Donohue ENGINEERS ARCHITECTS SCIENTISTS

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ARIZONA, U.S.A.

DRAINAGE REPORT FOR BELL ROAD

SECTION 6

Prepared by:

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I. INTRODUCTION

1.1 Purpose

This report was completed as a part of the Bell Road, Section 6 Improvements, Outer Loop to 67th Avenue. The purpose of this report is to describe design parameters, methodologies, and results of hydrologic and hydraulic analyses which were conducted to develop a drainage improvement plan for onsite and offsite runoff affecting the roadway.

1.2 Description of Improvements

Approximately 2 miles of Bell Road between the Outer Loop and 67th Avenue will be reconstructed or widened as a part of this project (see Exhibit 1). The planned modifications to the roadway will provide for an 8-lane urban arterial from the Outer Loop to 75th Avenue, and a divided 6-lane arterial from 75th Avenue to 67th Avenue. A grade separation structure (tramway), designed to provide access between the proposed retail centers north and south of Bell Road, will be constructed between 77th and 79th Avenues. The existing Bell Road bridge over Skunk Creek will be widened to provide sidewalks.

1.3 Concept Drainage Plans

The drainage improvements to be constructed as a part of this project will include:

1. a storm drain system beneath Bell Road to collect and dispose of roadway runoff;
2. two major storm drain outfalls for the conveyance of Bell Road, Westcor Mall, and offsite runoff emanating north of Bell Road, to Skunk Creek; and

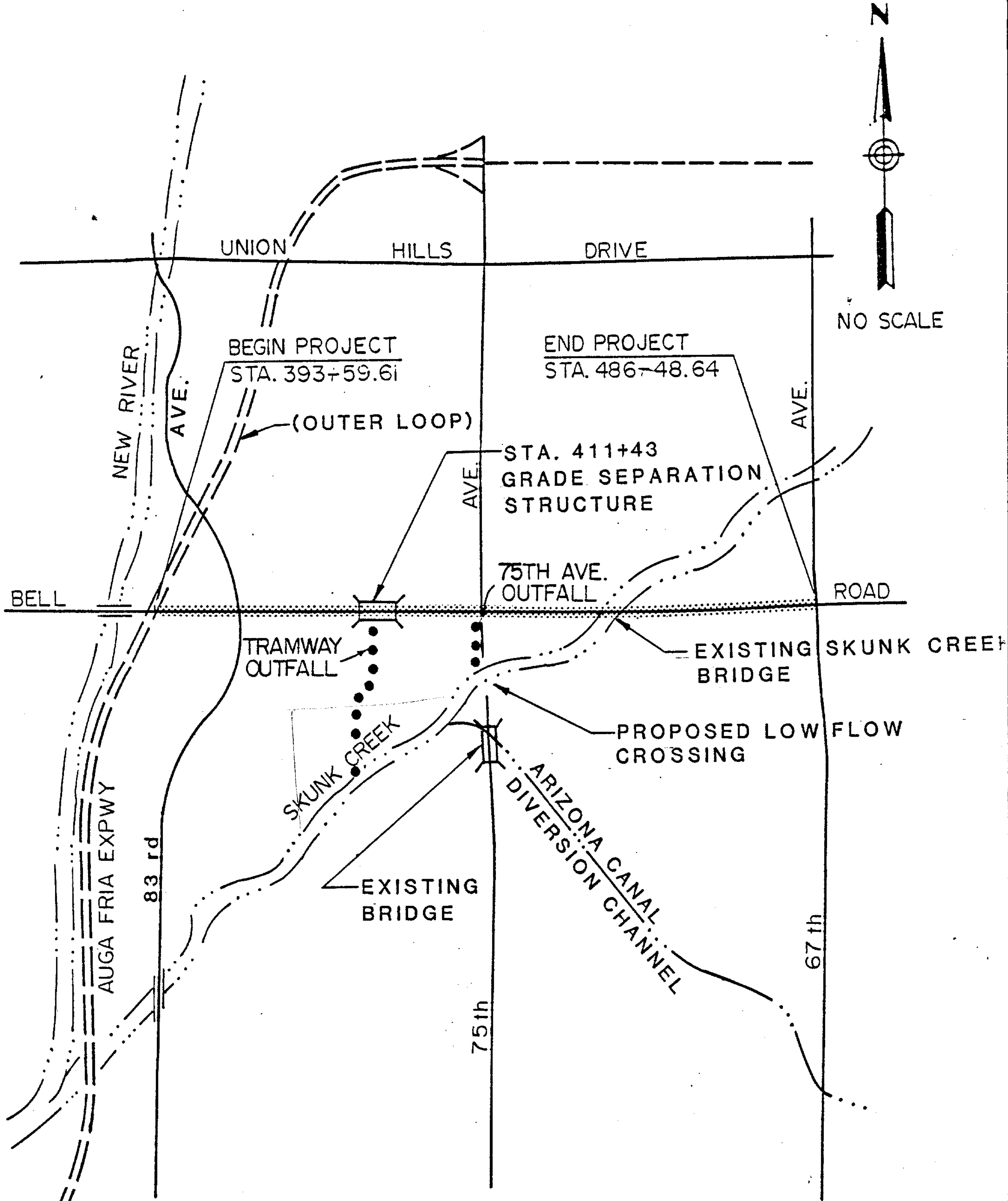

EXHIBIT I

LOCATION MAP

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3. minor modifications to the Skunk Creek bridge as necessary to ~~which are needed to control siltation and~~ protect the foundation from scour.

A complete list of the drainage design criteria and requirements for this project is given in Appendix A. The design capacity for the roadway storm drain system is based upon the 10-year, 24-hour rainfall event. Catch basin inlet spacing must be provided as necessary to maintain two dry lanes in each direction during the design event.

The storm drain outfalls to Skunk Creek will be located on the 75th Avenue alignment and on an alignment about mid-way between 75th Avenue and 83rd Avenue (see Exhibit 1). The 75th Avenue outfall will extend from Skunk Creek to Bell Road and then be stubbed out for a future extension by others. The diameter of this storm drain will be 78 inches and have sufficient capacity to carry the Bell Road storm drainage and runoff from an existing residential development between Bell Road and Union Hills Drive. The outfall which will be located about mid-way between 75th Avenue and 83rd Avenue (the Tramway Outfall) will extend from Skunk Creek to just north of Bell Road, where a stub out will be provided for a future extension into Westcor Mall. This outfall will have a diameter of 96 inches and will have adequate capacity to convey the roadway storm drainage and the 100-year runoff volume from Westcor Mall. The City of Glendale has agreed to the concept of increasing the capacity of the Tramway Outfall to convey future Westcor Mall runoff (without detention).

