FINAL REPORT

Navajo Wash, Oracle Road to Ft. Lowell Road

Drainage Alternatives Analysis

Submitted to

City of Tucson Department of Transportation
Engineering Division

HDR Engineering, Inc.
5210 East Williams Circle, Suite 530
Tucson, Arizona
HDR No. 40502

REGISTERED PROFESSIONAL ENGINEER (CIVIL)
CERTIFICATE NO.
10890
Michael Thorne
Johnson
DATE SIGNED 6/18/07
ARIZONA, U.S.A.

HDR

June 2007

June 18, 2007

CITY OF TUCSON
Department of Transportation
Engineering Division
201 N Stone Ave, 4th Floor
Tucson, Arizona 85701

RE: Navajo Wash Drainage Alternatives Analysis
Final Report Submittal
HDR No. 40502

Attn: M. J. Dillard

M.J.:

Attached is our final report for the referenced project. It includes all of the work anticipated in the scope of work except for determination of a recommended alternative and initial plans for that selected alternative. It is understood that the fee remaining under this contract will be used instead for miscellaneous studies and design work for dealing with other unrelated drainage issues.

We did begin some initial plan-profile drawings for a 100-year storm drain which are unofficially included at the end of the appendices for future reference. Should other than a 100-year design be chosen at some later date, those plans would still be largely applicable but with narrower RCBC and/or smaller RCP sections. Those, together with the cost estimates included in the report, should suffice for future planning purposes should this project be brought back to life.

We have enjoyed working with you on this project. Please let me know if you have any further questions or concerns.

Sincerely,

HDR ENGINEERING, INC.

Michael T. Johnson

Michael T. Johnson, P.E., R.L.S.
Vice President

Attachment

FINAL REPORT

Navajo Wash, Oracle Road to Ft. Lowell Road

Drainage Alternatives Analysis

Submitted to

City of Tucson Department of Transportation

Engineering Division

HDR Engineering, Inc.

5210 East Williams Circle, Suite 530

Tucson, Arizona

HDR No. 40502

HDR

June 2007

TABLE OF CONTENTS

FIGURES

TABLES

APPENDICES

[Progress prints for 15% plans of work completed prior to termination is provided at the end of these appendices for future reference]

REFERENCES

1. "Mountain Avenue Storm Drain Alternatives Study", HDR/Johnson-Brittain; February 2003
2. "Downtown Land Use and Circulation Study Phase II Conceptual Drainage Development Phase 1 -- Roadway Drainage", Johnson-Brittain & Associates; June 1996.
3. "Standards Manual for Drainage Design and Floodplain Management in Tucson Arizona", Simons, Li & Associates, Inc.; December 1989
4. "Existing Conditions Hydrologic Modeling for the Tucson Storm water Management study, Phase II, Stormwater Master Plan", Simons, Li & Associates, Inc.; December 17, 1993.

SECTION 1. OVERVIEW

Navajo Wash is the largest tributary to Flowing Wells Wash. It begins in the vicinity of Mountain Avenue and Hedrick Drive where it collects significant flow accumulated in both Hedrick and Mountain. The City of Tucson's Tucson Stormwater Management Study modeling indicates the 100-year peak discharge at that point to be 2,381 cfs from a tributary area in excess of three square miles.

This flow crosses Ft. Lowell Road on grade and then proceeds westerly in Navajo Road 1.5 miles to Oracle Road. While in the Navajo roadway, it also crosses First Avenue and Stone Avenue on grade. It crosses under Oracle in a culvert entering a channel downstream in which it continues westerly. It joins Cemetery Wash and Flowing Wells Wash just downstream of Fairview Avenue and continues westerly eventually crossing under the UPRR and I-10 before discharging into the Santa Cruz River.

The channelized portion of the wash downstream of Oracle is located between Holy Hope Cemetery (to the north) and Evergreen Cemetery (to the south). The term "Navajo Wash" as used here refers to both the street flow and channelized portions of the wash. Figure 1.1 shows the location of the major reaches of these washes.

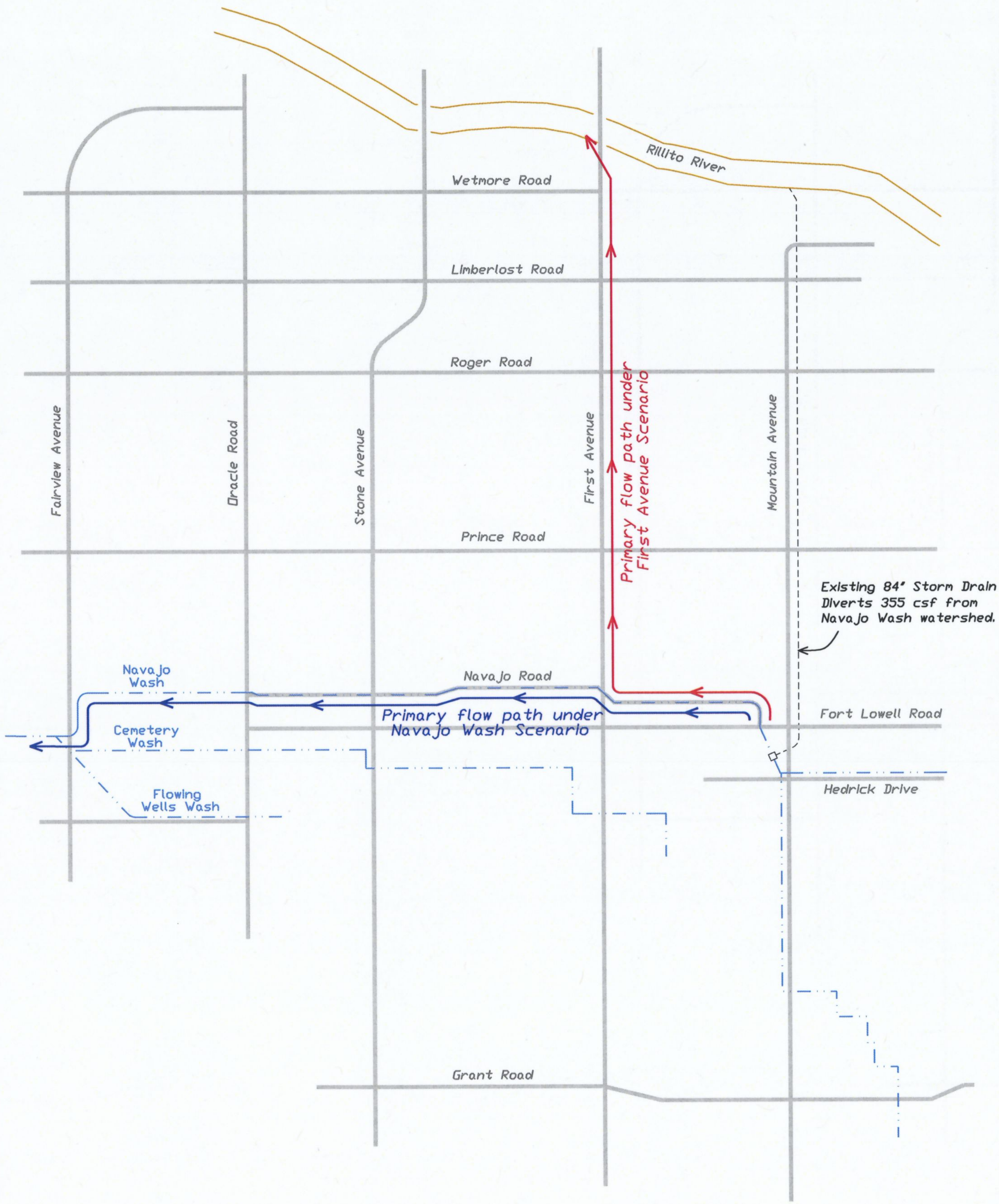


Figure 1.1
Project Vicinity

Project Needs

Navajo Wash presents three primary concerns -- (1) flood damage to homes and businesses along its course, particularly between Oracle Road and Fort Lowell Road, (2) interference with arterial traffic flow on Oracle Road, Stone Avenue, First Avenue, and Fort Lowell Road, and as a result cross-town mobility, and (3) impediment of access to adjacent property and connecting local streets.

