layout

Since a drop structure either with or without a sump will give acceptable performance, and the difference in cost and risk of damage between the two alternatives is low, PRC prepared final designs for a structure without a sump based on lower maintenance requirements. Additional land was obtained and the necessary 130

feet was made available to use a longer structure. A drop structure without a sump having a rectangular inlet was adopted. A rectangular shaped inlet was chosen because it is narrower than the shaft type and easier to construct.

The structure consists of a 26-foot wide by 13-foot long drop inlet without a sump, a 22-foot long open transition, a 79.5-foot long covered deaeration chamber, a 22-foot long rectangular to round transition and a 41-foot long, 14-foot diameter curved conduit connecting the structure to the tunnel. The open inlet and transition area has vertical walls open to the ground surface and will be covered with grating, with the exception of the portion of the area which infringes on the street connection between Moreland and Culver Streets. For this portion a cantilevered reinforced concrete overhang was provided along with a parapet wall to act as a protection barrier. The remainder of the structure is a buried cut and cover conduit.

3. Hydraulic Design of Final Layout

The hydraulic design of the drop structure is based on SAFL Model DS3 R9 and DS3 R10. All of the recommended features of the models are included in the structure and the dimensions of the structure are proportional to those of the models.

A vertical shaft conveys storm water from the box inlet to the plunge pool. The shaft is trapezoidal in section, 10 feet wide at the inlet and 15 feet wide at the diaphragm wall. The diaphragm wall is 10-3/4 feet from the mouth of the inlet.

The diaphragm wall provides thorough dispersion and aeration of the falling jet of water. Anti-vortex ribs are included on this wall as recommended by the SAFL report. The diaphragm wall also forms the air release and surge relief areas.

The height and location of the slotted weir were proportioned from the model. The weir provides the backwater to cushion the falling jet at the bottom of the shaft. Below the shaft, the plunge pool is widened to 26 feet and includes a 4.5-foot boot area. The effectiveness of the widened pool and boot were demonstrated in the model.

A 14-foot wide deaeration chamber with a slotted false crown will collect released air and return it to the vertical air vent. The length of the chamber and the number of air slots are proportional to the model.

The length of the transition from the 26-foot wide inlet to the 14-foot wide deaeration chamber is based on a convergence angle of 15 degrees as tested in the model. The length of the square to round transition is based on a maximum convergence angle of 7.5 degrees.

E. 10th Street Drop Structure

The 10th Street drop structure is designed for a maximum discharge of 1,326 cubic feet per second. Site constraints govern the location and length of the drop structure as follows:

- The alignments of the SWI and North Tunnel intersect at an angle of 11° 05′ 19.1". A curved segment is required to connect the drop structure to the tunnel. The length of this segment is 9.8 feet.

- For best hydraulic performance, it is undesirable to curve any portion of the drop structure. Therefore, a linear alignment must be maintained from the drop structure inlet to the end of the rectangular to round transition.

- There is a junction between the SWI and a 60 inch diameter storm drain just upstream from the drop structure inlet. In order to establish smooth hydraulic flow conditions into the drop structure, the length between the junction and the centerline of the drop shaft should be maximized by holding the overall length of the drop structure as short as physically possible.

Based on the above constraints, there is approximately 180 feet available for the drop structure from the end of the square to round transition to the junction between the 60 inch diameter storm drain and the SWI. The alternative with the shortest overall length would be preferable.

1. **Preliminary Layout**

The SAFL model studies showed that a drop structure with a sump was the most efficient structure. The studies also showed that a structure without a sump would also give an acceptable hydraulic performance. Therefore, both types of structures were evaluated.

Both structures would have a 15-foot diameter shaft, 14-foot by 18-foot deaeration chamber and 22-foot long square to round transition. Final dimensions for the depth of the sump and length of the transition and deaeration chamber were based on the results of the model studies.

A drop structure with a sump requires a sump depth of 9.4 feet, a 22-foot long transition and 73.5-foot long deaeration chamber. The overall length required from the centerline of the shaft to the end of the square to round transition is 125 feet.

A drop structure without a sump requires an overall length of 139.5 feet. The difference between the structure with and without a sump is the length required for the deaeration chamber. The

structure without a sump requires a 4-foot high slotted weir and an 88-foot long deaeration chamber. The controlling dimension is the length required for the hydraulic jump downstream of the weir.

As mentioned previously, a curved segment is required to connect the drop structure to end of the tunnel. For best hydraulic performance, it is undesirable to have any portion of the drop structure curved. Therefore, a curved 14-foot diameter conduit with a radius of 64.3 feet connects the end of the square to round transition to the tunnel. The length of this segment is 9.8 feet, which increases the overall length for a drop structure with or without a sump to 134.8 and 149.3 feet, respectively.

A drop structure with a sump was recommended for this site for the following reasons:

- It is the most efficient hydraulic structure.
- It best fits the site conditions.
- It maximizes the length between the 60-inch storm drain and the centerline of the drop shaft.
- It is consistent with the type of structure recommended for 7th Avenue.

2. Final Layout

Since either a drop structure with or without a sump will give acceptable performance and the differences in cost and risk of damage between the two alternatives are low, PRC was directed to prepare final designs for a structure without a sump based on the lower maintenance requirements of the structure without a sump.

3. Hydraulic Design of Final Layout

The hydraulic design of the drop structure is based on SAFL Model DS3 R9 and DS3 R10. The dimensions of the structure are proportional to the model.

A 15-foot diameter shaft conveys storm water from the box inlet to the plunge pool. A diaphragm wall as used in the model is not required because the diameter is only 15 feet and the trajectory of the jet entering the shaft will hit the backwall of the shaft. In effect, the air vent wall performs the function of the diaphragm wall in the model. Anti-vortex ribs have been included as recommended.

The height and location of the slotted weir were determined by proportion to the model. The weir provides the backwater to cushion the jet falling through the shaft. Below the shaft, the plunge pool is widened to 26 feet and includes a 5.5-foot boot area. The effectiveness of the widened plunge pool and the boot were demonstrated in the model. The deaeration chamber is 14 feet wide. The length of the chamber and the number of air slots are proportional to the model. The transition length set by the 15 degree convergence angle requirement is the same as the 7th Avenue drop structure.

II. DROP SHAFT DESIGN

In addition to the three drop structures previously discussed, the I-10 Inner Loop drainage system includes 14 drop shafts which convey flows from the storm water collection system to the outfall tunnels. The drop shafts perform a function which is identical to the drop structures; that is, they convey flows from the storm water collector system located near the ground surface to the outfall tunnels located at a significantly lower elevation. This requires the shafts to

dissipate the kinetic energy of the falling water in a safe and cost effective manner. The main differences between the drop structures and drop shafts are as follows:

- The design flows for the drop shafts are significantly smaller than those for the drop structures.

- Each drop shaft discharges flow to the outfall tunnel in intermediate locations along the tunnel whereas the drop structure discharges directly into the upstream end of an outfall tunnel.

Additional research was performed during the preliminary design phase to review existing data and formulate sound design concepts. The following is a summary of that research.

Drop shafts for the disposal of sewage and storm water runoff have been used for many years in conventional sanitary and storm sewer systems. The conventional sewer systems usually involve relatively low flows and relatively high drop heights. Many instances of structural deterioration and damage have been documented over the years due to inadequately estimating the kinetic energy associated with a free fall water jet.

In recent years, many urban and metropolitan areas have considered and in some cases constructed storm water conveyance systems which consist of surface collection systems discharging flow into an underground network of conduits and tunnels similar to the I-10 Inner Loop drainage system. Vertical drop shafts to conduct surface discharges to the underground network have been necessary due to space and economic constraints.

These systems are unique because they deal with higher flows, require higher drop heights and are considerably more expensive to construct and maintain than conventional storm water sewer systems. The I-10 Inner Loop drainage system drop shafts are considered to fall between

those utilized in conventional systems and the sewer tunnel systems with drop heights less than 60 feet and relatively high flows. Because of that, available data concerning both types of systems were reviewed.

Previous laboratory investigations of drop shafts included alternative types with straight and tapered vertical shafts. Shafts were provided with either an elbow or a vortex-type inlet. Energy dissipation at the bottom of the drop was accomplished in deep sumps, with or without impact cups. As a means of investigating the possibility of utilizing the hydraulic jump for energy dissipation after the drop, a 90-degree elbow was utilized at the base of the drop shaft. Design discharges for drop shafts tested were 300 cfs to 900 cfs and drop heights included drops of 62 feet, 100 feet and 130 feet. Subsequent hydraulic investigations have been performed by St. Anthony Falls Hydraulic Laboratory for drop shaft structures with design flow capacities as high as 4,496 cfs and drop heights from 51 feet to 215 feet.

The major conclusions reached after reviewing the available literature are as follows:

1. The principal hydraulic consideration for the development of an efficient drop shaft include the following factors:

    - Capacity of the inlet and drop shaft

    - Effect of energy dissipation at the bottom of the drop and at the entrance to the underground tunnel.

    - Stability of outflow to the tunnel.

    - Air entrainment control with a suitable venting system.

    - Hydrodynamic pressures on the structure.