1.4 Scope of Work Outline

The hydrologic and hydraulic analyses conducted as a part of this design project included:

1. U.S.A.C.E. HEC-I model for offsite watersheds to determine the volume of runoff to be collected by the two proposed outfall structures.
2. Hydrologic computations for determination of runoff volume and catch basin locations for the roadway storm drainage system.
3. Normal depth computations for the roadway pavement drainage to determine lateral flow spread and storm drain inlet spacing.
4. Storm drain inlet capacity computations for determination of curb opening lengths.
5. Backwater computations for hydraulic design of the Bell Road storm drain system and for the major outfall structures.
6. Preparation of a U.S.A.C.E. HEC-II model for Skunk Creek along the reach extending 1000 feet downstream and 1600 feet upstream of the Bell Road bridge.
7. Hydraulic and geomorphic computations for determination of scour depth and grade stabilization measures at the Bell Road bridge crossing of Skunk Creek.

II. DRAINAGE AREAS AND HYDROLOGY

2.1 Drainage Area Characteristics

The section of Bell Road between Outer Loop and 67th Avenue receives offsite runoff from areas extending as far north as Union Hills Drive. Runoff from this area drains southwesterly as sheetflow or streetflow, until it reaches Skunk Creek or New River.

Offsite watersheds which drain toward Bell Road are mapped on Exhibit 2 of this report. Areas draining to the planned 75th Avenue outfall (defined on Exhibit 2 by the light shading), consists almost entirely of residential subdivisions between Bell Road and Union Hills Drive. Sub-area #1 (0.26 square miles), is the Hidden Manor subdivision located north of Grovers Avenue and between 75th and 79th Avenues. Runoff drains within the subdivision streets to 77th Avenue, then south toward St. Johns Drive.

Sub-area #2 is a 0.27 square mile residential subdivision located between 71st Avenue and 75th Avenue on the east and west, and between Grovers Avenue and Union Hills Drive on the south and north. Runoff from this sub-basin drains southwesterly within the subdivision streets to the intersection of Grovers Avenue and 75th Avenue.

Sub-area #3 is a 0.082 square mile area of residential subdivision located east of 75th Avenue and south of Grovers Avenue. Runoff from this subdivision area drains south and west to 75th Avenue. The point of concentration is at the intersection of 75th Avenue with Angela Drive.

Sub-area #4 is currently a vacant parcel at the northeast corner of the intersection of 75th Avenue and Bell Road. This analysis assumed existing watershed conditions for sub-area #4 and that development of this parcel will require onsite retention/detention to prevent further increases in runoff (per City of Glendale requirements). The area of sub-area #4 is 0.058 square miles.

The principal areas draining to the Tramway outfall are sub-areas of the planned Westcor Mall. Runoff from sub-areas north of Bell Road (NW Mall and NE Mall) will drain as parking lot flow or storm drainage to a future point of inlet to the Tramway outfall within the mall. Runoff from the Power Center (PWRCTR) south of Bell Road, will drain southeasterly as parking lot flow or storm drainage to a future point of inlet to Tramway outfall.

Other small sub-areas which were defined for the purpose of the hydrologic analysis included 79th Avenue north of Bell Road, Paradise Lane, and Bell Road from a point just east of the intersection with 83rd Avenue to 75th Avenue. Each of these sub-areas consist entirely of street runoff which will drain within the street cross section to the Tramway outfall.

Table 1 summarizes the drainage basin characteristics for each of the sub-watersheds described above and mapped on Exhibit 2.

TABLE 1 - SUMMARY OF BASIN CHARACTERISTICS

Watershed	Area (acres)	Type/Cover	Curve #	Roughness Coefficient	Slope (ft/ft)
TRAMWAY OUTFALL					
N.E. Mall	122	Commercial	95	0.17	0.005
N.W. Mall	70	Commercial	94	0.17	0.005
1400*	192	--	--	--	--
79N	13	Street	95	0.10	0.020
BellW	5	Street	95	0.10	0.020
1300*	19	--	--	--	--
BellC	5	Street	95	0.10	0.020
1200*	211	--	--	--	--
PWRCTR	37	Commercial	95	0.17	0.025
PARADS	2	Street	95	0.10	0.020
1100*	250	--	--	--	--
75TH AVENUE OUTFALL					
SUB 1	166	Residential	84	0.17	0.002
SUB 2	173	Residential	84	0.17	0.002
1300*	339	--	--	--	--
StJhn	4	Street	95	0.10	0.020
SUB 3	52	Residential	84	0.17	0.004
1200*	397	--	--	--	--
SUB 4	37	Agricultural	77	0.17	0.006
BellE	5	Street	95	0.10	0.005
1100*	403	--	--	--	--

*Point of hydrograph combination

2.2 Offsite Hydrology

HEC-I rainfall/runoff models were prepared in order to estimate peak runoff rates from watersheds described in Section 2.1 of this report. Peak flows draining to the proposed outfall systems were determined using the kinematic wave routing routine provided in the HEC-I model. Flow elements included roof or parking lot areas modeled as overland flow elements and collector or main drains. The proposed outfalls were also included in the kinematic wave routing routine. Exhibits 3 and 4 of this report are schematic drawings of the HEC-I models for the 75th Avenue and Tramway outfalls.

A sensitivity analysis was performed to assess changes in discharge with the parameters of slope, length, roughness, bottom width and sideslopes. Results of this analysis are discussed in a November 19, 1990 memorandum to file provided in Appendix 2 along with HEC-I model input/output.

The design storms which were analyzed for the HEC-I rainfall/runoff models included the:

1. 25-year, 30-minute and 24-hour storms;
2. 100-year, 30-minute, 2-hour and 24-hour storms; and
3. 10-year, 2-hour and 24-hour storms.

Table 2 provides a summary of the HEC-I rainfall/runoff model results for all storms which were analyzed. The analyses for 25-year and 100-year return period events were done to determine the maximum runoff rate from the Westcor Mall (and other