Flooding. Flooding along Navajo Wash is well known. FEMA mapping shows a typical floodplain width of 1,000' from Fort Lowell to First Avenue and 1,500' from First to Oracle. Several hundred homes and businesses would be impacted in a 100-year storm. The study documented in Reference 1 which included a detailed flow profile analysis along Navajo Wash determined that of 106 structures fronting the roadway between Ft. Lowell Road and Oracle Road, 86 would be flooded in a 100-year event. It was also found that of 63 structures adjacent to the channel downstream of Oracle, five would be flooded. That study considered only structures immediately adjacent to the roadway and wash and not the many others further away that would also be impacted. Most of these structures are single family residences although a number of businesses would be damaged as well.

Arterial Traffic Flow. Navajo Wash impacts regional traffic movement. The at-grade crossings at Fort Lowell, First Avenue and Stone Avenue -- all significant arterials -- disrupt cross-town traffic in even relatively small storms.

Local Access. Over a hundred homes and businesses rely on Navajo Road for access which is also disrupted during even routine smaller storms. An 84" storm drain constructed northward from Hedrick and Mountain Avenue addresses this issue to some extent by intercepting a portion of this flow (up to 355 cfs according to Reference 1), but discharge quickly rebuilds along the roadway downstream of that point. The location of the existing 84" storm drain is shown in Figure 1.1.

In recognition of these problems, the City of Tucson and the Pima County Flood Control District have undertaken this study.

On grade crossing at First Avenue impacts regional traffic

Flow in Navajo Road impedes access in routine storms

OTHER PENDING PROJECTS

Recent passage of the Regional Transportation Plan and half-cent funding measure includes improvement of First Avenue from Grant Road to River Road. Dealing with Navajo Wash would clearly be a significant issue and cost for that project. The possibility of combining these funding sources to the advantage of both Navajo Wash and First Avenue improvements is part of the consideration here. Also anticipated for construction beginning in 2007 is *Mountain Avenue, Roger Road to Ft. Lowell Road.* That project includes construction of a new 72" storm drain at considerable expense and difficulty. The study of Reference 1 found that the 72" storm drain could be eliminated if the amount of Navajo Wash flow being diverted to the 84" storm drain were reduced.

The convergence of the Navajo Wash, First Avenue and Mountain Avenue projects presents several interesting questions and opportunities. The first is the relative cost and effectiveness of oversizing the First Avenue storm drain to carry Navajo Wash northward in First Avenue to the Rillito River. That would allow the improvements to Navajo Wash downstream of First to be eliminated or scaled back. The First Avenue construction will require some sort of storm drain in any event. The question is how much larger and more expensive would it need to be to accommodate Navajo Wash, and would that be more economical than the improvements that would otherwise be needed in Navajo Wash.

The second question is level of improvement. Typically, regional-level improvements are designed to remove residences and businesses from the 100-year flood plain, and flow in roadways contained within the right-of-way. That is the highest level of improvement considered here. Funding constraints may dictate a lesser level of improvement, however, and the option of designing to 10-year and 2-yr criteria is evaluated.

The third issue involves reducing or eliminating the amount of flow currently being diverted from Navajo Wash by the existing 84" storm drain. The study of Reference 1 found that eliminating the new storm drain in Mountain Avenue would save considerable cost and construction difficulty. The cost and other impacts of incorporating the necessary additional capacity into the Navajo Wash/First Avenue improvements is determined.

SCENARIOS

For convenience, the alternatives approaches of continuing to carry the flow along its present alignment or carrying instead along First Avenue are referred to here as "scenarios". These are as follows:

(1) The "Navajo Wash Scenario" which would involve the major storm drain being constructed along the current Navajo Wash alignment and a smaller storm drain in First Avenue.

(2) The "First Avenue Scenario" in which the size of the First Avenue storm drain would be increased to intercept Navajo Wash. The primary flow paths of the two scenarios are also shown in Figure 1.1.

Under the First Avenue Scenario, storm drain and other drainage improvements downstream of First Avenue in Navajo Wash would still be needed but would be smaller and less expensive to construct. Similarly, a storm drain will be associated with the construction of First Avenue under the Navajo Wash Scenario, but again would be smaller and less expensive.

To determine the most cost-effective approach, conceptual drainage plans for both scenarios have been developed, and costs and other impacts associated with each evaluated. This analysis is based on 100-year level of improvements, the assumption being that a similar result would be found for the lesser levels of improvement as well. The subsequent evaluation of level of improvement is based on the recommended scenario.

Relevant Past Work

The study of Reference 1 was performed in conjunction with the design of Mountain Avenue from Ft. Lowell Road to Roger Road. The Mountain Avenue project is to be constructed in a relatively narrow right-of-way already occupied by a number of major utilities including the aforementioned 84" storm drain. The project design requires that a second parallel 72" storm drain be added to this mix at a cost of $2.3 million in 2002 dollars (that estimate has since more than doubled). It also increases construction difficulty, impact to utilities, and the extent and duration of disruption of the area during construction. The proposal considered was to meter the amount of Navajo Wash flow being intercepted by the 84" storm drain such that sufficient capacity would be left in the existing storm drain for the project drainage. That would eliminate the need for the second storm drain, thereby reducing cost and other impacts its construction would create.

It was found that reducing the maximum intercepted flow from 355 cfs (the capacity of the 84") to 225 cfs would provide that capacity. It was also found that the additional 130 cfs in Navajo Wash would increase the depth of flooding from 0.1' to 0.2' in the roadway between Fort Lowell and Oracle, and 0.2' to 0.4' in the channel downstream of Oracle. The prospect of increasing an already serious flooding condition, by even a relatively small amount, was deemed unacceptable. The cost of providing sufficient underground conveyance to offset the 130 cfs increase was found to be $4.1 million -- $1.5 million higher than the cost of the second storm drain in Mountain -- and the metering proposal was dropped.

Metering Proposal Revisited

It will be seen later that the 100-year Navajo Wash discharge ranges from 1,701 cfs to 1,843 cfs in the roadway between Oracle and Fort Lowell. Increasing the capacity of the new storm drain to contain the additional 130 cfs may result in little additional cost in comparison to the cost and other difficulties associated with a new Mountain Avenue storm drain. It is for that reason that the metering proposal is reconsidered here.

Project Limits

From the earlier study, it is known that the channel downstream of Oracle will need to be deepened to accommodate a storm drain of any significance in Navajo Road. This is true regardless of the scenario adopted. It is anticipated that this will require reconstructing the channel through the cemeteries to the inlet of the box culvert at Fairview Avenue. Project mapping has been extended several hundred feet beyond that point to establish starting conditions for hydraulic analysis. The project hydrology is carried through the junction of Navajo Wash with Flowing Wells Wash.

The upstream limit is the north side of Ft. Lowell Road where the flow would be collected above the current at-grade crossing, possibly at the existing catch basin near Hedrick and Mountain. The TSMS node DG-N0200, which is actually located just north of Ft. Lowell Road is taken here to reflect the discharge reaching Fort Lowell including the inlet of the existing 84" storm drain.

Structure of Report

The remainder of this report discusses these issues in detail. The following report sections provided and their respective purposes are as follows:

Section 2 -- Base Hydrology outlines the development of base hydrologic modeling for the project starting with regional TSMS modeling provided by the City of Tucson and incorporating refinements needed for this project.

Section 3 -- Conceptual Future Roadway and Channel Improvements describes the development of future roadway and channelization plans along both Navajo Wash and First Avenue. These plans are first used for developing surface flow routing data for hydrologic modeling, and later for developing conceptual storm drain plans and cost estimates.

Section 4 -- Initial Storm Drain Sizing for Alternative Scenarios describes the development of hydrologic modeling for the Navajo Wash and First Avenue scenarios and preliminary sizing of storm drain and other improvements needed to achieve the 100-year level design. Since the hydrology is a function of storm drain size, this is potentially an iterative process. This is dealt with by first determining "preliminary" design discharges for each scenario assuming all flow is routed in the future roadway cross-sections. From that, preliminary storm drain sizing is determined and incorporated into the hydrologic modeling to provide "final" design discharges and storm drain sizing adjusted accordingly.