2. There appears to be unanimous agreement that, when designing a drop shaft which discharges into a tunnel, the primary consideration is the stability of flow as it enters the tunnel. Sigurd H. Anderson in his paper, "Model Studies of Storm-Sewer Drop Shafts", summarizes the problem as follows:

> "Uncontrolled discharge from the sump chamber without deenergization of the flow results in large-scale surges within the lower conduit system. This surging combined with the entrainment of large amounts of air will affect the operation of other drop shafts located downstream. Entrained air will accumulate in the lower interceptor and either flow downstream when excess pressure is reached or vent back through the originating drop shaft. Uncontrolled discharges may also result in high maintenance costs through action of large hydraulic forces occurring at unsuspected locations."

3. A great amount of effort has gone into the design of drop shafts associated with the most recent tunnel sewer schemes. It is inferred in the literature that such efforts are warranted to insure trouble-free operation and to protect the considerable investment involved with such schemes.

4. All of the tunnel system drop shafts reviewed were unique designs to account for site specific conditions.

5. Actual data on prototype performance of tunnel system drop shafts are practically non-existent. The limited data available indicate that physical hydraulic model testing adequately predicts hydraulic performance, and to a lesser extent deaeration performance.

A. Design Criteria

The hydraulic design criteria are essentially the same as for the drop structures and are repeated below for ease of reference:

- The structure should operate adequately for both pressurized and unpressurized flow conditions over a range of flows from zero to the design peak flow.

- The energy associated with the drop should be effectively dissipated before the flow enters the tunnel.

- Control of air entrainment and adequate venting should be accomplished within the structure.

- Head loss through the drop shaft should not adversely affect the drainage system's flow-carrying capacity.

- The structure must be cost effective.

- The structure must be constructable within available land areas.

B. Evaluation of Alternatives

As part of the preliminary design process, various alternative types of structures were first identified and then screened to eliminate those which were obviously unsuitable. There were four basic types of structures considered; ogee spillway, in-line drop shaft, morning-glory, and offset drop shaft. The following is a brief summary of the preliminary evaluation:

1. Ogee Spillway

The ogee spillway alternative is similar to ogee spillway considered for the drop structures. The hydraulic performance of

the ogee spillway is adequate under all flow conditions.

2. In-Line Drop Shaft

The in-line drop shaft was not considered a viable alternative for the following reasons:

1. Air Entrainment. The SAFL model studies indicated that the volume of air entrained in a plunge drop shaft ranges from 8% to 15% of the drop shaft design flow. Since the in-line drop shaft has no provision for air removal, the entrained air could produce undesirable waterhammer effects.

2. System Vulnerability. Mathematical modeling demonstrated that the surge pressure may build to a significant level as it moves upstream during the tunnel pressurization stage. Since the in-line drop shaft is mounted directly over the crown of the tunnel, it is vulnerable to the steep pressurization wave.

3. Impact. The impact of a fluid is in direct proportion to the flow velocity. For this project, the average head drop in the drop shafts is approximately 36 feet and the falling velocity in the shafts is on the order of 50 feet per second. A wear-resistant lining and special reinforcement would be required to protect the tunnel base from the tremendous impact caused by such a high velocity.

4. Wave Propagation. When a considerable volume of liquid abruptly drops in a closed conduit, a large portion of the dynamic energy will be converted to elastic energy and a series of positive and negative pressure waves will travel back and forth until they are damped out by friction. If this dynamic phenomenon coincides with the surge which occurs during tunnel pressurization, it may seriously damage the tunnel system.

3. Morning-Glory Type Drop Shaft

The morning-glory type drop shaft would not provide adequate energy dissipation unless special provisions, such as a stilling basin, are introduced. The extra construction cost for the curved elbow and stilling basin made this alternative economically infeasible.

4. Offset-Plunge Type Drop Shaft

The offset-plunge type drop shaft structures have been installed in St. Paul, Minnesota, where the design discharge and drop range are similar to the I-10 system. According to personal communications with Operations and Maintenance personnel at the City of St. Paul, the offset-plunge type drop shafts have performed satisfactorily with little or no maintenance. In addition, hydraulic model testing at the St. Anthony Falls Hydraulic Laboratory has verified that this offset-plunge type drop shaft system would be suitable for the I-10 Inner Loop system. Therefore, the offset-plunge type was selected as the recommended drop shaft for this project.

C. Additional Evaluation of Alternatives

1. Vortex Drop Shaft

Additional evaluation work was performed to determine if a vortex-type drop shaft could be used on this project to lessen the space required and/or reduce the construction costs. The vortex-type drop shaft is similar to the plunge-type drop shaft and consists of the following major elements:

- Inlet transition
- Circular shaft

- Horizontal deaeration chamber
- Outlet transition
- Air vent

One of the main differences between the vortex design and the plunge design is the geometry and configuration of the inlet transition. The vortex design has a tangential inlet transition with constriction while the plunge design has a box-shape free trajectory inlet. The other difference is that the deaeration chamber of the vortex design has a circular cross-section while that of the plunge design has a horseshoe shape cross-section.

The horizontal deaeration chamber for the vortex design is approximately 35 percent longer than that for the plunge design, while the shaft diameter for the vortex flow is about two feet smaller than that for the plunge flow. The shaft diameter for the plunge design is larger because the shaft of the plunge design includes a "built-in" air vent. All of the other element dimensions are compatible between the two types of design. It is apparent that the vortex drop shaft does not gain any economic or spatial advantage over the plunge design.

2. In-Line Drop Shaft

Additional evaluation work was also performed on the feasibility of using an in-line drop shaft on the West Tunnel where discharges are lower and a smaller drop shaft is possible. It was found that the in-line drop shaft should not be used due to the following reasons:

1. When a stream of liquid abruptly drops into a closed conduit, a large portion of the dynamic energy will be converted to elastic energy and a series of positive and negative waves will travel back and forth until they are damped out by

friction. Therefore, it is important to dissipate a portion of the energy before the flow enters the tunnel. The in-line drop shaft does not provide a method of energy dissipation.

2. Another important factor in drop shaft design is to remove the entrained air in the flow because the uncontrolled air will reduce the tunnel capacity and cause waterhammer phenomenon in the tunnel system. The in-line drop shaft does not have any provision for air removal.

3. Due to the high falling velocity in the drop shaft, the section of tunnel adjacent to the in-line drop shaft would have to be reconstructed with an impact-resistant lining and special reinforcement.

4. The in-line drop shaft is mounted directly over the crown of the tunnel and will act as a conduit for air release from the tunnel. As we know, the West Tunnel drop shafts are located in the downtown area. During the tunnel pressurization stage, the high-velocity air releasing through the shaft may create a safety hazard to nearby pedestrians. A side-feeding drop shaft discharges storm water at the spring line of the tunnel and hence is not subject to air release after the water level in the tunnel rises above the crown of the connecting conduit.

3. Helicoidal Drop Shaft

Engineers at HNTB developed a new drop shaft design concept to meet the requirements of the Inner Loop drainage system. This design, the helicoidal drop shaft, accomplishes the energy dissipation and deaeration functions while minimizing the amount of space needed for construction.

The helicoidal drop shaft consists of a vertical cylindrical shaft with one or more continuous helicoidal ramps attached to the shaft wall. Flow enters and exits the drop shaft almost tangentially, at approximately the mean uniform-flow velocity in the helicoidal channels. The center of the drop shaft is open, maintaining atmospheric pressure throughout its length. Figure 7-2 shows a schematic diagram of the helicoidal drop shaft.

The Iowa Institute of Hydraulic Research (IIHR), affiliated with the University of Iowa, performed extensive hydraulic modeling of the helicoidal design under contract with HNTB. This modeling was done to define the flow characteristics and air entrainment effects associated with variations in the basic design. Single, double, and triple ramp configurations were modeled. Two connecting conduit cross-sections were tested, one circular and one consisting of a parallelogram. Tests of each configuration were conducted for a range of flows.

Results of the model testing indicated that a double ramp design is optimal. Flows down the drop shaft were steady, stable, and free of disturbances. At high flow rates, the free surface in the helicoidal channels was nearly vertical and the flow was bounded by both the upper and lower ramps. At lower flows, the free surface had a greater inclination from the vertical, and the flow was bounded by the lower ramp and the shaft wall. Flows remained attached to the wall of the shaft at all flow rates.