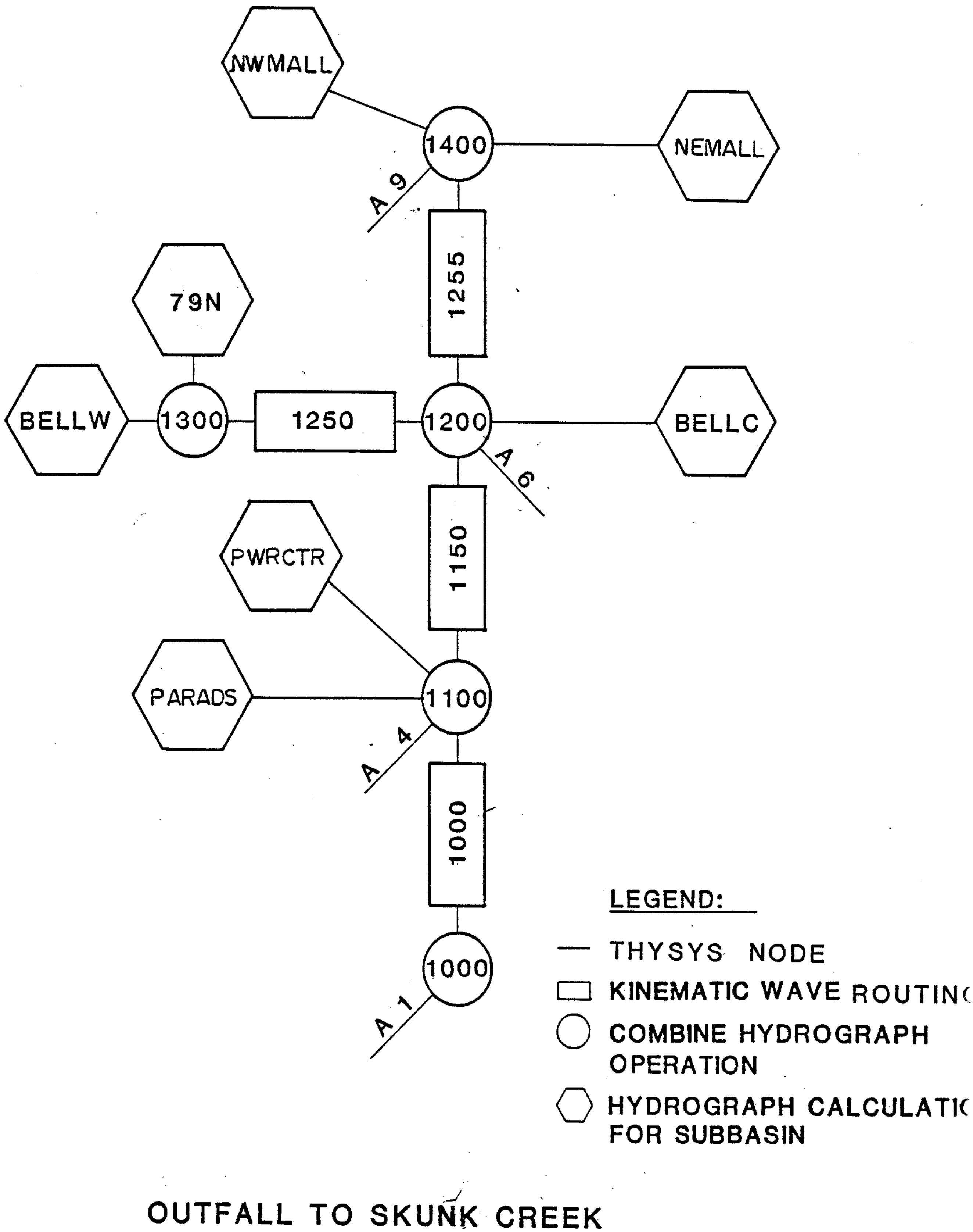

EXHIBIT 3

HEC-I SCHEMATIC FOR THE TRAMWAY OUTFALL

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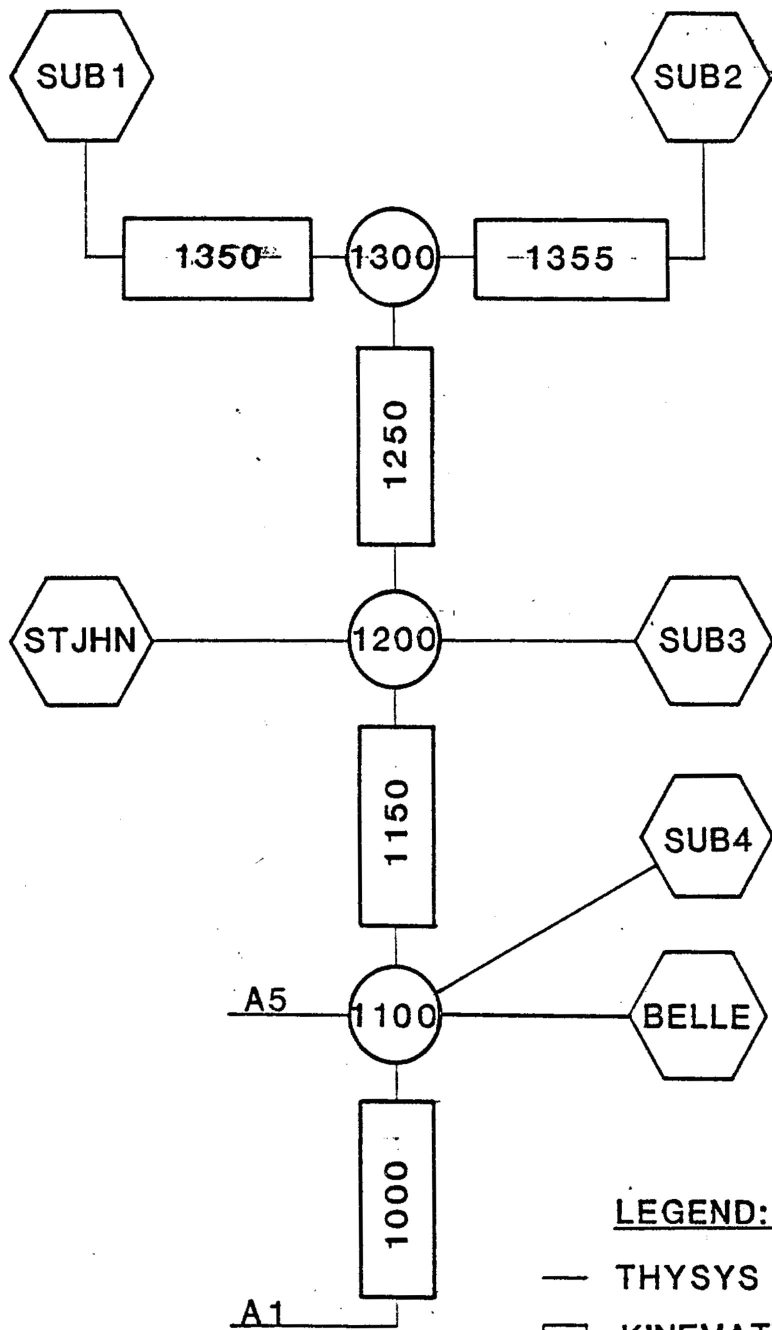

LEGEND:

— THYSYS NODE

▭ KINEMATIC WAVE ROUTING

◯ COMBINE HYDROGRAPH OPERATION

⬡ HYDROGRAPH CALCULATIO FOR SUBBASIN

OUTFALL TO SKUNK CREEK AT 75TH

EXHIBIT 4

HEC-I SCHEMATIC FOR THE 75th AVE. OUTFALL

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TABLE 2 - SUMMARY OF HEC-I RAINFALL/RUNOFF MODEL RESULTS

WATERSHED	Q10 (2hr)	Q10 (24hr)	Q25 (30min)	Q25 (24hr)	Q100 (30min)	Q100 (2hr)	Q100 (24hr)
TRAMWAY OUTFALL							
N.E. Mall	107	173	121	227	218	277	306
N.W. Mall	60	96	64	126	116	157	172
1400*	168	268	185	353	325	432	478
79N	26	19	33	24	52	49	32
BellW	12	7	17	9	24	22	12
1300*	35	27	46	33	75	71	44
BellC	13	9	19	11	28	25	14
1200*	205	302	236	396	405	510	536
PWRCTR	32	52	36	68	62	84	93
Parads	6	3	9	4	13	11	5
1100*	238	353	266	467	458	590	633
75TH AVENUE OUTFALL							
SUB 1	--	129	--	--	--	329	--
SUB 2	--	111	--	--	--	324	--
1300*	--	238	--	--	--	642	--
StJhn	--	7	--	--	--	11	--
SUB 3	--	44	--	--	--	106	--
1200*	--	277	--	--	--	748	--
SUB 4	--	11	--	--	--	46	--
BellE	--	7	--	--	--	12	--
1100*	--	285	--	--	--	793	--