Section 5 -- Conceptual Plans for Alternative Scenarios documents an evaluation of costs, utility impacts, and other factors germane to the selection of a scenario. Conceptual plans for the preliminary sizing determined previously are provided. The need for water and sewer relocation, including how to handle penetrations through the storm drain that will be necessary, are addressed. The Navajo Wash Scenario is found to be substantially less expensive and chosen for subsequent investigations regarding level of improvement and diversion of flow at Mountain Avenue.

Section 6 -- Alternative Levels of Improvement repeats the hydrology, conceptual design and cost estimating for the Navajo Wash Scenario for ten and two-year storms This is to determine the approximate cost savings that would be realized under a lesser design criterion in the event that sufficient funding for the 100-year level of improvement is not available.

Section 7 -- Impact of Metering Inflow to Existing Mountain Avenue Storm Drain determines the increase in size and cost in storm drain improvements that would be needed to accommodate some or all of the flow currently being diverted by the 84" storm drain. The 2002 cost estimate for the 72" storm drain for Mountain Avenue is updated to determine the overall cost-effectiveness of its elimination.

Section 8 -- Hydraulic Effects of Alternative Levels of Improvement discusses the evaluation of existing hydrologic and hydraulic conditions along Navajo Wash. HEC-2 is used for this purpose but, as with the TSMS modeling, is updated based on more detailed information that is now available, and to meet specific needs of this project. The project hydrologic base modeling is used to establish discharges, and cross-sections are refined based on the topographic mapping and manual survey data. This information is used later to evaluate the relative benefit of designing to a lesser level of improvement.

[Section 9 -- Selected Alternative was under the original contract intended to summarize the results of the study and identify the chosen course of action. Conceptual plans would have been provided reflecting the selected approach, and hydrology and hydraulic modeling updated accordingly. Due to lack of construction funding, this last step was omitted prior to its completion.]

SECTION 2.
BASE HYDROLOGY

This section describes the development of base hydrologic modeling used in evaluating various drainage improvement proposals. The City of Tucson's TSMS modeling is intended for use in planning and design of drainage improvements of regional significance. TSMS modeling is relatively broad in scope however, and typically needs to be refined to add detail and account for other conditions not reflected in the city-wide modeling. For a study of downtown drainage made in conjunction with DLUCS II , for example, the four TSMS subbasins representing the downtown area were subdivided ("densified") into 92 separate subbasins and over 350 total modeling elements to reflect the complex network of street flow, storm drain flow, flow splits at intersections, and so forth (Reference 2). The modifications made here are not nearly that extensive but are nonetheless an important aspect of creating hydrologic modeling that provides the level of detail needed for this project.

TSMS METHODOLOGY

The TSMS modeling applicable to this project includes portions of Flowing Wells Wash and First Avenue Watersheds (DG and DR). A map showing these watersheds and their modeling elements is provided as Figure 2.1. The locations of key points of concentration used for this project (POC_A through POC_K) are also shown.

An important supposition of the City's adopted hydrologic methodology (as described in Reference 3) and its TSMS methodology (Reference 4) is the concept of areal reduction in basins with tributary areas exceeding one square mile. This is done by applying an "areal reduction factor" (ARF) to basin precipitation (PB). ARF is computed as

$$ARF = A^{-(.027 + .07 \, Log \, A)}$$

where A is the tributary area in square miles. This effect makes it necessary to conduct separate HEC-1 runs using adjusted PB values to determine design discharges for each point of concern once the cumulative tributary area exceeds one square mile.

Further complicating this situation is that TSMS uses SCS Curve Numbers (CN) to establish the runoff-to-rainfall ratio. City methodology calls for adjusting CNs to account for the phenomenon observed in arid regions -- that this ratio varies with precipitation intensity. This inconveniently requires that all LS cards in the input data file also be changed if precipitation is adjusted. To deal with this in a practical manner, the City uses special software termed a "modeler" to create individual HEC-1 input data files for each point where a discharge is needed.

BASELINE INFORMATION

The City used the modeler to produce data files for each project point of concentration. These were provided for this study along with corresponding output files. The results are discharges at key points for which the precipitation has been areally reduced and CNs adjusted, providing a baseline of discharge results that is pure in terms of TSMS methodology. That has been used to evaluate the effects of certain simplifying assumptions and approaches that are necessary here.

TRIBUTARY AREAS FOR DETERMINING ARFS

The cumulative areas provided for each node in the TSMS modeling are net of non-contributing area and cannot be used directly for determining ARFs. The actual areas are maintained by the City in "watershed reports" (CR2 files).

There are four separate drainage basin configurations used in the course of this study for evaluating existing and proposed conditions. Each results in different tributary areas for at least some project points and therefore require separate calculations of ARFs and PBs. These calculations are provided together in Appendix 2a for convenience, identified with the HEC-1 models TSMS(1) through TSMS(12) for which they will be used. These models and their purpose are discussed later at applicable locations throughout this report.

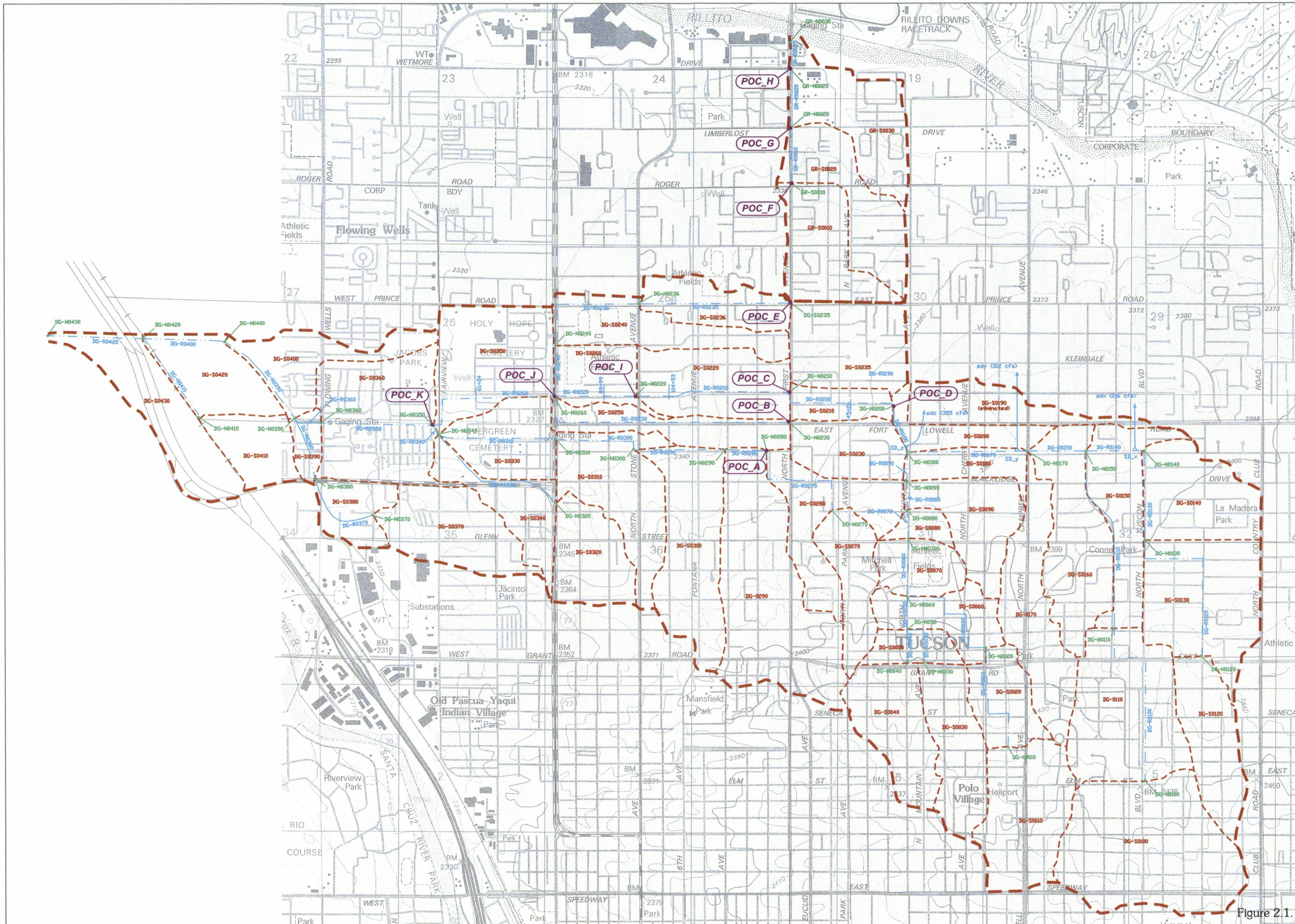


Figure 2.1. Drainage Map

TSMS(1) -- Initial TSMS Modeling

The first of these models, with the exception noted here, reflects current TSMS modeling. The modeler-generated input files for DG-N0350 (POC_K in Figure 2.1) and GR-N0030 (the outlet downstream of POC_H) were combined to create what is termed here as the "Initial TSMS Modeling". This file served as the starting point for developing the various hydrologic models needed for this project. It has been run separately for each point using the applicable PB value. The results are summarized in Table 2.1 along with tributary area, ARF and PB for each point.