The spiralling flow leaving the ramps plunged into a pool formed at the bottom of the shaft, and entrained some air. Most of the air bubbles converged toward the center of the pool because of the radial pressure gradients produced by swirling flow, and rose toward the free surface. The shaft itself serves as a vertical

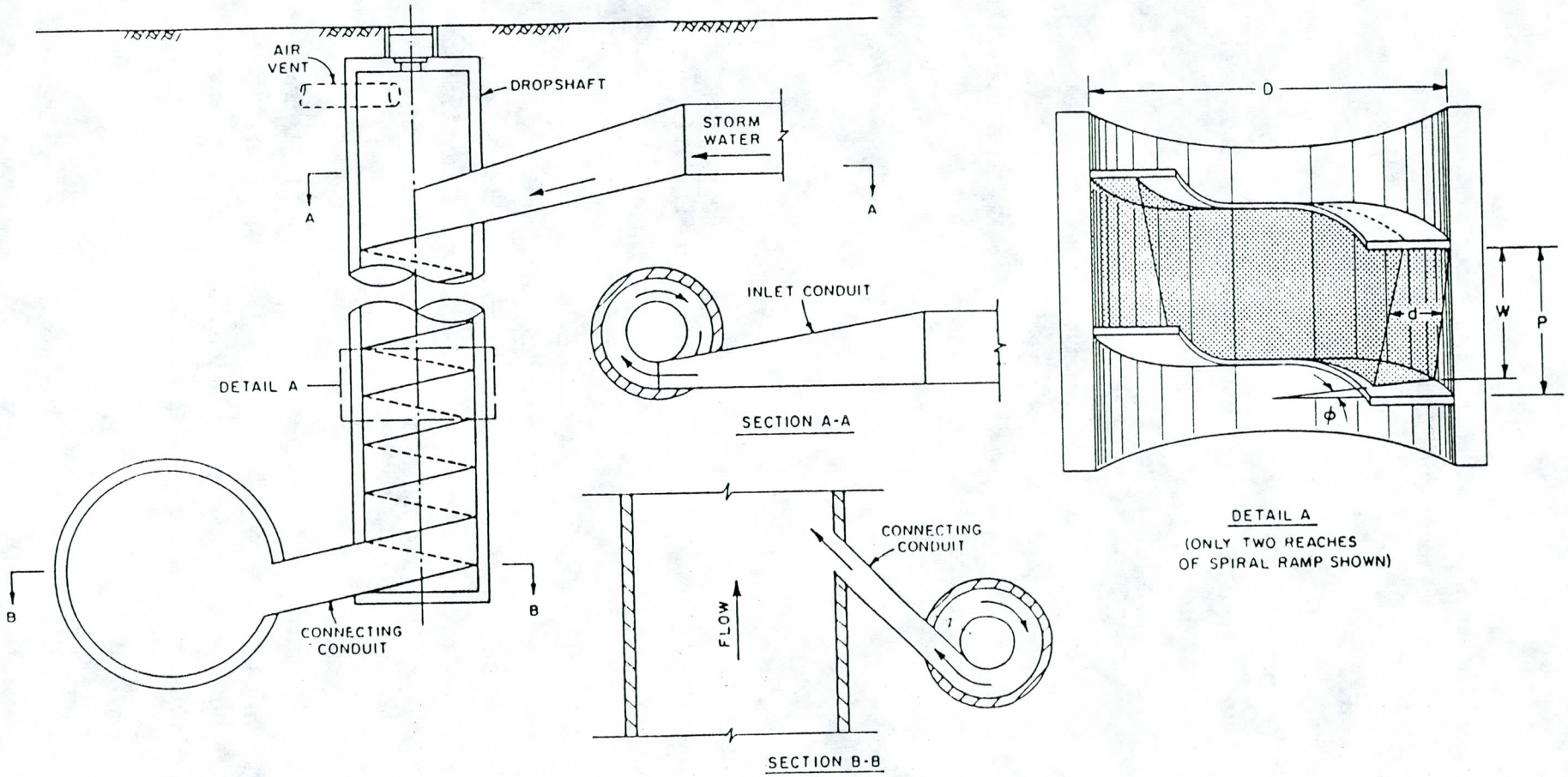


Figure 7-2 Helicoidal Drop Shaft Typical Sections

deaeration chamber. The small remaining amount of entrained air was carried into the conveyance tunnel. At large pool depths, a thin air-bubble core existed in the center of the pool, and the amount of air transported into the tunnel was virtually zero. No separate deaeration chamber is required because the concentration of air transported into the conveyance tunnel is so low. The cross-sectional shape of the connecting conduit had little effect on air concentrations.

Energy dissipation in the helicoidal drop shaft occurs mostly as a distributed friction loss. This energy loss occurs along the helicoidal channels rather than in the plunge pool at the bottom of the shaft. The physical modeling conducted by IIHR indicated that friction factors for the helicoidal channel flow are significantly higher than those for equivalent straight channel flow. This eliminates the need for additional energy dissipaters.

The main advantage realized from use of the helicoidal drop shaft is the reduction in space required for its construction. This reduction occurs principally because the long deaeration chamber required by other designs is eliminated. Benefits include reduced construction costs, low impact on traffic, and minimization of utility relocations. These benefits are especially important in a developed urban setting such as Phoenix. In such dense urban settings, minimization of pavement, curb and sidewalk removal and replacement amounts to a considerable saving in costs associated with construction of the drop shaft.

The helicoidal drop shaft was adopted for use at Fillmore, Grant and Tonto streets, and the Maricopa Freeway along the West Tunnel. Table 7-1 shows the dimensions of these drop shafts as recommended in the IIHR report. Figure 7-3 shows the double-ramp model used to produce the prototype dimensions.

TABLE 7-1

Prototype Dimensions for the Helicoidal Drop Shafts

Drop Shaft	Fillmore Street	Grant Street	Tonto Street	Maricopa Fwy.
Design Discharge, Q_p (cfs)	140	170	135	60
Model Scale, L_r	9.68	10.46	9.54	6.90
Helicoidal-Ramp Dimensions				
Inner Radius, R_1 (ft)	2.87	3.10	2.83	2.05
Outer radius, R_2 (ft)	4.64	5.01	4.57	3.31
Pitch, P (ft)	4.03	4.36	3.98	2.87
Connecting Conduit Dimensions				
Diameter D_1 (ft)	2.62	2.83	2.58	1.87
Drop, Z (ft)	2.02	2.18	1.99	1.44
Inlet Conduit Dimensions				
Width, W_3 (ft)	5.34	5.77	5.26	3.81
Width, W_4 (ft)	2.32	2.50	2.28	1.65
Height, H_3 (ft)	5.34	5.77	5.26	3.81
Height, H_4 (ft)	4.03	4.36	3.98	2.87
Length, L_1 (ft)	26.87	29.05	26.49	19.15

NOTES:

1. R_1 does not include allowance for freeboard.

2. The Fillmore Street and Tonto Street drop shafts can have the same dimensions as those for Grant Street drop shaft.

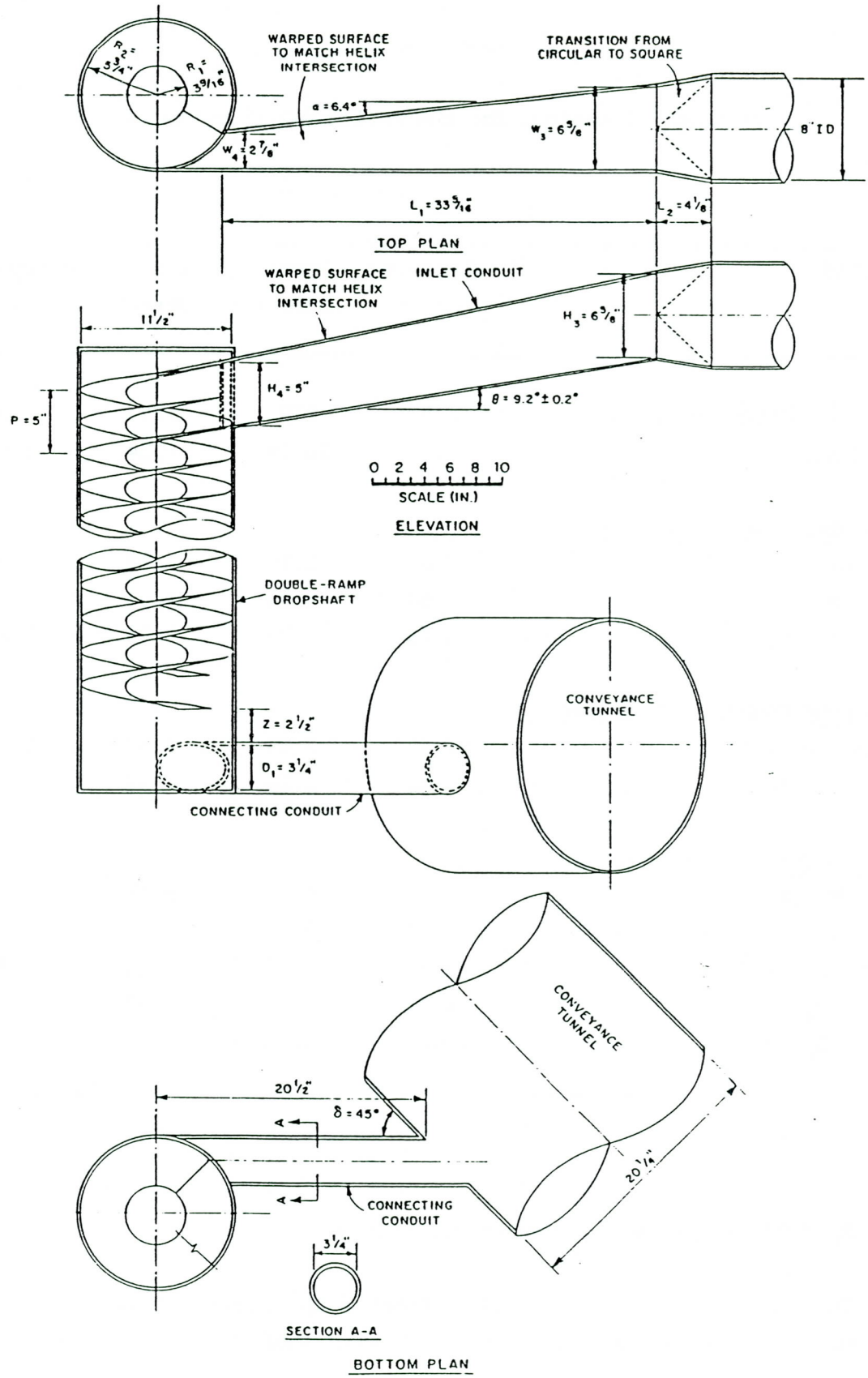


Figure 7-3 Configuration of the Double-Ramp Structure

D. Central Avenue Drop Shaft

The City of Phoenix installed a 96-inch storm drain in Central Avenue to connect to the North Tunnel of the Inner Loop drainage system. Both plunge and helicoidal dropshaft designs were considered for this location. Site constraints, including the Central Avenue detour, were considered so significant that the plunge dropshaft was rejected for this site. The helicoidal design, with some modifications, was selected in order to meet the site constraints and minimize the impact of construction on Central Avenue traffic.