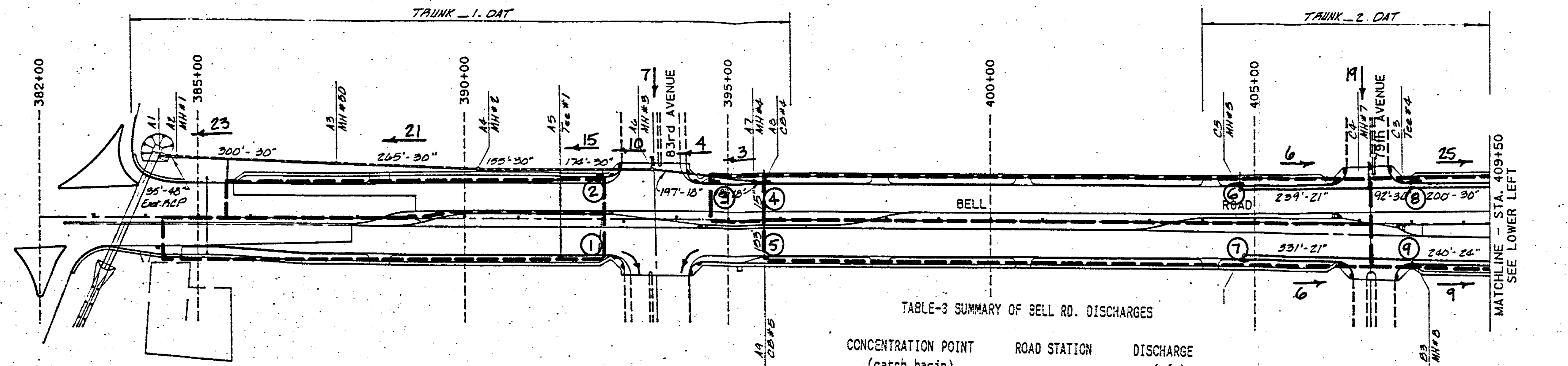
* Point of combined hydrographs

contributing areas). Examination of the HEC-I model results for each of these storm events found that the 100-year frequency, 2-hour duration storm resulted in the greatest peak runoff rate. This design frequency/duration was used for the Tramway outfall design. The 75th Avenue outfall capacity was determined based upon the results from the 10-year, 24-hour duration storm.

2.3 Bell Road Hydrology

The rational equation was used to calculate roadway pavement flows for inlet design and storm sewer hydraulics for the Bell Road storm sewer system. The design storm for the Bell Road storm drainage system was the 10-year, 24-hour storm event as specified by the City of Glendale. Runoff coefficients used for the roadway drainage were 0.90 for pavement surfaces, and 0.60 for landscaped areas. A minimum time of concentration of 10 minutes was assumed in all cases because of the relatively short length of flow to a catch basin inlet point. The longest length of travel to a catch basin inlet is approximately 1200 feet.

Table 3 (on Exhibit 5) provides a summary of the Bell Road pavement drainage. Drainage basin boundaries and the location of concentration points (which are catch basin inlets) are shown on Exhibit 5. Appendix 3 contains a summary of calcultions sheet showing the parameters used for the discharge calculations.



LEGEND

- BASIN BOUNDARIES
- ② CONCENTRATION POINT
- 23 ACCUMULATED STORM DRAIN DISCHARGE

TABLE-3 SUMMARY OF BELL RD. DISCHARGES

CONCENTRATION POINT (catch basin)	ROAD STATION	DISCHARGE (cfs)
1	391+80	5.47
2	392+60	4.79
3	394+70	0.78
4	395+70	1.33
5	395+70	0.99
6	404+71	5.99
7	404+79	5.99
8	408+10	3.06
9	408+10	2.51
10	414+00	4.08
11	414+00	4.13
12	419+65	2.37
13	419+65	2.05
14	421+86	9.01
15	421+84	7.87
16	434+60	3.06

N

0 100 200
SCALE IN FEET

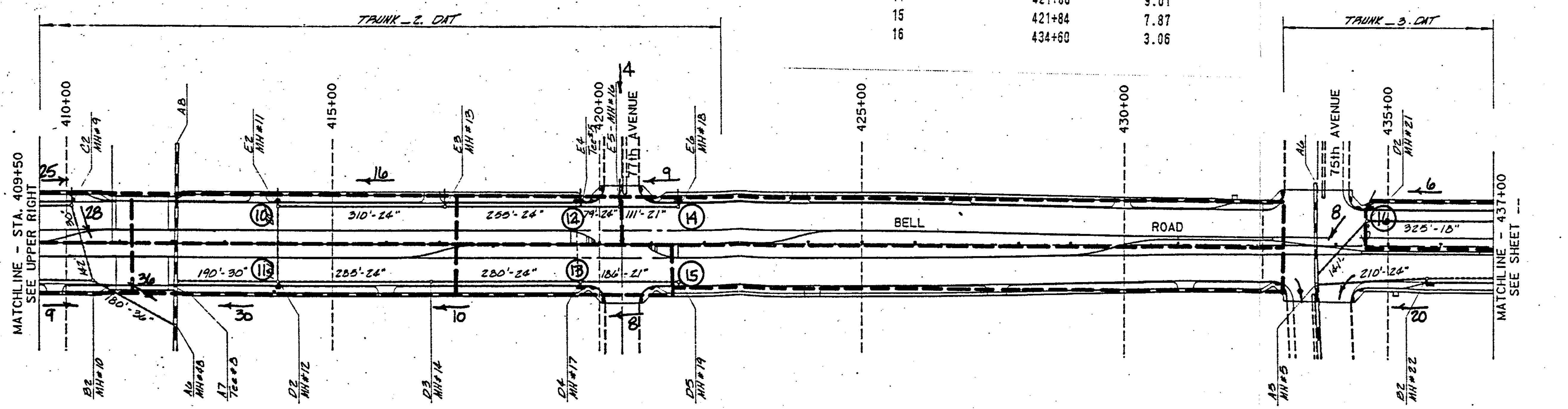
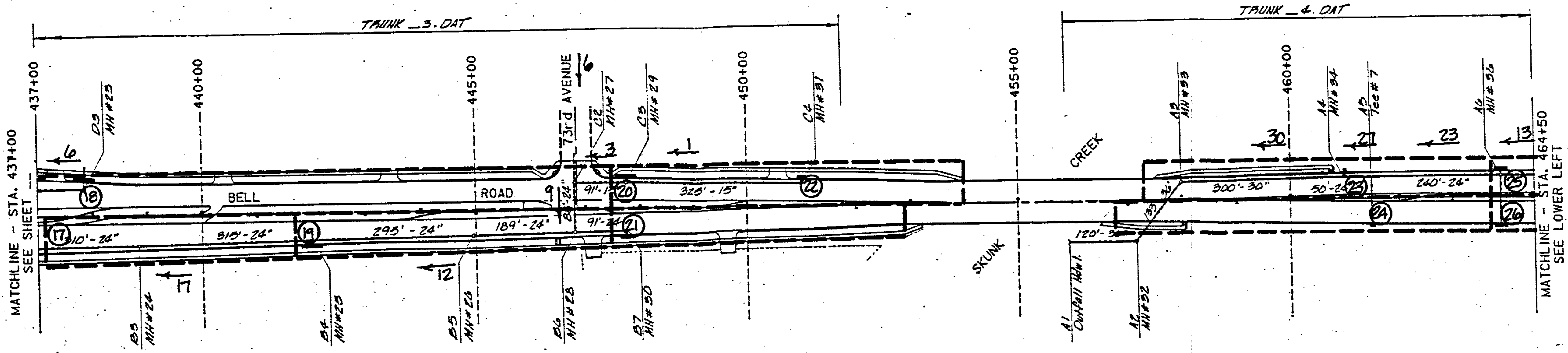