The CN values for the DG portion of this file, however, have not been changed for each run due to the amount of effort that would entail. Instead, constant CN values are based on PB = 3.131 corresponding to POC_K have been used and, as a result, only the discharge for that point is in the strict sense correct. This is not an issue for the GR portion of the watershed since it is less than one square mile in area and areal reduction does not apply.

To assess the effect using constant CNs, the baseline results provided by the City are also shown in Table 2.1. It can be seen that the greatest deviation is about 2% for the relatively small watershed of POC_E. Error along the main flow path, which is of greatest concern, is less than 1% in all cases. These results justify the use of constant CNs for this project, a simplification that is essential here.

The files and results associated with the Initial TSMS Modeling are referred to as TSMS(1) to help distinguish it from other analyses later. The printout of the HEC-1 run associated with POC_K is provided in Appendix 2b.

Table 2.1. Comparison of TSMS(1) Initial and Baseline TSMS Results

	Initial TSMS Modeling TSMS(1) PB Varies but CN's are Constant					Initial TSMS Output Files Provided by City of Tucson			
POC	TSMS Node	Trib Area	ARF	PB	Q100	File	PB	Q100	Error
POC_A	DG-N0280	.274	1.000	3.600	528	DG0280.OUT	3.600	537	-1.7%
POC_B	DG-S0230	.107	1.000	3.600	188	--	--	--	--
POC_D	DG-N0200	3.095	.933	3.359	2,363	DG0200.OUT	3.359	2,381	-0.8%
POC_C	DG-N0210	3.150	.931	3.353	2,106	DG0210.OUT	3.353	2,122	-0.8%
POC_I	DG-N0220	3.331	.926	3.335	1,940	DG0220.OUT	3.335	1,952	-0.6%
POC_J	DG-N0260	4.068	.907	3.265	2,109	DG0260.OUT	3.265	2,113	-0.2%
POC_K	DG-N0350	5.718	.870	3.131	2,811	DG0350.OUT	3.131	2,811	--
POC_E	DG-N0235	.192	1.000	3.600	236	DG0235.OUT	3.600	241	-2.1%
POC_F	GR-S0010	.125	1.000	3.600	227	GR0010.OUT	3.600	227	--
POC_G	GR_N0020	.303	1.000	3.600	343	GR0020.OUT	3.600	343	--
POC_H	GR_N0030	.508	1.000	3.600	632	GR0030.OUT	3.600	632	--

TSMS(2) -- REFINED TSMS MODELING

Refinement of the regional TSMS modeling made for this project includes the following:

1. Diversions have been added to model the effect of larger storm drains that carry flow out of the area:
   - o Just below DG-N0140, 216 cfs capacity of Tucson Blvd storm drain
   - o Just below DG-N0170, 312 cfs capacity of Campbell Ave storm drain
   - o Just below DG-N0180, 355 cfs capacity of Mountain Avenue storm drain

The location of these diversions are indicated in Figure 2.1.

2. The following routing elements have been revised based on surveying performed for the Mountain Avenue study:
   - o DG-R0200, based on section 145+26
   - o DG-R0210, based on section 104+53
   - o DG-R0220, based on section 89+99
   - o DG-R0260, based on section 61+24

This was done to better represent the actual characteristics of those routing reaches. The original data resulted in significant amounts of overbank storage in the developed areas along Navajo Road, thereby relying on a substantial level of flooding in determining peak discharges downstream. That may be appropriate for evaluating existing flooding but not for programming improvements.

The locations of the revised sections are shown in Figure 2.1. Plots of the surveyed cross-sections and the eight-point cross-sections taken from them needed for the RX/RY card are provided in Appendix 2c. A second set of plots shows the original TSMS sections for comparison.

3. Subbasin DG-N0190 will be cut off by Mountain Avenue construction and has been eliminated.

This modeling is referred to here as TSMS(2). As with TSMS(1), this model was run for each POC using the applicable PB value. The ARF and PB values differ from TSMS(1) because S0190 has been excluded.

The TSMS(2) results are tabulated in Table 2.2 along with those of TSMS(1) for comparison. The discharges at POC_D (Ft. Lowell Road) best show the effect of the existing storm drains, reducing the discharge there from 2,363 to 1,701 cfs, about 28%. Moving downstream, this benefit is eventually offset by the reduction in storage attenuation with a net increase in peak discharge from Oracle on. The discharge at POC_K increases 822 cfs or 29%. The HEC-1 printout reflecting these adjustments for POC_K is provided as Appendix 2d.

TABLE 2.2. Comparison of TSMS(2) Refined and TSMS(1) Initial Results

		Refined TSMS(2) Modeling				Initial TSMS(1) Modeling				
POC	Reach	Trib Area	ARF	PB	Q100	Trib Area	ARF	PB	Q100	Δ Q100
A	--	.274	1.000	3.600	528	.274	1.000	3.600	528	--
B	BC	.107	1.000	3.600	188	.107	1.000	3.600	188	--
D	DC	3.017	.935	3.367	1,701	3.095	.933	3.359	2,363	-662
C	CI	3.071	.934	3.361	1,722	3.150	.931	3.353	2,106	-384
I	IJ	3.253	.929	3.343	1,843	3.331	.926	3.335	1,940	-97
J	JK	3.989	.909	3.272	2,382	4.068	.907	3.265	2,109	273
K	--	5.693	.871	3.137	3,609	5.718	.870	3.131	2,811	798
E	EF	.192	1.000	3.600	236	.192	1.000	3.600	236	--
F	FG	.317	1.000	3.600	227	.125	1.000	3.600	227	--
G	GH	.495	1.000	3.600	343	.303	1.000	3.600	343	--
H	--	.700	1.000	3.600	632	.508	1.000	3.600	632	--

TSMS(3) -- PROJECT BASE MODELING

The base modeling for this project is equivalent to the TSMS(2) modeling except that the hydrographs of flow reaching each project point are temporarily taken from the system using diversion elements, but are stored and remain available for later use. They are later recalled and routed in a manner that reflects the scenario being considered. This approach simplifies the evaluation of various scenarios later.

TSMS(3) reflects this approach with the hydrographs recalled and routed using the TSMS(2) routing elements. This -- if done correctly -- would be exactly equivalent to TSMS(2). The results summarized in table 2.3 show that this is in fact the case. The printout corresponding to POC_K is provided as Appendix 2e.

Developing the diverted hydrographs for project points constitutes the bulk of the modeling needed for evaluating any of the scenarios and conditions considered later. The generating of the stored hydrographs will not change. This approach results in coding specific to a particular scenario being accomplished in the last 100 or so lines of coding in what would otherwise be on the order of 800 lines. This removes the repetitive elements that form the bulk of the project modeling, making the pertinent modeling for each particular situation easier to follow as well as reducing the volume of output. From this point, data and results from subsequent HEC-1 runs common to TSMS(3) will be deleted from the printouts.