Constraints imposed by existing and proposed utilities, adjacent buildings, relative elevations of the Central Avenue storm drain and the North Tunnel where it crosses under Central Avenue, and the Central Avenue detour necessitated changes to the original helicoidal design used at the four West Tunnel sites. IIHR performed additional model tests to identify what design modifications were required to produce acceptable hydraulic conditions in the dropshaft. The modified dropshaft design was required to meet the following performance criteria:

1. The flow should be stable in all components of the structure. There should be no objectionable surging, separation, or other unsteady features.

2. The hydraulic grade line (HGL) in the manhole box should not exceed an elevation of 1085 ft. for the critical condition of Q=383 cfs and conveyance tunnel HGL = 1078 ft.

3. The model air-discharge concentration in the connecting-conduit flow should not exceed 0.25% for any condition in which the conveyance tunnel HGL is above the tunnel crown.

4. Spillage off the ramps should be minimized.

Two peak flow rates were evaluated in the model testing. The first flow rate of 383 cfs, combined with a tunnel HGL of 1078 ft., represents the 50-year design discharge for the Central Avenue storm drain and the maximum expected HGL in the North Tunnel at Central Avenue during the 50-year storm event. A second flow rate of 532 cfs, with a relatively low tunnel HGL, represents the 2-year design discharge for the Central Avenue storm drain. The 50-year flow of 383 cfs is based on the 96-inch pipe operating under surcharged conditions, with a hydraulic gradient equal to the ground slope in that area. The 2-year flow of 532 cfs is the design discharge for the Central Avenue storm drain at a hydraulic gradient equal to the pipe slope, which is steeper than the ground slope.

Several configurations of the inlet, two connecting-conduit configurations, and various helicoidal ramp positions were evaluated for performance at the two design dsicharges. The recommended configuration, shown in Figure 7-4 satisfies the design criteria listed above. Dimensions shown in Figure 7-4 are model dimensions. The model scale is 1:16.7. Note also that the model configuration is a mirror image of the prototype Central Avenue dropshaft. In other words, in the model water swirls in a clockwise direction, whereas in the prototype water swirls in a counterclockwise direction.

The final design of the Central Avenue dropshaft incorporated the modifications recommended by IIHR. The inlet conduit was redesigned to introduce flow into the dropshaft smoothly, and was shortened to allow construction within the site constraints at Central Avenue. The connecting conduit is attached to the dropshaft wall tangentially to reduce exit head losses.

Air discharge concentrations in the most critical conditions, Q=532 cfs and tunnel HGL at the tunnel crown, are well below the upper limit of 0.25% in the model. The maximum HGL in the manhole box is predicted to be 1084.5 ft., which is at 0.5 ft. below the permissible upper limit.