EXHIBIT 5- BELL ROAD DRAINAGE BASINS AND CONCENTRATION POINTS



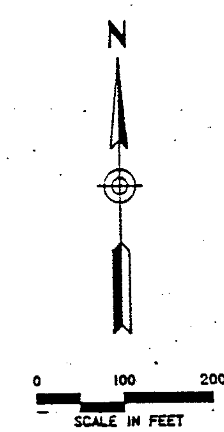


TABLE-3 SUMMARY OF BELL RD. DISCHARGES

CONCENTRATION POINT (catch basin)	ROAD STATION	DISCHARGE (cfs)
17	437+45	2.83
18	438+45	5.07
19	442+00	5.31
20	447+45	1.74
21	447+45	2.92
22	450+95	1.23
23	461+00	3.35
24	461+50	3.07
25	463+90	5.03
26	463+90	5.03
27	474+10	3.47
28	474+10	3.47

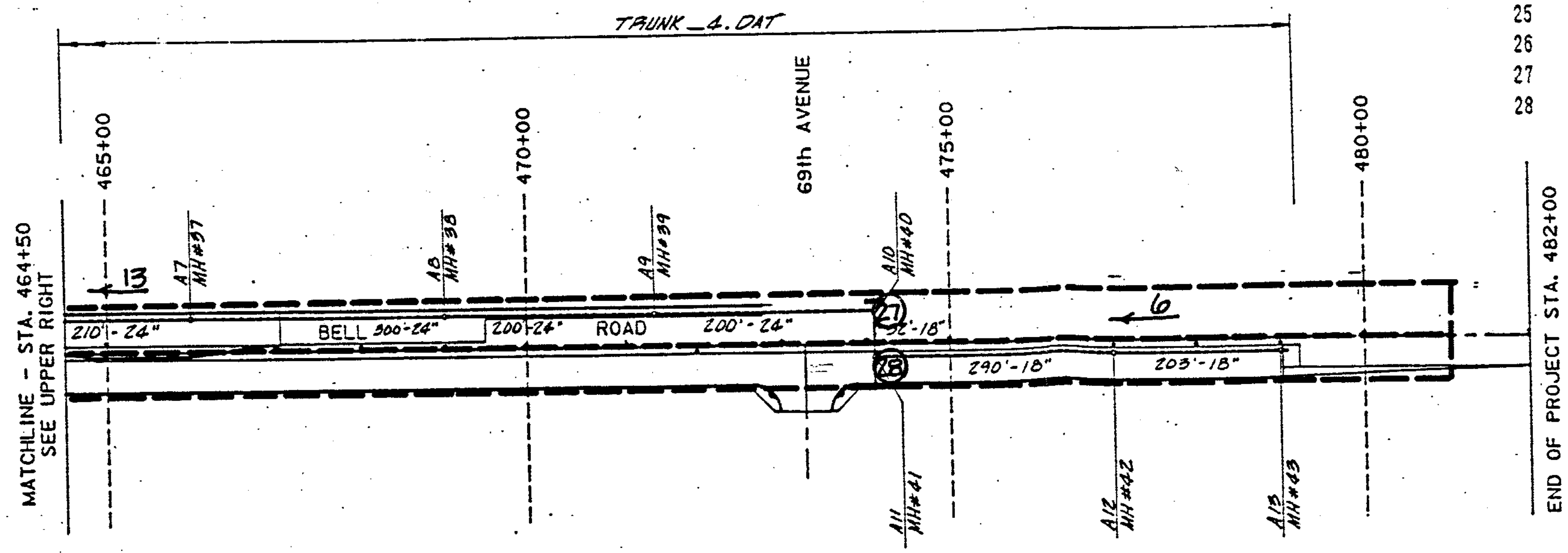


EXHIBIT 5 - BELL ROAD DRAINAGE BASINS AND CONCENTRATION POINTS

III. PAVEMENT DRAINAGE

3.1 <u>General Description</u>

A storm sewer system will be installed to provide pavement drainage for Bell Road. The capacity of this system will be adequate to convey runoff from the pavement surface and adjoining areas within the right-of-way such as landscaping, sidewalks, and driveways for the 10-year, 24-hour event.

The location and spacing of catch basins was established based upon the following criteria:

1. Catch basins were provided at all sumps in the pavement profile.
2. Catch basins were placed at locations on continuous grade segments of the roadway where the lateral spread of accumulated flows left two dry lanes during the design storms. Using this criteria, the allowable spread for pavement drainage is 28.5 feet (two exterior lanes plus curb and gutter) for the section of roadway between 83rd Avenue and 75th Avenue. For the section of roadway between 75th Avenue and 67th Avenue, the allowable spread is 16.5 feet (right lane plus curb and gutter). The only location within the section of roadway between 75th Avenue and 67th Avenue where the allowable spread is 28.5 feet, is on the westbound side of the roadway just before 75th Avenue (at a turn lane).