TABLE 2.3. TSMS(3) Project Base Model Check

POC	Reach	Trib Area	ARF	PB	TSMS(3) Q100	TSMS(2) Q100
A	--	.274	1.000	3.600	528	528
B	BC	.107	1.000	3.600	188	188
D	DC	3.017	.935	3.367	1,701	1,701
C	CI	3.071	.934	3.361	1,722	1,722
I	IJ	3.253	.929	3.343	1,843	1,843
J	JK	3.989	.909	3.272	2,382	2,382
K	--	5.693	.871	3.137	3,609	3,609
E	EF	.192	1.000	3.600	236	236
F	FG	.317	1.000	3.600	227	227
G	GH	.495	1.000	3.600	343	343
H	--	.700	1.000	3.600	632	632

SECTION 3. CONCEPTUAL FUTURE ROADWAY AND CHANNEL IMPROVEMENTS

This section describes the development of preliminary plans for future Navajo Road and First Avenue improvements. These plans are used to determine the rating data needed for hydrologic modeling as well as provide a basis for developing and evaluating alternative drainage improvement proposals.

SOURCE OF TOPOGRAPHIC INFORMATION

Design-level topographic mapping was obtained along Navajo Wash in anticipation of final design being initiated soon after this study is complete. This mapping was created photogrammetrically and consisted of topographic mapping, a digital terrain model (DTM), and digital orthophotos. The DTM provides an existing ground surface used by *InRoads*, specialized software package used for roadway and in this case channel design.

The photogrammetric mapping has been supplemented with manually-surveyed cross-sections of the Navajo roadway and channel that were taken for the Reference 1 study. This was accomplished by merging that surface into the photogrammetrically created surface such that it overrides photogrammetry since it is more accurate.

The DTM has also been used for obtaining cross-sections for hydraulic design. In some cases, it has been necessary to extend HEC-2 cross-sections beyond the photogrammetry. That has been done using 2005 mapping obtained from PAG. Corresponding PAG orthographic photos have been used to identify structures lying outside the project mapping for evaluating flooding impact in Section 8.

Mapping was not obtained along First Avenue. Base sheets and InRoads modeling have been developed from as-built plans with appropriate datum and stationing adjustments. Five plans sets used for this were:

- D-88-01: 16'x 8' storm drain from Wetmore Road to Rillito River (1988)
- I-828: Wetmore Road to Roger Road (1960)
- I-813: Roger Road to Navajo Road (1956)
- I-674: Blacklidge to Ft. Lowell Road (1957)
- I-606: Glenn Street to Elm Street (1956)

Datum adjustments were based on field measurements. Stationing was mathematically adjusted to a single consistent project stationing.

INROADS

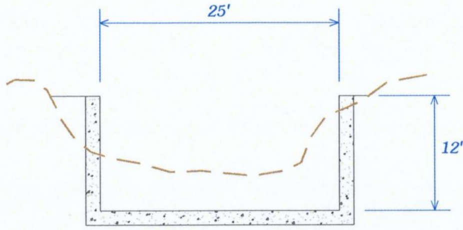
The merged surfaces were used with InRoads to develop the roadway and channel designs, and to create the plan-profile sheets. InRoads has also been used to plot cross-sections and determine quantities used for estimating costs.

EXISTING WATER AND SEWER

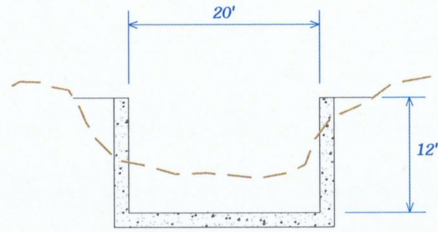
Major water and sewers have been added to the plans based on records obtained from Tucson Water and Pima County Wastewater Management. That information will be used later to assess utility impact and relative cost of utility relocation of the two scenarios.

TYPICAL SECTIONS

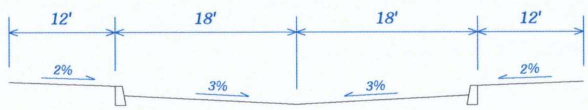
Typical sections for the future roadways and channels are shown in Figure 3.1. Navajo Road is anticipated to be a 36' residential street, inverted 3% (per City of Tucson development standards) to enhance flow capacity. First Avenue is a six-lane divided arterial. The nominal width of right-of-way has been assumed 120' for hydraulic rating purposes though is likely to be larger in some locations such as major intersections.



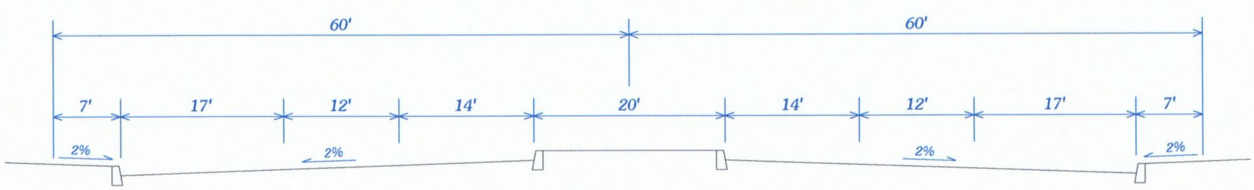
Navajo Wash Channel -- Navajo Wash Scenario
Fairview Avenue to Oracle Road
Routing Element CF_JK



Navajo Wash Channel -- First Avenue Scenario
Fairview Avenue to Oracle Road
Routing Element CF_JK



Navajo Road, Oracle Road to Ft. Lowell Road



First Avenue, Ft. Lowell Road to Wetmore Road

Figure 3.1. Future Roadway and Channel Typical Sections

Channel Design

As suggested earlier, it will be necessary to lower the channel downstream of Oracle to achieve sufficient depth to outlet a storm drain in Navajo Road. That is true under either scenario even though the storm drain under the First Avenue approach would be smaller. It is necessary for the channel reconstruction to extend to the existing culvert at Fairview in either case. A very flat longitudinal slope (0.25%) is needed to achieve the necessary depth. A concrete bottom is proposed to maintain velocity of flow and minimize sediment and trash buildup.

Vertical walls have been assumed due to the limited space available between improvements belonging to the two cemeteries. The south fence and roadway in Holy Hope Cemetery appears to extend approximately 10' into the 60' existing right-of-way, but it is unlikely that those improvements can reasonably be removed. Vertical walls are necessary to avoid interfering with those improvements (and probably would be in any event). The height of the walls is dictated by the need to retain adjacent ground rather than hydraulic capacity.

The bottom width on the other hand *is* based on hydraulic capacity. It has been sized such that 100-year normal flow depth for the particular scenario is at or just below the crown elevation of the upstream storm drain to avoid undue tailwater conditions. During final design of the storm drain, it may be found that some degree of tailwater is acceptable and the channel size reduced accordingly.

Roadway Design

The conceptual roadway designs for Navajo Wash and First Avenue are provided in Appendices 3a and 3b respectively including plan-profile sheets and plotted cross-sections.

Surface Routing Elements

The centerline profiles of the Navajo Wash and First Avenue concept plans have been used to develop stage-discharge-storage rating data needed for various modeling purposes. The typical sections of Figure 3.1 are assumed. Tables 3.1, 3.2, and 3.3 present the geometric information used to formulate the rating data for the channel downstream of Oracle, the inverted 36' street of Navajo Road, and the six-lane divided arterial for First Avenue.

The calculation of stage/discharge/storage data is found in Appendix 3c. Note that "W" in Tables 3.2 and 3.3 refers to half the street width. Stage has been expressed in logical height increments for the given cross-section. The lowest elevation is set to zero making stage equivalent to depth of flow which is readily found in the HEC-1 summary printout. Discharge has been determined assuming Manning's formulation for shallow triangular flow separately for the increments shown. Storage volume is simply the area of flow for the given depth times the length of reach. These calculations have been performed and the SE/SQ/SV data cards created with spreadsheets.