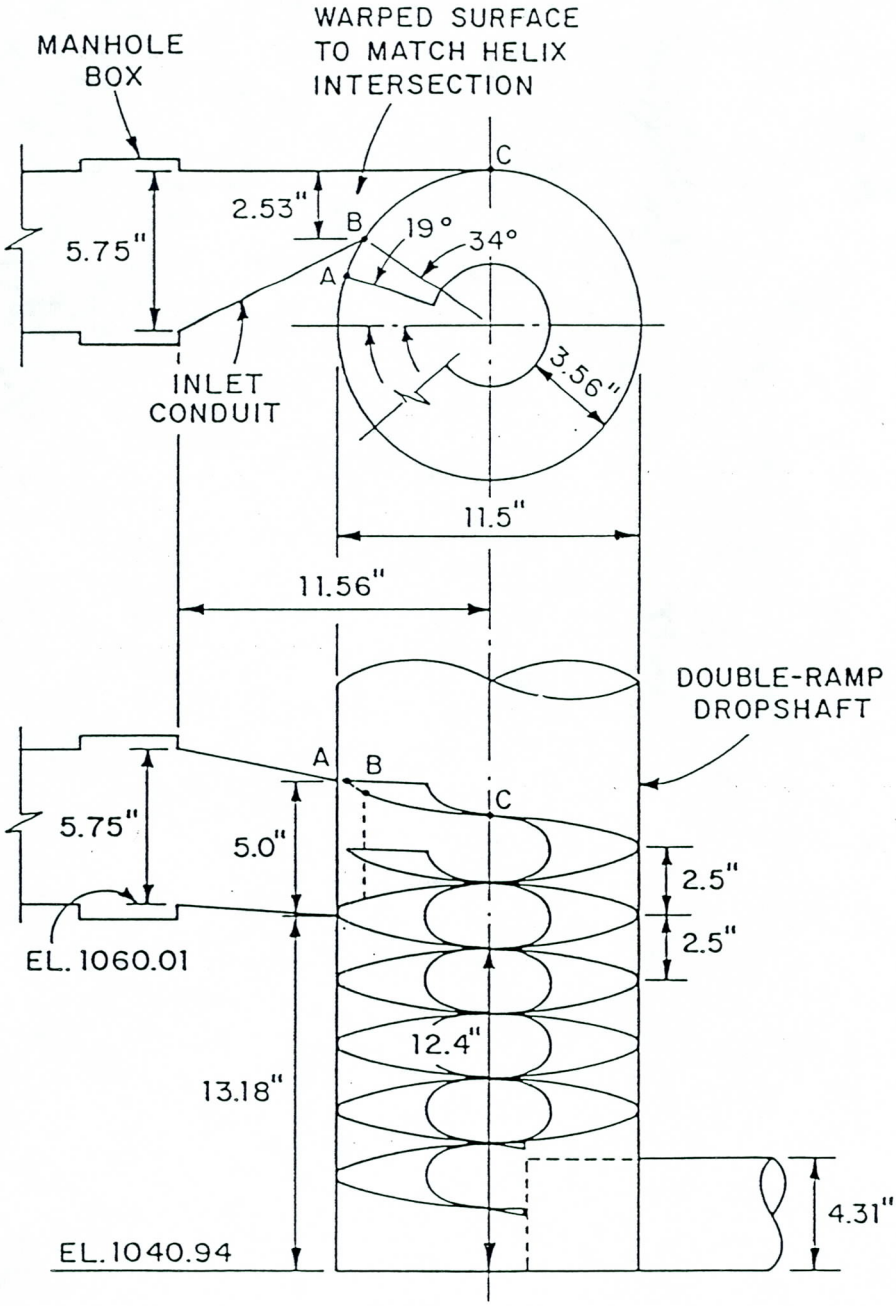


Figure 7-4 Central Avenue Dropshaft Configuration

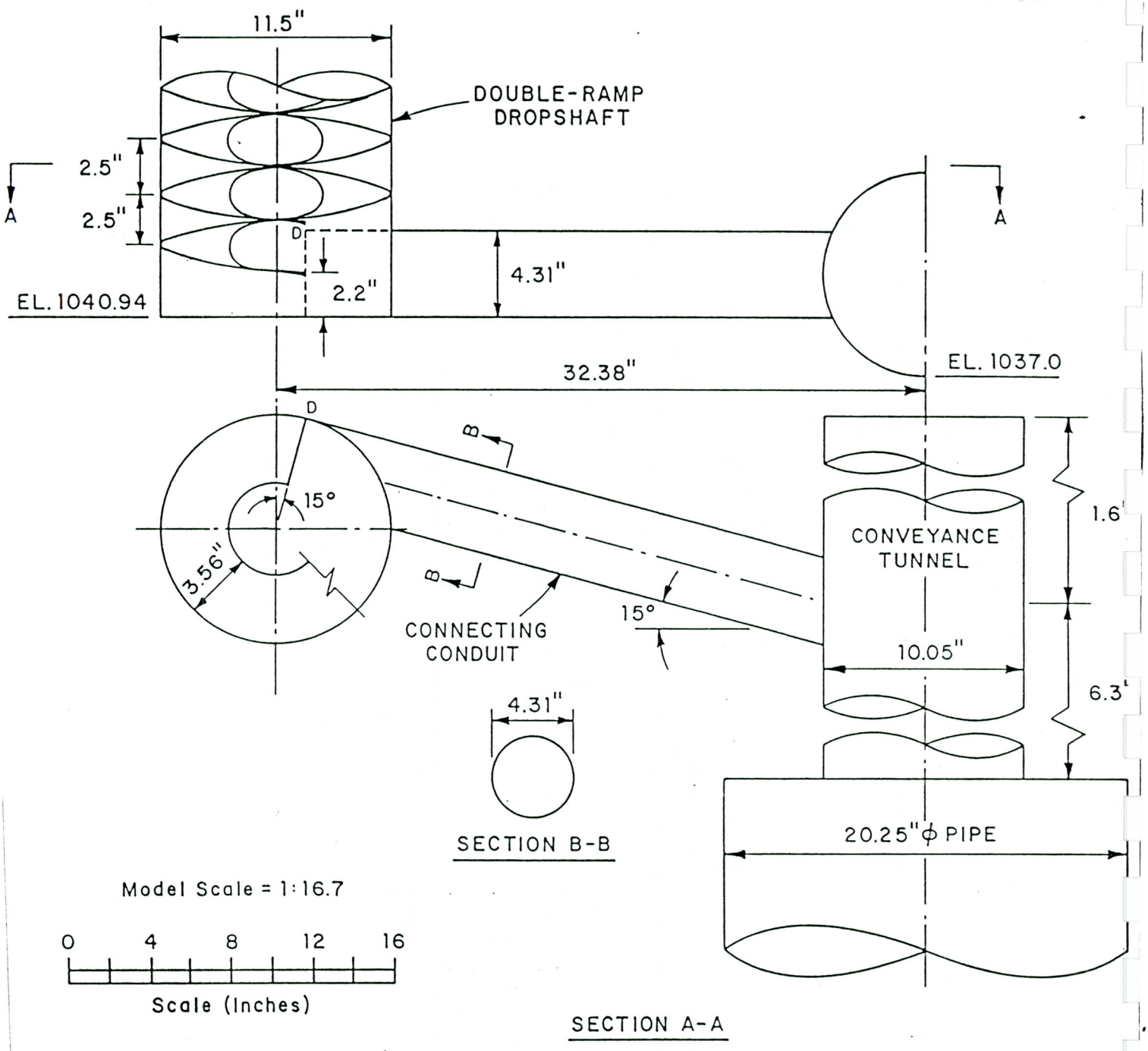


Figure 7-4 (continued)

III. 2ND STREET JUNCTION STRUCTURE

The North Tunnel joins the West Tunnel at Second Street. The connection is made through a specially designed junction structure. Design peak flows into the junction are 2220 cfs from the west branch of the North Tunnel, and 2020 cfs from the east branch of the North Tunnel.

Studies conducted by Dr. Song indicate that flow at the junction goes through three phases:

- Open channel combining flow from the North Tunnel branches to the West Tunnel, with tunnels partially full.

- Dividing flow from the West Tunnel to the North Tunnel branches, with pressurization from downstream to upstream. Shock wave moves up the West Tunnel through the junction and into the east and west branches of the North Tunnel.

- Combining pressure flow, from the North Tunnel branches to the West Tunnel.

Energy losses at branches and junctions are generally low, and head loss coefficients for prototypes are likely to be less than those determined by model testing. The basic factors affecting form losses on branching conduits include:

(a) general geometry and type of branch such as wye, manifold, bifurcation, trifurcation, etc.,

(b) variations in branch geometry such as angle of branches, cross-sectional areas, shape and ratio of cross-section,

(c) flow conditions for dividing, combining, reverse dividing or reverse combining flows and

(d) proportion of total design discharge flowing in the branch.

A. Design Concepts

The basic design concepts for the junction structure are as follows:

- Keep the overall and branch geometries of the junction structure simple.
- The configuration and type of junction structure should be easily constructable.
- Accommodate both open channel and closed conduit flows for combining and dividing flow conditions.
- Open channel flow from one branch of the North Tunnel into the other is to be avoided.
- Excessive energy losses during pressure flow are to be avoided.
- The structure is to be cost effective.

The type of structures considered include a "wye" junction and a "tee" junction.

The "wye" junction includes two individual branches of the North Tunnel, each connected to the West Tunnel by a conical transition at a 60 degree bifurcation angle. Each branch of the tunnel is 14 feet in diameter and is combined to the "wye" junction by a 60 degree bend.

The principal advantages of the "wye" junction when compared to a "tee" junction are:

(a) the streamlining and maximum efficiency of flow for symmetrical conditions of combining or dividing flows, and

(b) head loss coefficients for a "wye" type are likely to be less.

Although the "wye" type junction is more efficient hydraulically for pressure flow, its complexity of configuration, structural arrangement and constructability render it uneconomical.

With the connection of the North Tunnel to the West Tunnel at a 90 degree angle, the "tee" junction can be designed to function adequately. The "tee" junction fulfills the requisites of:

(a) simplicity of overall geometry,

(b) convenience of constructability,

(c) minimum allocation of space and

(d) overall economy.

B. Connection to West Tunnel

Original plans called for the West Tunnel to terminate at a point approximately 300 feet south of the junction structure. A conduit was therefore required to connect the junction structure to the West Tunnel. Three methods of construction were considered for this conduit:

Method A - To extend the 21-foot diameter tunnel north to the junction structure. Concrete bedding would be required because the freeway is less than 6 feet above the top of the tunnel.

Method B - To install a 21-foot diameter cast-in-place concrete conduit, with proper concrete bedding.

Method C - To extend the 21-foot diameter tunnel, with a 30-foot span bridge over that portion of the tunnel under the freeway to accommodate freeway live loads.

C. Evaluation of Alternatives

The three tunnel connection alternatives were combined with the two alternative connection geometries to generate six alternatives for the Second Street junction structure. The combination of a "tee" connection and extending the tunnel, with the protective bridge, was found to be most cost-effective. This combination was adopted for the junction structure design.

A 45-foot diameter access shaft at the north end of the extended West Tunnel was left in place for construction of the junction structure. This necessitated a 38-foot transition section from the end of the tunnel to the outlet of the junction structure. The transition is a 21-foot diameter cast-in-place concrete conduit. This section is flattened on the bottom, rather than perfectly circular. No protective slab is required, since the transition section is outside the I-10 roadway limits.

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CHAPTER 8 - OUTFALL STRUCTURES

SYNOPSIS

This chapter discusses design of the outfall structures which connect the East and West Tunnels to the Salt River. Pump stations which are provided at each outfall are also described.

I. GENERAL OVERVIEW

Outfall structures convey storm water from both the East and West Tunnels at their terminus near the Salt River. The outfalls conduct storm flow from the tunnels up to the level of the Salt River. Both outfalls are designed to carry a 50-year peak flow of 5,000 cfs.

A pump station is provided at each outfall structure to dewater the corresponding tunnel after a storm event. The pump stations are required to remove water left in the tunnel after the inverted siphon depressurizes and outflow to the river ceases.

II. EAST TUNNEL OUTFALL

The East Tunnel outfall connects the East Tunnel of the Inner Loop drainage system to the Salt River. It is located south of University Drive, at approximately 20th Street.

The outfall consists of a double 14′ x 12′ box culvert riser from the tunnel at a 10% upslope, about 700 additional feet of box culvert and approximately 800 feet of open channel leading to the river. A pump station is located at the transition between the 21-foot diameter tunnel and the box culvert riser to dewater the system after a storm event.

A. Design Criteria

The outfall structure was designed to meet several different performance criteria. These include access to the tunnel for maintenance, energy dissipation, flow capacity and cost-effectiveness.

1. Tunnel Access

Vehicular access to the tunnel system is required for maintenance of the system after storm events. Sediment and debris will have to be removed after the tunnel operates and is dewatered. An access pad was provided at the entrance to the box culverts.

An access control device was provided. This device is a passive system reasonably capable of preventing unauthorized access to the tunnels. Access control alternatives are dealt with later in this chapter.

2. Energy Dissipation

There are two areas where energy dissipation may be a potential problem. The first area is at the end of the open channel, where flow discharges to the river. Energy dissipation devices at this point will be at risk from large, uncontrolled river flows. The most efficient solution was to install 30 inch diameter rip rap in the channel.

The second area where energy dissipation may have been a problem is at the end of the access pad. Maintaining subcritical velocity through the access pad is very important. Many configurations of access pad and open channel were modeled to determine the most satisfactory configuration.

3. **Flow Capacity**

The outfall structure will carry the 50-year design discharge of 5,000 cfs from the East Tunnel without creating adverse flow conditions in the tunnel or outfall. The discharge requirement must be considered in the design of access control devices and energy dissipaters.

4. **Cost Effectiveness**

Initial construction cost and future maintenance costs were also important criteria for the outfall design. Each element of the design was carefully selected for both functional purposes and cost effictiveness.

B. **Evaluation of Alternatives**

Fourteen different configurations of the outfall were evaluated on the basis of the design criteria given above. The double box culverts and the pump facility were not included in the evaluation since they are common to all fourteen alternatives. The evaluation of the alternatives is summarized below.

1. A 52′ wide access pad transitioning with a converging walls hydraulic jump energy dissipater into a 30′ wide, 0% slope channel with the flowline at the river elevation.