3.2 <u>Inlet/Catch Basin Location and Size Determination</u>

Table 4 provides a summary of the catch basin locations, required length of opening, length of opening provided, and structure type. The methodology used to determine

TABLE 4 - CATCH BASIN SIZE COMPUTATION RESULTS

Inlet #	Inlet Station	Length of Opening Required (including clogging factor)	Structure Type	Length of Opening Provided
1	391+80	5.9 ft	M-1,L=6'	9 ft
2	392+60	5.7	M-1,L=6'	9
3	394+70	11.3	M-1,L=10'	13
4	395+70	16.3	M-1,L=17'	20
5	395+70	11.0	M-1,L=10'	13
6	404+71	7.2	M-1,L=6'	9
7	404+79	8.1	M-1,L=6'	9
8	408+10	18.1	M-1,L=17'	20
9	408+10	16.3	M-1,L=17'	20
10	414+00	4.7	M-1,L=6'	9
11	414+00	4.7	M-1,L=6'	9
12	419+65	2.0	M-1,L=6'	9
13	419+65	19.4	M-1,L=17'	20
14	421+86	10.2	M-1,L=10'	13
15	421+84	9.5	M-1,L=10'	13
16	434+60	21.9	M-2,L=10'	23
17	437+45	18.1	M-1,L=17'	20
18	438+45	28.8	M-2,L=17'	37
19	442+00	28.8	M-2,L=17'	37
20	447+75	25.0	M-2,L=17'	37
21	447+75	25.0	M-2,L=17'	37
22	450+95	11.0	M-1,L=10'	13
23	461+00	3.6	M-1,L=6'	9
24	461+50	3.8	M-1,L=6'	9
25	463+90	23.1	M-2,L=10'	23
26	463+90	25.6	M-2,L=17'	37
27	474+10	28.1	M-2,L=17'	37
28	474+10	24.4	M-2,L=17'	37

the catch basin lengths is based upon procedures presented in FHWA Hydraulic Engineering Circular #12, "Drainage of Highway Pavements." The initial step of the computational procedure was to determine the lateral flow spread for pavement drainage being collected and conveyed toward the catch basin opening. This determination was made regardless of whether the catch basin type was on continuous grade or a sump because normal depth flow conditions precede inlet conditions and the allowable spread criteria applies to all sections of roadway pavement. The methodology used to determine the actual flow spreads was Chart #3 of Hydraulic Engineering Circular #12. This nomograph determines flow spread based upon slope, cross-slope, and discharge. Copies of the nomograph charts used for the lateral spread determination along with a summary of results sheet are provided in Appendix 3 of this report.

Two subsequent steps were then undertaken to determine the required inlet length for catch basins on a continuous grade. The first step involved the use of Hydraulic Engineering Circular #12, Chart 4, to determine the ratio of frontal flow to total gutter flow. This chart is used to determine the percentage of flow (E_0) within the gutter width (1.5 feet). The inlet efficiency increases with the greater quantity of flow conveyed within the gutter section. The next step of the computational procedure was to determine the equivalent cross-slope (S_E) for composite pavement cross sections. The equation for S_E is given on Hydraulic Engineering Circular #12, Chart 9, which was used to determine the required catch basin opening lengths. Chart 9 determines catch basin length based upon Mannings n value, slope, equivalent cross-slope, and design discharge. Copies of Chart #9, which were used to determine the catch basin lengths, are provided in Appendix 3 of this report. A summary sheet of parameters is also provided therein.

The methodology used to determine the required length of inlet for catch basins in a sump is given by Hydraulic Engineering Circular #12, Chart 12. The design discharge value and flow depth (given by the normal depth analysis of Chart 3) were plotted on Chart 12. A value of P was determined from Chart 12, then the equation $P = L + 1.8W$ was rearranged to solve for L (the required catch basin opening length). Examination of the chart provided in Appendix 3 will show that minor extrapolation was necessary to solve the equation because the depths at the catch basin inlets fall just above the boundary for weir flow conditions, but well below the boundary for orifice conditions. Weir flow conditions were assumed to give slightly conservative results.

3.3 Storm Drain Design

The storm sewer system is designed to satisfy the project requirements as specified by City of Glendale standards and to minimize cost. Hydrologic routing was not conducted to lag hydrographs because the time of concentration to all inlets was less than 10 minutes and the greatest length of travel to an outfall point is 1700 feet. Discharges were determined by summing the sub-basin flows given in Table 3 along with offsite flows determined from the HEC-I analyses.

Exhibit 5 provides a small-scale plan view of the Bell Road storm sewer system. Pavement drainage collecting on the roadway surfaces between the Outer Loop and a point approximately 300 feet east of the 83rd Avenue intersection will drain into a storm sewer system which will convey flow west to an outfall channel running parallel to the Outer Loop. Inlets to this storm drain will include catch basins #1 through #5, and a future lateral collecting 7 cfs (Table 2, CP-BELLW) of runoff draining south from 83rd Avenue. The cumulative quantity of water within this storm drain at its outfall is 23 cfs.

Runoff from Bell Road along the segment between 83rd Avenue and the Tramway will be collected within a storm drain beginning at a point just west of 79th Avenue. Dual, parallel storm drain trunks will be used along this section of roadway to minimize the number of laterals crossing Bell Road. Inlets to this storm drainage system will include catch basins #6 through #9, and a future lateral connection at 79th Avenue which will collect 19 cfs (Table 2, CP-79N) draining south within 79th Avenue to the Bell Road intersection. The cumulative quantity of water in this storm sewer system at its point of outfall is 36 cfs.

Pavement runoff from the section of roadway between the Tramway and 75th Avenue will be collected in another dual, parallel trunk system and conveyed west to the Tramway outfall. Inlets to this storm drain system will include catch basins #10 through #15. The upstream end of this storm drain system will be catch basins #14 and #15 which are located just east of the intersection with 77th Avenue. No additional storm drain inlets were provided between 77th Avenue and 75th Avenue because the lateral spread for the design runoff quantities did not exceed the allowable limit. The cumulative quantity of water discharging from this storm drain system at its outfall was determined to be 30 cfs, which includes 4 cfs of offsite flow from 77th Avenue.

Pavement runoff from the section of roadway between 75th Avenue and Skunk Creek will be collected within a storm drain system and conveyed westerly to an outfall beneath 75th Avenue. Inlets to this storm drain system include catch basins #16 through #22, and a future lateral at 73rd Avenue which will collect 6 cfs, draining south to Bell Road within 73rd Avenue. A short section of dual, parallel trunk will be used for the section of roadway extending approximately 400 feet east of the 75th Avenue intersection. The storm drain for the north side of the roadway will collect runoff from catch basins #16 and #18 and have a total discharge at its point of outfall to the 75th Avenue storm drain of 8 cfs. The storm drain constructed along the south side of the roadway will include runoff from pavement surfaces on the south side of the road between 75th Avenue and 73rd Avenue as well as runoff from both sides of the roadway for that section extending east of 73rd Avenue. The cumulative quantity of water for the storm drain on the south side of the roadway at its point of outfall was determined to be 20 cfs.

Runoff from the pavement along the section of roadway between Skunk Creek and a point about 800 feet east of 69th Avenue will be collected within a storm drain system that conveys runoff west to Skunk Creek. Inlets to this storm drain system will include catch basins #23 through #28. The storm drain capacity also provides for an additional 6 cfs which will be conveyed into the system at its east end by an adjoining roadway project being designed by Stanley Engineers. The cumulative quantity of water collected by this storm drain system at its point of outfall to Skunk Creek was determined to be 30 cfs.