TABLE 3.1. Geometric Data -- Future Navajo Wash Channel

Scenario	W	H	n	L	So
Navajo Wash Scenario	25.0	8.0	.016	3,176	.0025
First Avenue Scenario	20.0	6.0	.016	3,176	.0025

TABLE 3.2. Geometric Data -- Future Navajo Road Improvements

				Roadway				Parkway				
POC	Reach	Location	Sta	W	Sx	n	h	W	Sx	n	L	So
D		Ft. Lowell	160+12									
	SF_DC			18.0	.030	.016	.50	12.0	.020	.025	2,780	.0050
C		First Ave	132+30									
	SF_CI			18.0	.030	.016	.50	12.0	.020	.025	3,450	.0045
I		Stone Ave	97+72									
	SF_IJ			18.0	.030	.016	.50	12.0	.020	.025	1,870	.0037
J		Oracle	79+10									

TABLE 3.3. Geometric Data -- Future First Avenue Improvements

POC	Reach	Location	Sta	Roadway W	Roadway Sx	Roadway n	Roadway h	Parkway W	Parkway Sx	Parkway n	L	So
B		Ft. Lowell	128+71									
	SF_BC			43.0	.020	.016	.50	7.0	.020	.025	440	.0030
C		Navajo	124+26									
	SF_CE			43.0	.020	.016	.50	7.0	.020	.025	2,220	.0030
E		Prince Road	102+06									
	SF_EF			43.0	.020	.016	.50	7.0	.020	.025	2,650	.0030
F		Roger Road	75+59									
	SF_FG			43.0	.020	.016	.50	7.0	.020	.025	1,320	.0078
G		Limberlost	62+39									
	SF_GH			43.0	.020	.016	.50	7.0	.020	.025	1,320	.0030
H		Wetmore	49+19									

Regarding Table 3.2 -- Note that parkway overflow is erroneously calculated at 0.54' too deep. Though this will not materially change any of the results presented here, it should be corrected for final design, particularly if a less than 100-year storm drain approach is chosen.

Section 4
Initial Storm Drain Sizing for Alternative Scenarios

This section describes the process by which preliminary storm drain sizing has been determined for the two scenarios. These sizes are based on full flow capacity and used only for selecting between the Navajo Wash and First Avenue Scenarios. More detailed hydraulic grade and flow-profile calculations would be used later in the final design of the selected alternative. The preliminary sizing is used here to determine approximate relative costs of the two scenarios. That information will be considered in Section 5 in selecting between the Navajo Wash and First Avenue Scenarios.

TSMS(4) -- Initial Hydrologic Modeling -- Navajo Wash Scenario

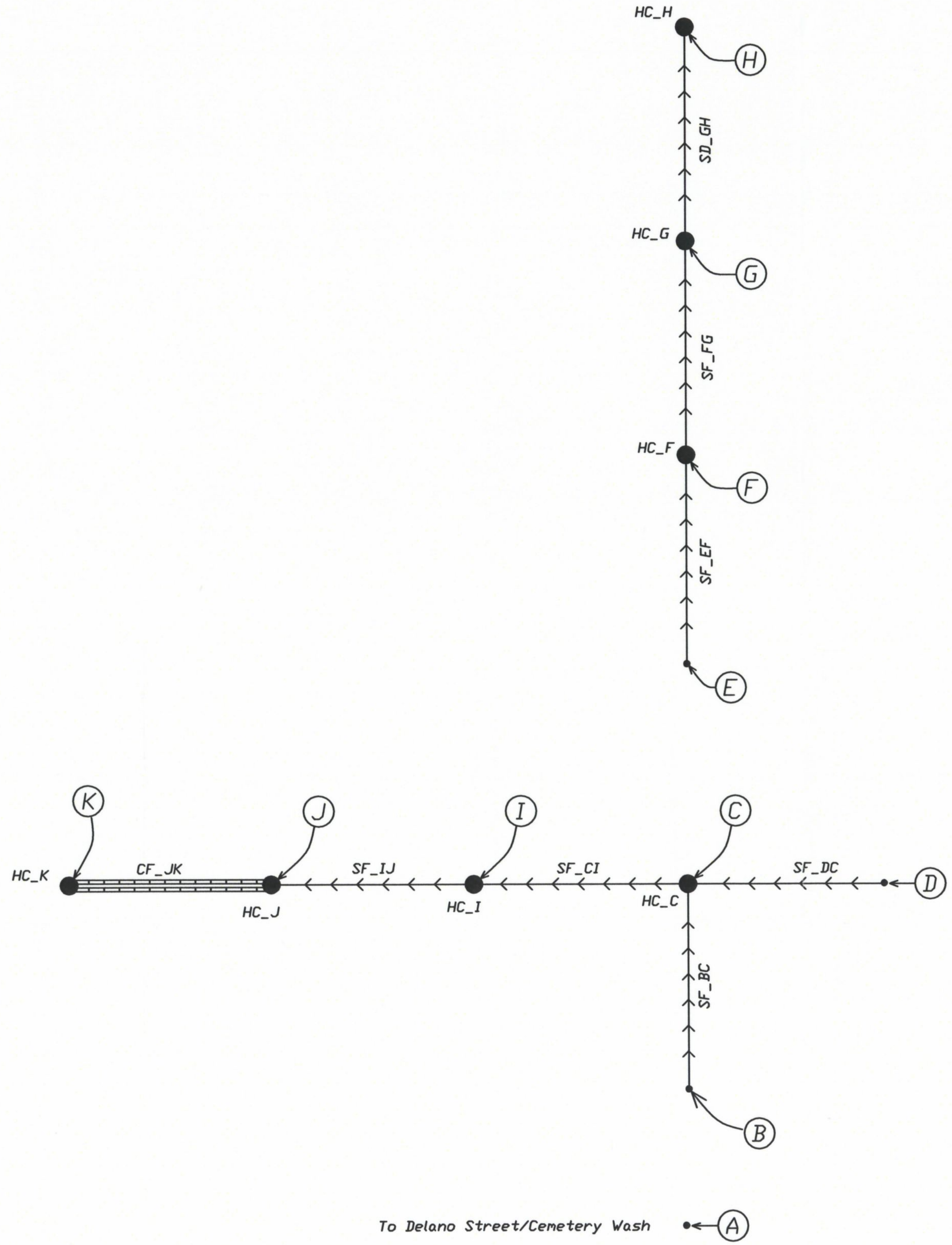
An initial hydrologic model was created for the Navajo Wash Scenario, TSMS(4), from the project base TSMS(3) modeling. Storm drain routing elements were not included at this point since storm drain sizing was not yet known. All flow was routed as street flow in the future roadways. The results provide an initial estimate of discharges from which preliminary sizing of the storm drain system can be estimated. The preliminary storm drain sizes are later incorporated into the hydrologic modeling to better reflect actual flow conditions. That modeling is referred to as "detailed" modeling and discussed below.