   The hydraulic analysis of this design is questionable due to the existence of converging flow in the hydraulic jump. The length of the required structure is excessive, and the location of the hydraulic jump is highly sensitive to tailwater conditions.

2. A 52′ wide access pad transitioning with a Saint Anthony Falls (SAF) Energy Dissipater to a 52′ wide, 0% slope channel with the flowline at the river elevation.

   The design is relatively efficient in construction material quantities. However, the design is highly dependent upon tailwater conditions. The SAF design causes some of the energy dissipation to occur downstream of the structure, in the outfall channel.

3. Same as #2 except with the United States Bureau of Reclamation (USBR) Low Froude Number Stilling Basin.

   The design requires a large amount of concrete forming and material, and is tailwater dependent.

4. Same as #2, except with the USBR Baffle Apron Spillway as an energy dissipater.

   This design is independent of tailwater conditions, but the material and forming costs are high.

5. A 30′ wide access pad with vertical walls and a side gate, with an abrupt transition to a 30′ wide channel. The channel has a slope of 0.67%.

   The hydraulics of this configuration are unacceptable; the maximum design flow velocity reaches 21.5 feet per second (fps) just downstream of the access pad in the riprap and gabion channel, with a Froude number of 1.7 and a mild hydraulic jump.

6. Same as #5 but with vertical transition walls and expanding channel width at 14° from the channel centerline.

Adding walls only reduces maximum design flow velocity to 21.2 fps.

7. Same as #6 but with the transition walls roughened to the same Manning's "n" value as the channel.

   Roughening the walls reduces the maximum design flow velocity to 20.4 fps.

8. Same as #7 but with the walls expanding at 7° from the channel centerline. Reducing the expansion angle reduces the maximum design flow velocity to 20.2 fps.

9. A 30′ wide access pad transitioning to a 15′ wide channel, with a 40′ long concrete transition from the rectangular access pad cross-section to the trapezoidal channel cross-section. The access pad has a side gate, and the channel has a slope of 0.67%.

   The reduced channel width raises the energy elevation downstream of the access pad and transition enough so that no energy dissipation is necessary at that location. Backwater analysis (by HEC-2) for 1250, 2500, 3750 and 5000 cfs indicates that all four water surface profiles remain well above critical depth throughout the channel transition, access pad and culvert entrance. The design flow velocities vary from 12.5 fps in the channel and at the toe of the transition, to about 16.6 fps in the culvert. The box culvert and the open channel both flow at a subcritical normal depth, with the flow depths in the transition and access pad area kept well above critical depth thru backwater effects of the smaller downstream channel.

   The reduction in channel width from 30′ to 15′ causes a slight increase in channel velocity but leads to a slight reduction in the Froude number of the channel flow.

The 15′ channel significantly reduces the excavation and riprap volume necessary for construction.

10. A 52′ wide access pad transitioning to a 15′ wide channel, with a 40′ long concrete transition from the rectangular access pad cross-section to the trapezoidal channel cross-section. The channel has a slope of 0.67%.

    The design causes a heavy fluctuation of flow velocities through the access pad and transition. Because of the velocity changes, the energy losses are high and the design flow in the culvert is forced into an unacceptable pressure flow condition.

11. Same as #9 but with a 20′ wide channel. The channel and culvert flow conditions are acceptable for the design discharge, but the velocities of flow still vary excessively through the access pad and the transition.

12. A 30′ wide access pad with a 40′ long concrete transition. The access pad is 1′ lower than the culvert outlet, the drop occurring over the 10′ adjacent to the culvert. There is an opening with no gate along one wall of the access pad, and a short wall at 45° to the access pad and transition centerline. The 45° wall will serve to redirect flow expanding through the access opening back into the transition. The channel has a slope of 0.51%.

    The computed hydraulics of this configuration are good and there is the advantage of no gate mechanism. However, the 45° wall may cause some unpredictable turbulence that could prove damaging.

13. Same as #12 except that the 45° wall is replaced by lengthening the transition from the vertical wall to the

channel sideslope, making the transition asymmetric. The access pad is 35′ wide at the point where the transition begins on the access opening side.

Design flow passes through critical depth at the culvert outlet. The asymmetric transition should cause less turbulence than the 45° training wall, but is still hydraulically unpredictable.

14. A 30′ wide access pad with parallel walls for 24′ below the culvert outlet, flaring out 5′ wider on one side with a 20′ long gap for vehicular access. The 35′ wide downstream end of the access pad transitions across 45′ with concrete to the 15′ wide trapezoidal channel section. The access pad is 0.5′ lower than the culvert outlet, the drop occurring over the 5′ adjacent to the culvert. The channel has a slope of 0.59%.

    The computed hydraulics of this configuration are very good. Vehicular access is simple, and the design is efficient in construction material quantities required.

C. Final Configuration

Final design of the East Tunnel outfall was based on alternative number 14. Truck access through a 20-foot gap in the flow-training wall is easy. No gate is required. A 12-foot wide road allows vehicular travel to and from the access pad.

The Corps of Engineers program HEC-2 was used to compute velocities and water surface profiles through the channel, transition and access pad. Five flow rates were modeled: 1250 cfs, 2500 cfs, 3750 cfs, 5000 and 6000 cfs. All flow rates produced smooth water surface profiles, and the higher discharges also resulted in relatively constant computed velocities. The lower discharges produced somewhat more variable velocities, but flow conditions are subcritical in all cases.

The open channel section is a trapezoidal channel with a 15-foot wide base. The sides slope up at 1.5:1 to match existing grades at the top of the channel. The base of the channel is lined with 18-inch riprap to a depth of 3 feet. The sides of the channel are lined with 3-foot thick gabions which continue to the channel outlet.

A 10-foot thick section of 30-inch riprap is provided at the channel outlet to the river. This should retard headcutting if the river bed degrades at this location. Inspection and maintenance of the channel outlet will probably be necessary after all significant tunnel flows. Other than the outlet area, the entire outfall system should be stable for discharges up to 5000 cfs, subject to routine cleanup of debris. Figure 8-1 shows the plan and profile of the east outfall.

III. WEST TUNNEL OUTFALL

The West Tunnel outfall connects the West Tunnel to the Salt River. The outlet to the river is approximately 1300 feet south of Watkins Street and 150 feet downstream of the Central Avenue bridge.

The west outfall consists of a double 14′ x 12′ box culvert riser from the West Tunnel at a 9.65% upslope; a 29-foot wide vertical-walled channel where the depth of cover over a box culvert would be less than 10 feet, also at a 9.65% upslope; an 87-foot long horizontal vehicular access pad; and a 52-foot wide baffled apron spillway dropping to the Salt River. A pump station is located at the transition between the tunnel and the box culvert riser to dewater the tunnel after a storm event.