The January 1989 version of the program THYSYS, a public domain drainage design aid developed by the Texas State Department of Highways was used to evaluate alternatives and conduct the hydraulic analyses for the storm drain systems. Documentation for the THYSYS program is included in Appendix 4 of this report along with computer model input and output (on disk). The THYSYS hydraulic grade line (HGL) calculations for the roadway drainage system are based upon Mannings equation, assuming either open channel flow or pressure flow with losses computed as a function of friction slope. The HGL calculations for the storm drain systems were set equal to the 10-year water surface elevation within the Skunk Creek channel or the design water surface elevation within the outfall, whichever applies. The outfall invert elevations and flowline profiles for the storm drain systems were set based upon available grade and utility conflicts. Plan and profile drawings of the storm drain system along with the computed hydraulic grade line for the Tramway and 75th Avenue outfalls are provided in Appendix 5 of this report.

3.4 Street Capacity

The capacity of selected street cross sections to carry runoff within the right-of-way has been estimated and summarized on Table 4 of this report. The street drainage capacity of all cross sections is sufficient to carry the 100-year storm at a depth of 1.5 feet above top of curb except at 79th Avenue south of Bell Road where a historic swale to Skunk Creek is being supplanted by retail development.

TABLE 5 - SUMMARY OF STREET CAPACITY ESTIMATES[1]

Road Segment	So (ft/ft)	Flow[2] Area (SF)	V (fps)	Q[3] (cfs)	Q Max[4] (cfs)
79th Ave. north of Bell Rd.	.005	27	2.9	77	600
75th Ave., Union Hills Dr., Skunk Creek	.005	28	2.9	79	560
83rd Ave. south of Bell Rd.	.005	22	2.2	49	411
77th Ave., Paradise Lane	.005	21	2.7	56	340
Bell Rd., 75th Ave. to 83rd Ave.	.004	58	3.4	198	1023

Notes: [1]Capacity based on approximations of roadway prisms - templates vary
N = 0.016 (does not include storm drain capacity)
[2]Flow area at top of curb
[3]Q to top of curb
[4]Q at 1.5 feet above top of curb within right-of-way

IV. OFFSITE DRAINAGE

4.1 General Description

Offsite runoff emanating from areas north of Bell Road will be conveyed to Skunk Creek within two storm drain outfall systems to be constructed as a part of this project. These outfalls will also be the point of discharge for elements of the Bell Road storm drainage facilities between 83rd Avenue and the Skunk Creek bridge. The Tramway outfall will cross Bell Road in the area of the Westcor Mall. It will follow an alignment essentially due south along the east side of the PWRCTR property past Paradise Lane, then turn southwesterly for a distance of approximately 500 feet, then due south again to Skunk Creek. The Tramway outfall will collect runoff from the Westcor Mall and PWRCTR properties, and runoff collected by the Bell Road storm drain system between 83rd Avenue and 75th Avenue. The pipe is sized to convey the 100-year, 30-minute; or the 100-year, 2-hour storm; whichever is greater (as required by the City of Glendale) for runoff emanating from the Westcor Mall property and other offsite contributing areas. Hydrologic model results (HEC-I) determined that the 100-year, 2-hour storm produces the largest peak.

The other major outfall structure will be located beneath 75th Avenue. This outfall system is designed to convey runoff from the 10-year, 24-hour storm event. The drainage area for the 75th Avenue outfall includes 403 acres of offsite development north of Bell Road (see Exhibit 2) and runoff from Bell Road between 75th Avenue and the Skunk Creek bridge.

Runoff from the section of Bell Road west of 83rd Avenue will be conveyed via a new storm drain system to an ADOT channel constructed as a part of the Outer Loop project. The specific point of discharge will be into a box culvert beneath Bell Road. It will then drain south into an open channel that will convey runoff to New River.

4.2 Coordination With Adjacent Design Sections and Local Agencies

Records of correspondence with adjacent design sections and local agencies are provided in Appendix 6 of this report. This includes:

a. authorization from ADOT to utilize the Outer Loop drainage facilities for discharge of Bell Road storm drainage west of 83rd Avenue,

b. Maricopa County Flood Control District approval on the conceptual design of the Tramway outfall, and

c. City of Peoria approval of the proposed Tramway outfall alignment.

4.3 Outfall Design

The Tramway outfall will have a diameter of 96 inches between its point of discharge to Skunk Creek and Paradise Lane (a total distance of 1,482 feet). The design capacity for this section of pipe is 590 cfs. This discharge is the runoff from all contributing areas during a 100-year, 2-hour duration storm. Between Paradise Lane and Bell Road, the diameter of the Tramway outfall will be 84 inches (1500 feet), and the design capacity for this section will be 510 cfs. The change in design discharge and pipe diameter at Paradise Lane reflects an addition of 80 cfs of runoff from the PWRCTR property and from a section of Paradise Lane.

The 75th Avenue outfall will have a diameter of 78 inches between its point of discharge to Skunk Creek and Bell Road (a total distance of 900 feet). The design capacity for the 75th Avenue storm drain is 285 cfs, which is the runoff volume from the contributing drainage area during the 10-year, 24-hour duration storm event.

THYSYS model input and output for the Tramway and 75th Avenue outfalls is provided in Appendix 4. Plots of the hydraulic grade lines are provided in Appendix 5.

V. SKUNK CREEK CROSSING

5.1 General Description

The existing Bell Road bridge over Skunk Creek will be widened to provide sidewalks. This widening will require a horizontal extension of the existing piers, and these structures have been identified as being vulnerable to exposure and possibly even failure by scour. Hydraulic and geomorphic analyses were conducted as a part of this study to establish flow conditions in the vicinity of the bridge structure, estimated scour depth and determine if measures are needed to stabilize the bridge foundation.

5.2 Skunk Creek Hydrology

The 100-year peak flow for Skunk Creek is 13,000 cfs upstream of the ACDC confluence. This value was supplied by the Maricopa County Flood Control District at initial meetings with District personnel in March, 1989. MCFCD is currently engaged in a restudy which will lower the 100-year discharge to 11,000 cfs. HEC-II runs are provided for the 11,000 cfs discharge and compared with the ongoing FEMA restudy, however, the hydraulic conditions for the 13,000 cfs discharge are used for design purposes.

A 10-year peak discharge value for Skunk Creek was not available from the MCFCD. Based upon conversations with the MCFCD staff (John Smeltzer), the assumed 10-year peak discharge values were calculated to be 50% of the 100-year peak discharge value.

5.3 Hydraulic Analysis

Hydraulic conditions for Skunk Creek were determined using the U.S. Army Corps of Engineers computer program HEC-II. The limits of this analysis extended from a point 1,000 feet downstream to a point 1,600 feet upstream of the Bell Road bridge. The point of beginning for the hydraulic model (1,000 feet downstream of Bell Road) was at a contraction in the channel width where flow passes through critical depth (identified in previous Flood Insurance Study modeling). This was established as the limit of modeling since downstream hydraulic conditions do not affect flow conditions at the Bell Road Bridge.