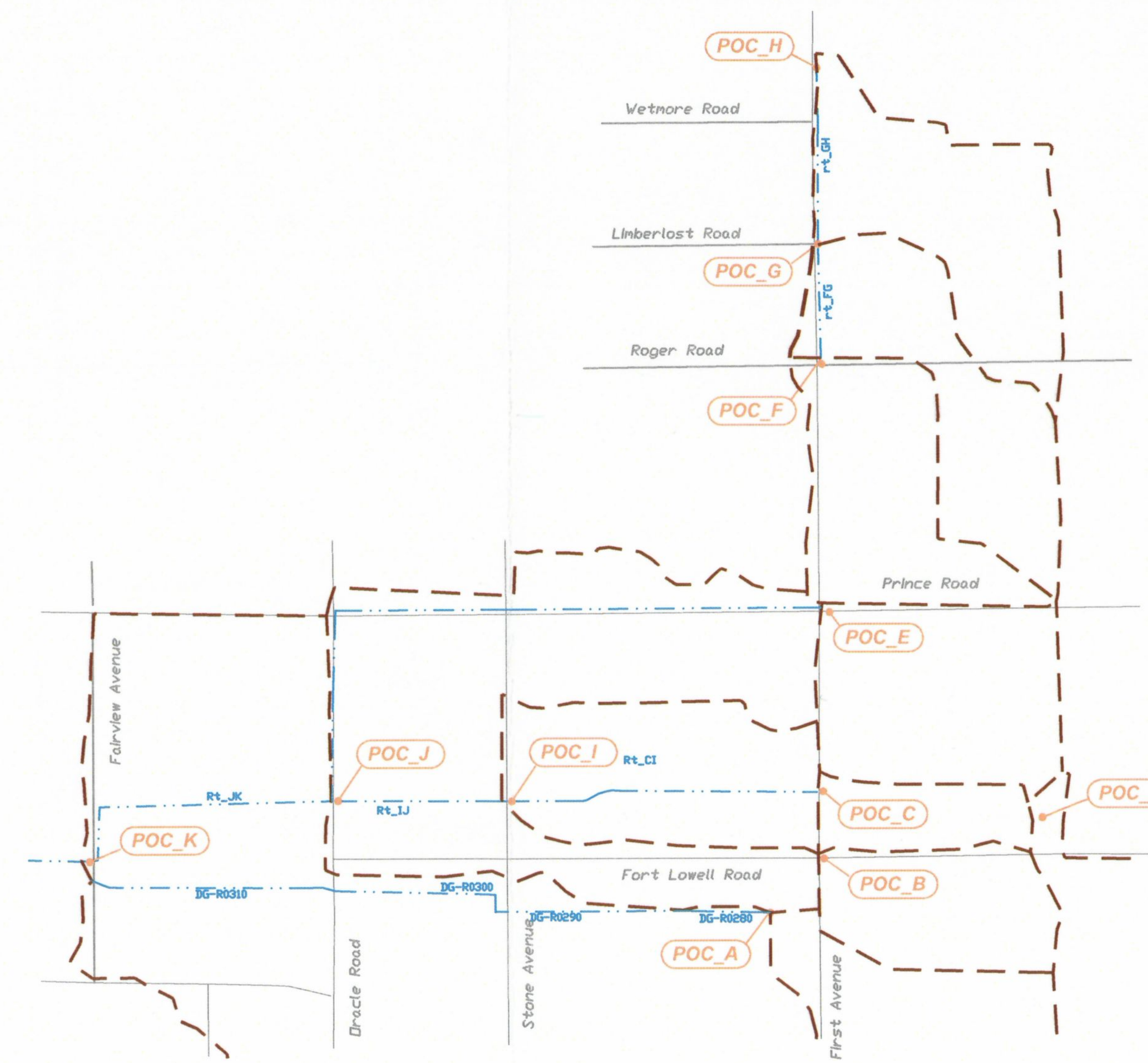
The results of the initial analysis for the Navajo Wash Scenario are tabulated in Table 4.1. The equivalent values for TSMS(2) are shown to provide a general comparison to existing conditions. It can be seen that the values are similar as one would expect since the routing follows the same path. The TSMS(4) values increase somewhat moving downstream because the street routing sections constrain the flow to the future roadway. That both reduces the storage volume and the resulting level of attenuation, and increases the velocity of flow -- in effect decreasing the time of concentration. There is also some difference due to the POC_E flow being carried north in First Avenue as opposed to west in Prince Road as TSMS has it.

Again, separate HEC-1 runs have been made for each PB value. A representative printout for PB = 3.151 (corresponding to POC_K with POC_E redirected to First Avenue) is provided as Appendix 4a. Input data and results in common with TSMS(3) have been deleted as discussed earlier. A diagram of the TSMS(4) modeling is shown in Figure 4.1 along with the applicable portion of the drainage map for reference.

TABLE 4.1. TSMS(4) Preliminary Discharges -- Navajo Wash Scenario

		TSMS(4)				TSMS(2)				
POC	For Reach	Trib Area	ARF	PB	Q100	Trib Area	ARF	PB	Q100	Incrs in Q100
A	--	.274	1.000	3.600	528	.274	1.000	3.60	528	--
B	BC	.107	1.000	3.600	188	.107	1.000	3.60	188	--
D	DC	3.017	.935	3.367	1,701	3.017	.935	3.367	1,701	--
C	CI	3.178	.931	3.350	1,828	3.071	.934	3.361	1,722	106
I	IJ	3.359	.926	3.332	1,995	3.253	.929	3.343	1,843	152
J	JK	3.904	.911	3.280	2,583	3.797	.914	3.290	2,382	201
K	--	5.554	.873	3.143	3,931	5.447	.875	3.151	3,609	322
E	EF	.192	1.000	3.600	236	.192	1.000	3.60	236	--
F	FG	.317	1.000	3.600	347	.317	1.000	3.60	227	120
G	GH	.495	1.000	3.600	509	.495	1.000	3.60	343	166
H	--	.700	1.000	3.600	789	.700	1.000	3.60	632	157





TSMS(4).dwg

Figure 4.1
Preliminary Modeling Configuration -- Navajo Wash Scenario

TSMS(5) -- Initial Hydrologic Modeling -- First Avenue Scenario

A separate hydrologic model was also created for the First Avenue Scenario TSMS(5) from the project base TSMS(3) modeling. As with TSMS(4), storm drain routing elements were not yet included, and all flow routed along the flow path as street flow.

The results of this analysis are shown in Table 4.2. As before, separate HEC-1 runs have been made for each PB value. A representative printout for PB = 3.282 corresponding to POC_H is provided as Appendix 4b. Input data and results in common with TSMS(3) have again been deleted. A diagram of the TSMS(5) modeling is shown in Figure 4.2.

Table 4.2. TSMS(5) Preliminary Discharges -- First Avenue Scenario

POC	For Reach	Trib Area	ARF	PB	TSMS(5) Q100
A	--	.274	1.000	3.600	528
B	BD	.107	1.000	3.600	188
D	DC	3.017	.935	3.367	1,701
C	CE	3.178	.931	3.350	1,828
E	EF	3.369	.925	3.331	1,956
F	FG	3.495	.922	3.319	2,010
G	GH	3.672	.917	3.301	2,203
H	--	3.877	.912	3.282	2,454
I	IJ	.274	1.000	3.600	423
J	JK	.274	1.000	3.600	1,101
K	--	1.996	.967	3.483	2.846