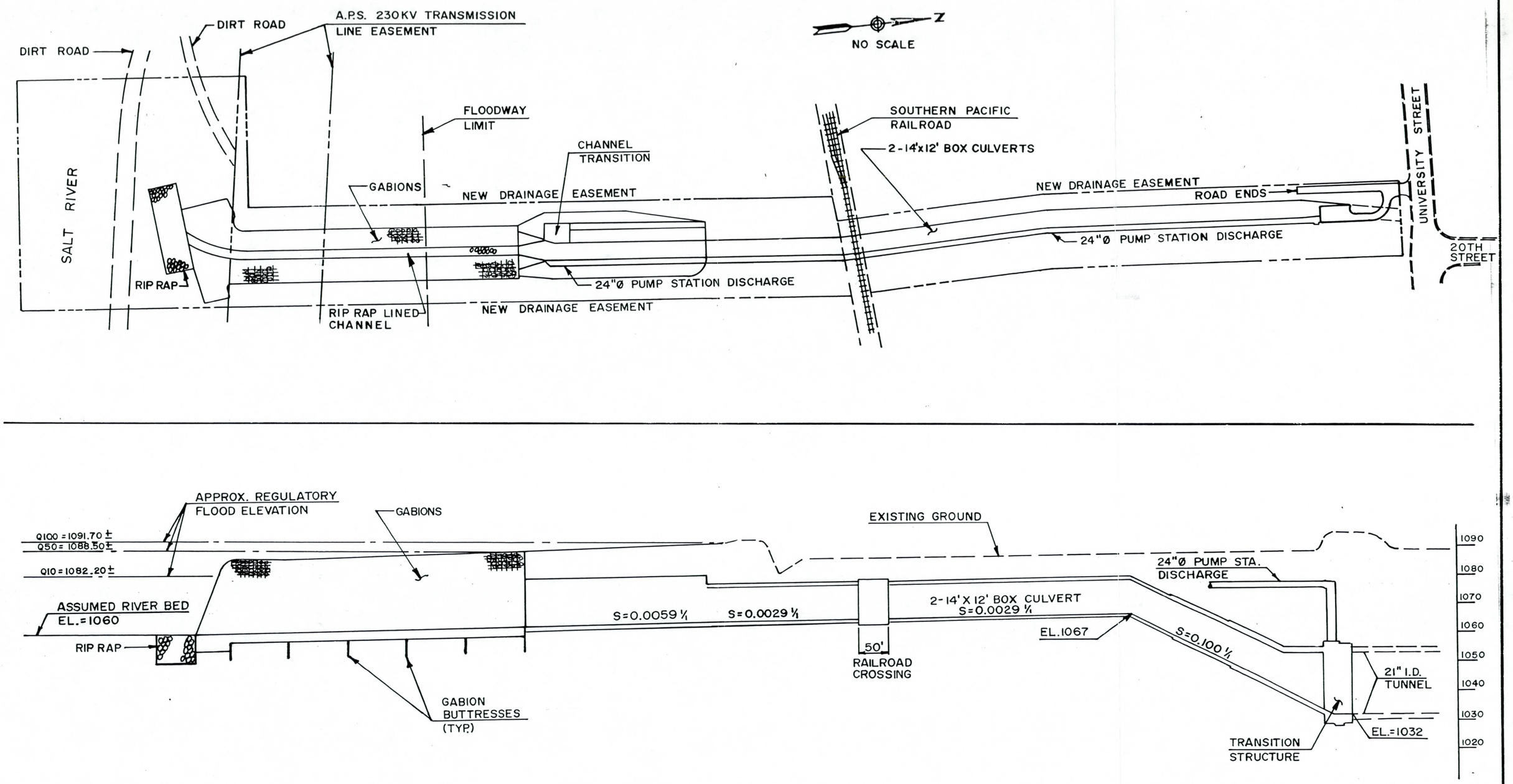

Figure 8-1

East Tunnel Outfall Plan & Profile

A. Design Criteria

The design criteria for the west outfall were the same as for the east outfall, described above. Space constraints, however, dictated a different energy dissipation method. The design flow is 5000 cfs in this outfall, similar to the east outfall. Vehicular access is required. Cost-effectiveness was also a consideration.

B. Evaluation of Alternatives

Site constraints at the west outfall dictated the installation of an energy dissipater. An dissipater that operates independent of tail water conditions is required, due to the variability possible in the water surface elevation of the Salt River. The energy dissipater best suited to site conditions is the United States Bureau of Reclamation (USBR) Baffled Apron Spillway.

The Baffled Apron Spillway begins with a 4-foot high, broad-crested weir to create a critical depth condition for low tailwater. The weir also controls velocities through the channel width transition area upstream.

The factor most critical to the Baffled Apron Spillway design is the discharge per foot width. Because the design discharge per foot for the west outfall exceeds the range used in the original USBR design procedure, the modified design procedure developed by the USBR for higher discharges per foot was used. Two additional rows of baffles were used beyond the normal requirements for such a short drop because of the long-term unpredictability of the Salt River bed elevation.

The width of the structure was set by the vehicular access requirement. A 52-foot wide access pad was designed to allow

access to the box culverts and tunnel. The energy dissipater is the same width as the access pad.

C. Final Configuration

The final configuration of the west outfall was controlled by more severe constraints than those imposed by the east outfall site.

Design of the west outfall also drew on the experience gained during the evaluation process for the east outfall.

The final configuration consists of a channel with a reverse curve to avoid construction work around old abandoned Central Avenue bridge piers. The channel transitions to the Baffled Apron spillway with the 4 foot weir as described above.

Due to the proximity of the river, the energy dissipater and access pad are supported on 40-foot deep drilled shafts. Figure 8-2 shows the plan and profile of the west outfall.

IV. ACCESS CONTROL EQUIPMENT

Access to the 21-foot diameter tunnels by unauthorized personnel represents a potential safety hazard. Some type of control device is required at the east and west outfall structures to limit access to authorized persons.

Four alternative access control measures were identified for evaluation. They include a camera surveillance system, a vertical slide gate, a swing gate, and a breakaway gate. These alternatives represent a cross-section of available concepts to limit access.

A. Design Criteria

Evaluation of the four alternatives was based on this hydraulic concern, as well as operational reliability, safety, maintenance requirements, effectiveness of access control, and costs.

B. Evaluation of Alternatives

The additional head losses resulting from a closed access gate when the outfall channel is operating at design capacity would cause excessive backwater problems in the I-10 storm drainage system. Therefore, any access control device must be removed from the channel before the channel fills completely.

A matrix evaluation scheme was used to compare the alternatives. A summary of the alternatives and their evaluation is presented below.

1. Camera Surveillance

The camera surveillance option consists of a single camera mounted in a dustproof, waterproof housing installed in each outfall structure. The camera continuously monitors the outfall opening, and transmits images to the central processing unit (CPU) via microwave. Motion detectors and an on-site audio alarm system could be installed with the camera system to enhance its effectiveness.

The camera surveillance system would not physically prevent persons from entering the box culverts or tunnels. Alarms activated by motion detectors and posted signs would deter unauthorized individuals from entering the outfall area. Those

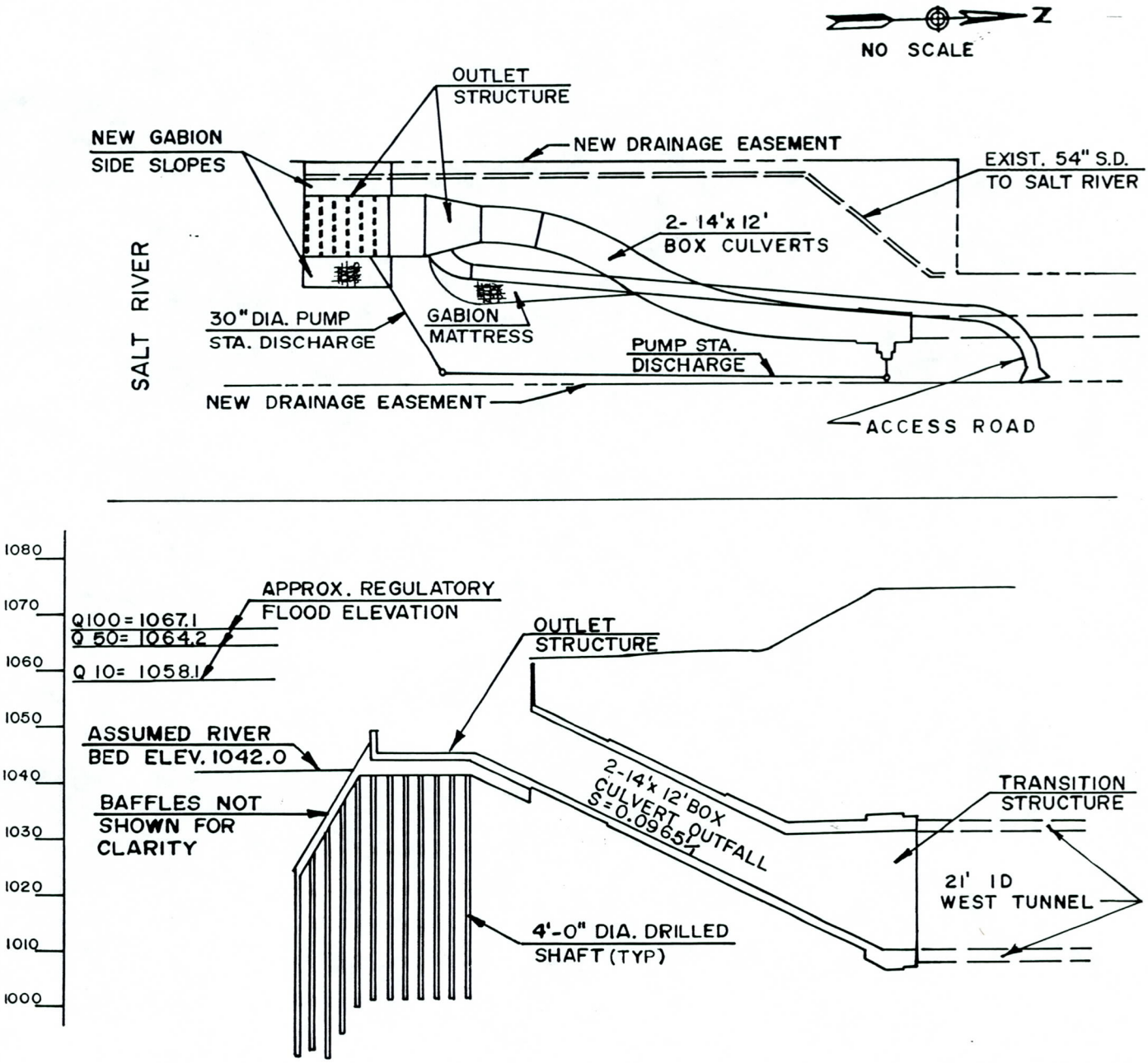

Figure 8-2 West Tunnel Outfall Plan & Profile

who do persist in attempting to enter the tunnel system would be detected, and the police department could be contacted to remove any trespassers.

This system can be installed at either outfall location with no modification to the box culverts. It provides a reliable means of determining when unauthorized personnel attempt to enter the area. The system is subject to mechanical failure, but failure would not adversely affect the hydraulic performance of the storm drainage system.

2. **Vertical Slide Gate**

The vertical slide gate alternative consists of a motor operated gate which slides up and down on guide rails. Gate operation is triggered by a water level sensor at the upstream end of the tunnel system.

Either a prefabricated steel mesh gate or a custom-designed structural steel gate could be used. The prefabricated mesh gate is more susceptible to tampering, but is easier to replace if damaged. The structural steel gate provides a greater level of security, but is more expensive. Both types of gate were carried forward in the evaluation.