The cross section interval used in the HEC-II modeling averaged about 300 feet. A closer interval was used in the vicinity of the Bell Road bridge with 4 cross sections being placed at the entrance and exit transition points and on the upstream and downstream sides of the bridge in accordance with the geometry defined in the application of HEC-II bridge routine references (HEC, June 1974). The model results found that the Bell Road bridge will operate as Class A low flow during the 100-year event (13,000 cfs).

The HEC-II model input/output (hard copy and disk) are provided in Appendix 7 of this report. Also provided therein is a copy of portions of the mapping and computer model input/output for the analyses conducted as a part of the most recent Flood Insurance Study by Coe & Van Loo Engineering (MCFCD Contract #89-72). Comparison of the FIS model with that of the model prepared as a part of this study show close agreement in water surface elevations at the bridge.

Donahue's 100-year water surface elevation on the upstream side of the bridge is 1215.25, while Coe and Van Loo's is 1215.27. Exhibit 6 shows both the Donohue and Coe and Van Loo floodplain limits. In most areas, the difference in delineations is indiscernable.

The bridge modifications to be implemented as a part of this study propose some minor excavation of bridge cells to increase conveyance capacity and reduces siltation. This work will also repair some of the alterations to the Skunk Creek channel caused by previous construction activities. These grading modifications propose lowering the flowline elevation through the bridge structure of the 3 westernmost and 3 easternmost cells to an elevation of 1208.5. The center 3 cells would remain at their current elevation of approximately 1207.5. Drawings of the proposed bridge modifications are provided in Appendix 9.

Comparison of the hydraulic model for the with-project (graded) conditions shows that the proposed channel bottom elevation changes will result in a slight decrease in water surface elevations and velocity through the bridge. The decrease in water surface elevation on the upstream side of the bridge structure during the 100-year event was determined to be 0.32 feet, and the velocity decreased from 10.50 to 10.27 fps. Similar changes also occur at the cross section on the downstream side of the bridge. The HEC-II model input/output for the improved bridge opening condition is provided in Appendix 7 of this report.

5.4 Scour Analysis

Channel bed scour at the Bell Road bridge was computed using procedures presented in the U.S. Department of Transportation Federal Highway Administration publication, Interim Procedures for Evaluating Scour at Bridges (Sept. 1988). Two cases were evaluated to determine scour at the bridge piers and at the bridge abutments. Scour at the bridge structure can also occur as a result of the contraction caused by the bridge aperture. The computed contraction scour is added to the bridge pier and abutment scour depths to determine total scour depth at each structure. The discharge value used for the scour computations was 13,000 cfs (100-year event).

Contraction scour was computed using Case 2, Equation 1 of the FHWA Interim Procedures. This equation bases contraction scour depth on the change in width of flow and upstream flow depth. This equation estimated the contraction scour depth at the Skunk Creek Bridge to be 1.25 feet. This minor depth of contraction scour is consistent with expected results since the Skunk Creek bridge is not a significant contraction to the upstream or downstream flow width.

Pier scour depth was computed using Equation 12 (Chang) of the FHWA Interim Procedures. This equation determines pier scour depth based upon pier width, pier shape, flow angle of attack upon the pier, and depth of upstream flow. This equation estimated the pier scour depth at the Skunk Creek bridge to be 6.8 feet. Total pier scour depth was determined to be 8.0 feet, with the addition of contraction scour.

Abutment scour was computed using Equation 5 (Liu and Gill) of the FHWA Interim Procedures. This equation relates abutment scour to the upstream depth of flow abutment length and froude number. This equation estimated the abutment scour depth at the Skunk Creek bridge to be 15.3 feet. This result was added to the estimated contraction scour depth of 1.25 feet to give a total estimated abutment scour depth of 16.55 feet.

Computation sheets for the above-stated results are provided in Appendix 8 of this report. Appendix 8 also contains excerpts from the project soils report which identified materials comprising the channel bed to consist of silty sand and gravel overlying a clayey sand. FHWA and ADOT procedures normally require use of the 500-year discharge for scour analyses. It was not used for this analysis since results for the 100-year event already established that the foundation is vulnerable to exposure at a lower discharge.

5.5 Bank Stabilization and Scour Protection

Hydraulic and geomorphic analyses conducted as a part of this study have identified that the stability of the Skunk Creek bridge is questionable during high-magnitude flow events. The existing bridge foundation is at elevation 1198.0 while the pier and abutment scour depths may reach elevation 1199.5 and 1190.9, respectively, during the 100-year flood. Examination of the downstream channel profile have further identified that a headcut is propagating upstream and could ultimately result in 3 to 4 feet of long-term degradation through the bridge structure. This degradation potential along with scour during high-magnitude events lead to the possibility of a partial or complete bridge failure during future flows. As such, it is recommended that bottom protection be provided through the bridge structure to protect against both short-term and long-term scour potential. The recommended approach proposes lining the channel bottom through the bridge structure with 6-inch thick concrete reinforced with a welded wire fabric. A plan view and cross section of the proposed stabilization measures are shown in Appendix 9 of this report. The horizontal limits of the concrete bottom protection extend approximately 24 feet beyond the downstream face (future) of the bridge and 20 feet beyond the upstream face of the bridge. These distances are being recommended to incorporate protection of existing and proposed telephone, gas, and water line crossings of Skunk Creek which are located immediately beyond the bridge structure (see Appendix 9). These utility lines should be protected since scouring caused by the bridge piers and by a vertical drop on the downstream side of protection could cause their failure.

The concrete bottom lining should have a 6-foot toe down on its upstream side and 12-foot toe down on the downstream side. The 6-foot depth on the upstream side is a minimum toe down depth since this side of the structure is not affected by channel bottom scouring. The toe down depths for the downstream edge of the concrete bottom lining is 12 feet. This depth was determined based upon the potential scour hole which could form as a result of flow spilling over a 4-foot drop created by downstream channel bed degradation. Scour computation conducted as a part of this report estimated the potential scour depth due to flow over this drop to be 11 feet. An additional 1 foot depth of toe down (ie. a total of 12 feet) provides a factor of safety.

The horizontal limit for the proposed concrete bottom lining extends outward at the southeast corner of the bridge to include the 36-inch storm outfall (see Appendix 9). This extension will provide a splash pad (scour protection) for the 36-inch storm drain outfall point and will provide an effective means of incorporating an outfall headwall into the bridge abutment stabilization system.

The outlet of the Tramway outfall to Skunk Creek will be stabilized with soil cement bank protection. Existing soil cement bank protection ends approximately 50 feet upstream of the outlet point and this protection will be extended just past the outlet to stabilize the end of the pipe.

The 75th Avenue outfall will discharge into Skunk Creek at a location where there is no bank protection in place or planned. A standard concrete headwall will be installed but this is not a long-term solution for the end of pipe treatment. Permanent protection will be provided when a bridge is constructed at the 75th Avenue crossing and the end of pipe can be incorporated with bridge abutment, wingwalls, or bank protection constructed as a part of that project.