This system is susceptible to mechanical failure. If the gate is not raised before the outfall structure begins conveying storm flow, excessive backwater problems will be created in the I-10 Inner Loop storm drainage system. System monitoring devices which would transmit power failure and gate position information to the CPU should be installed if this option is chosen.

3. Swing Gate

Two types of swing gates were considered, a swing-out gate and a swing-up gate. Both alternatives would be more costly and more difficult to maintain than the vertical slide gate. Neither swing gate alternative offers any advantage over the vertical slide gate. Therefore, the swing gate was not given any further consideration.

4. Breakaway Gate

The breakaway gate alternative requires two gates to swing open when the hydrostatic pressure behind the gates causes failure of a shear pin. The shear pin could be designed to shear at a load of 1000 pounds. This should be adequate to deny access to unauthorized persons, without creating an excessive backwater effect in the tunnels.

Maintenance personnel would need to keep an adequate supply of shear pins on hand, and replace the shear pin each time the gates are opened. Replacement of the shear pin with a bolt or some other material could cause the gates to not open, creating backwater problems. Telemetering equipment would be necessary to relay gate position information to the CPU.

B. Recommendations

A fifth option, added after the primary evaluation was completed, was chosen to control access to the tunnel system. The selected method is installation of a 4-strand barbed wire fence across the outfall channels, connected to chain link fences along each side of the channels. The barbed wire fences are breakaway sections which will have to be replaced after significant flows occur in the outfall channels. Warning signs in English and Spanish are posted in several locations.

The significant advantage of this method is the low installation cost. The combination of fences and signs should be adequate to control access to the tunnel system. No adverse hydraulic effect is anticipated.

If additional security measures are required, the camera surveillance system alternative is recommended. It can be installed at either site without the concrete modifications required by the control gate alternatives. It also has the least possibility of adverse hydraulic effects of all feasible alternatives.

See the report entitled "Feasibility Study: I-10 Inner Loop Storm Drain Access Control Equipment", prepared by HNTB for a detailed analysis of proposed access control devices.

V. OUTFALL PUMP STATIONS

Because the tunnel systems act as inverted siphons, a considerable volume of water will remain in the East and West Tunnels after outflow ceases. Approximately, 110 acre feet and 120 acre feet of water is stored in the East Tunnel and West and North Tunnels respectively. A pump station is provided at each outfall to dewater the tunnels after a storm event.

Pump station sizing is primarily controlled by the amount of time water can be allowed to remain in the tunnel. This time, in turn, is dictated by expected water quality conditions in the tunnels. An accurate prediction of conditions would require a detailed study, including field sampling, of runoff water quality in the Phoenix area. However, an indication of the types and concentrations of pollution that can be expected is found in the EPA publication "Urban Stormwater Management and Technology," EPA 600/8-77-014, September

1977. This data represents the best information available for storm water runoff quality.

Based on information given in this publication, it can be expected that the dissolved oxygen concentration in water left standing in the tunnels would reach zero in 7 to 15 days. After this time anaerobic conditions would exist in the tunnels, and problems with odor and gas would begin.

Anaerobic conditions may occur in the tunnels where the stagnant water is exposed to air. At these points, particularly at the upstream ends of the tunnels, gases could collect. Gases produced could include methane, which is highly corrosive. In order to avoid the potential hazards associated with these gases, it was recommended that the tunnels be pumped dry within 3 to 7 days after the conclusion of a storm event.

The pump station at each outfall is located at the low point of the transition between the tunnel and the box culvert riser. In order to dewater the tunnels within the 7 days recommended, two submersible pumps, driven by electric motors, are provided at each station. The pumps are rated at 3300 gpm each, for a total station capacity of 6600 gpm at each outfall. Discharge is through a 24 inch pipe into the outfall channel at both locations.

Continuous pumping could dewater a completely filled tunnel in approximately 4 days. If field tests of conditions in the tunnels indicate no air or water quality problems, pumping times could be adjusted to take advantage of off-peak power rates. This would result in significant operational cost savings while achieving more acceptable air and water quality in the tunnel system.

CHAPTER 9 - OPERATIONS AND MAINTENANCE

SYNOPSIS

This chapter provides a brief description of operations and maintenance procedures for the Inner Loop drainage system. Operation and maintenance of pumps and control systems used in the pump stations are not included.

I. GENERAL

Ease of maintenance has been a major design criterion for all components of the Inner Loop drainage system. As a result, many portions of the drainage system require no special maintenance procedures. The maintenance sections of this chapter will be restricted to special procedures necessitated by the unique design of this drainage system.

Hydraulic conditions control the operation of the Inner Loop drainage system. Little or no active control is necessary for safe and effective system performance. However, manual operation of the pumps at the East and West Tunnel outfalls is required whenever the tunnels are to be dewatered.

As mentioned above, the drainage tunnels operate as a passive system. There are no unusual operating characteristics due to the design of the tunnel system.

II. TUNNELS AND OUTFALLS

Siltation should be no more of a problem for the drainage tunnels than for an ordinary storm sewer. It is anticipated that the first portion of major inflows and more frequent minor flows will carry the bulk of the sediment that will enter the tunnels. The Salt River may be a secondary source of fine sediments when the river stage at the outfalls is high enough to allow water to enter the tunnel system through the outfall channels and culverts.

Significant flows, to be defined by operational experience, will flush out the majority of unconsolidated sediments. Accumulated sediments will need to be removed by maintenance personnel when the level of sedimentation would have a detrimental effect on the function of the drainage tunnels. This level can best be defined through operational experience. If sediments become cemented to the inner tunnel walls they should be removed. Cleanout may be accomplished by machinery equipped to operate in an enclosed environment.

Tunnel inspections need to be performed. An inspection schedule is recommended as follows:

- After the first 2-3 major events, or at 5 and 10 years after beginning of operations.

This schedule should be modified as operational experience and maintenance requirements dictate. Structural inspections should be performed with maintenance inspections.

The inspection could be limited to drop structures and limited distance up and down stream thereof. There should be no need to inspect the entire tunnel reach unless dictated by selective reach conditions near structures. These areas are most likely to show potential problems.

Workers entering the tunnel system should be equipped with air supplies, explosive and toxic gas detectors and oxygen concentration monitoring equipment, as required for any enclosed sewer or confined entry operation . Sufficient personnel should be on hand to maintain a surface presence along with the interior work party. Two-way communication contact, for dispatch of emergency crews or warning of storm events, should be maintained with base. Power equipment, if used in the tunnels, should be equipped with scrubbers and air

supplies in accordance with appropriate OSHA requirements for work in an enclosed environment. Special care should be taken not to enter the drainage system either during threatening weather or during storm events.

III. DROP STRUCTURES, DROPSHAFTS, AND 2ND STREET JUNCTION STRUCTURE

Drop structures and dropshafts are designed for passive operation whenever inflow from the collector system occurs. No active operational control is necessary. These components of the drainage system also act as air release facilities during operation. There are no unusual operating characteristics due to the design of the drop structures and dropshafts.

The 2nd Street junction structure joins the two branches of the North Tunnel to the West Tunnel. Hydraulic conditions in the tunnels control the flow through the junction structure. No active control is necessary for operation.

No special maintenance problems are anticipated as a result of the design of the drop structures, dropshafts and junction structure. Inspection of these components of the drainage system can be performed concurrent with tunnel inspection. Maintenance would be performed as necessary. Criteria similar to those used for the tunnels can be applied to drop structure, dropshaft and junction structure maintenance and inspection.

The helicoidal dropshafts used on the West Tunnel and at Central Avenue are a new type of structure. Velocities obtained in model tests indicate that the helicoidal dropshaft will be self-scouring in normal operation. The helicoidal ramps are fabricated of steel, and coated with a coal-tar epoxy to help prevent oxidation. The ramps are designed to last for the operational life of the dropshaft. However, if it is necessary to perform maintenance on the ramps, they can be lifted out of the dropshaft.

Access to the dropshaft sites along the East Tunnel is provided by maintenance pullouts along the I-10 roadway. In addition, access roads are provided at the following locations:

- Off 21st Place at Pierce Street, for the Pierce Street dropshaft.

- Off Ramp N-E at Buckeye Road, for the Buckeye Road dropshaft.

Access to the West tunnel dropshafts is by manholes set in city streets at the appropriate locations. Access can be gained to the North Tunnel drop structures and dropshafts from city streets, except for the 3rd Avenue and 3rd Street dropshafts. These two dropshafts can be accessed from their respective pump station locations.

IV. SURGE RELIEF FACILITIES

Mathematical modeling performed by Dr. Song (described in Chapter 6 of this report) indicated a need for surge relief facilities. These facilities help absorb the transient pressures associated with the filling of the drainage tunnels, lessening street flooding during the design storm event.

Two important components in the surge relief system are the surge ponds located at the 13th Avenue inlet to the Storm Water Interceptor and at the Moreland Street drop structure. These ponds are normally dry, but accept overflow water during the temporary backflow in the tunnel system which occurs as the East and West Tunnels fill. The surge ponds should be maintained at their design elevations, surface areas, and volumes in order to insure their satisfactory performance. Some clean-up maintenance may be necessary at the ponds after backflow events.