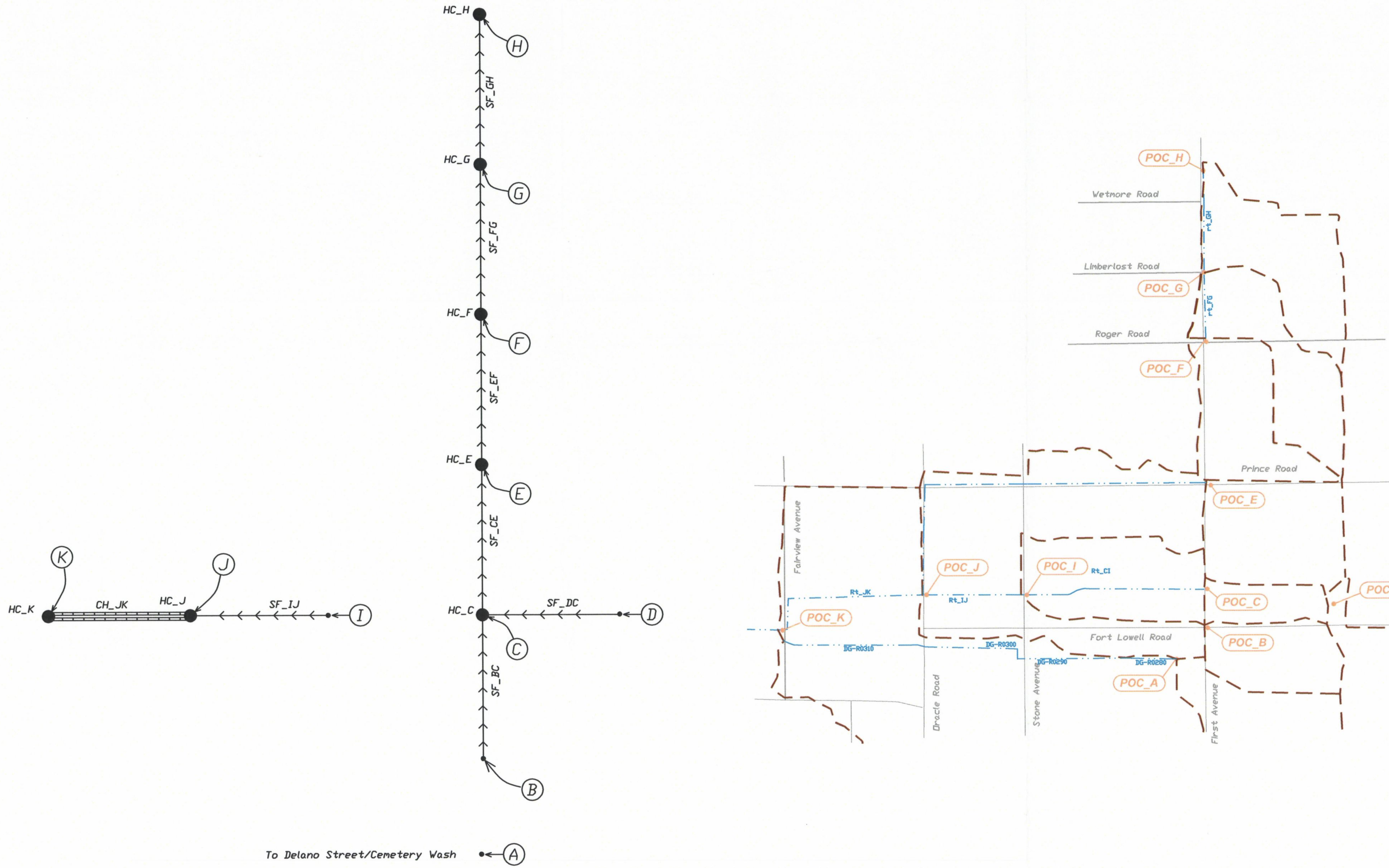


Figure 4.2
TSMS(5) Preliminary Modeling Configuration -- First Avenue Scenario

TSMS(5).dwg

INITIAL STORM DRAIN SIZING

The TSMS(4) and TSMS(5) analyses provide an initial set of design discharges from which preliminary storm drain sizing has been determined. To do so, the capacity of street flow (at a depth that just reaches the right-of-way in the proposed roadway section) has been subtracted to provide the approximate storm drain capacity needed.

Initial storm drain sizes were then determined assuming full flow. Manning's "n" was increased from the normal .012 to .016 to approximate the effects of aging and minor losses. Generally, the largest pipe size and deepest culvert section allowed by the future roadway profile has been used assuming that to be the most hydraulically efficient. In cases of relatively small flow, one or more RCPs has been assumed, sized in 0.5' increments to the smallest diameter providing sufficient capacity. For larger flows, ADOT standard RCBC sizes have been assumed, again using the deepest structure possible, and experimenting with the number and widths of cells to best suit each particular set of conditions. ADOT cells vary in depth in one foot increments and in width in two-foot increments.

These increments have been used here for the purpose of comparing scenarios. For final design, pre-cast cells whose widths vary in one-foot increments may instead be used.

It has also been assumed that the number of cells does not increase in the upstream direction. Because of that, the storm drain configuration in Navajo Road under the two scenarios is different even though the design flows are the same.

The preliminary storm drain sizing was performed using a spreadsheet application which shows the future roadway profile as well as that of the storm drain being tested. That provides for an interactive process by which various storm drain sizes, slopes, and configurations can be easily examined in arriving at a suitable result. These calculations are provided in Appendix 4c and 4d for the Navajo Wash and First Avenue scenarios respectively. The results are summarized in Tables 4.3 and 4.4.

Table 4.3. TSMS(4) Preliminary Storm Drain Sizing -- Navajo Wash Scenario

Reach	Location	Q100	Strt/ Chnl Cpcty	Cpcty Needed	Cnvync Type	nCells	B	D	So	n	L	Qcap	Excess Cpcty
	-- Navajo Wash --												
JK	Fairview to Oracle	2,583	--	2,583	Channel	--	25.0	8.0	.0025	.016	--	2,679	+96
IJ	Oracle to Stone	1,995	251	1,744	RCBC	3	10.0	7.0	.0040	.016	1,874	2,002	+258
CI	Stone to First Ave	1,828	277	1,551	RCBC	3	8.0	7.0	.0050	.016	3,449	1,677	+126
DC	First Ave to Ft. Lowell	1,701	292	1,409	RCBC	3	8.0	7.0	.0050	.016	2,786	1,677	+268
	-- First Avenue --												
	Exst 16'x8' Outlet to Inlet	789	1,255	-466	Existing	1	--	--	.0030	.016	869	--	+466
GH	Exst Inlet to Limberlost	509	59	450	RCBC	1	10.0	8.0	.0020	.016	1,239	567	+117
FG	Limberlost to Roger	347	95	252	RCP	1	--	6.0	.0080	.016	1,321	309	+57
EF	Roger to Prince	236	59	177	RCP	1	--	6.0	.0040	.016	2,647	218	+41
BC	Navajo to Ft. Lowell	188	59	129	RCP	1	--	5.0	.0050	.016	650	150	+21

Table 4.4. TSMS(5) Preliminary Storm Drain Sizing -- First Avenue Scenario

Reach	Location	Q100	Strt/ Chnl Cpcty	Cpcty Needed	Cnvync Type	nCells	B	D	So	n	L	Qcap	Excess Cpcty
	-- First Avenue --												
--	Exst 16'x8' Outlet to Inlet	2,454	1,255	1,199	RCBC	1	16.0	8.0	.0030	.016	869	1,255	+57
GH	Exst Inlet to Limberlost	2,203	--	2,203	RCBC	2	16.0	8.0	.0025	.016	1,239	2,292	+89
FG	Limberlost to Roger	2,010	95	1,915	RCBC	2	12.0	8.0	.0050	.016	1,321	2,266	+351
EF	Roger to Prince	1,956	59	1,897	RCBC	2	12.0	8.0	.0040	.016	2,647	2,027	+130
CE	Prince to Navajo	1,828	59	1,769	RCBC	2	12.0	8.0	.0040	.016	2,220	2,027	+258
DC	First Ave to Ft. Lowell	1,701	292	1,409	RCBC	2	10.0	8.0	.0050	.016	2,786	1,792	+383
	-- Navajo Wash --												
JK	Fairview to Oracle	1,101	--	1,101	Channel		20.0	6.0	.0025	.016	2,313	1,349	+248
IJ	Oracle to Stone	423	187	251	RCBC	1	8.0	5.0	.0055	.016	2,320	368	+117

Detailed Hydrologic Modeling Approach

With the preliminary sizing known, routing elements reflecting proposed storm drains were added to the hydrologic models for both scenarios. The general modeling approach is depicted in Figure 4.3 and the corresponding code in Figure 4.4. This process is as follows:

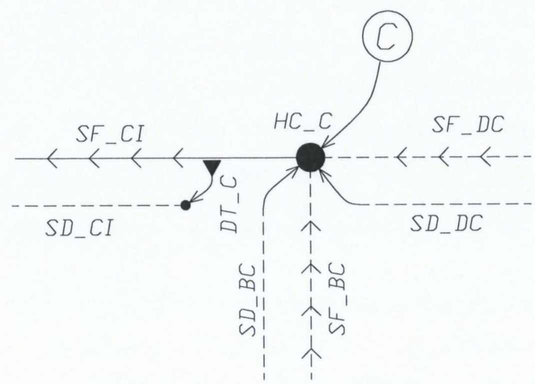


Figure 4.3. General Final Hydrologic Modeling Approach

o The offsite discharge at the point in question is recalled using a DR card sequence as in the previous models. In Figure 4.3 the offsite flow is shown as the circled letter C.

o The offsite flow is combined with any incoming street and storm drain flow from upstream using an HC card set. The combination element is designated "HC_" plus the letter designations of the current node, HC_C in the figure. HC in this case combines flow from two storm drain and two street flow elements with the offsite flow reaching C.

o The combined flow up to the capacity of the downstream storm drain reach is diverted using a DT card sequence. The diversion element is designated as "DT_" plus the letter designation of the current node or DT_C in this case.

o The flow remaining is routed to the next downstream node using the street flow routing elements previously developed. Street flow elements are designated with "SF_" and the current and next node letter designations (SF_CI in Figure 4.3). Channel flow elements designated with "CF_" are used in lieu of street flow elements for Navajo Wash downstream of Oracle Road.

o The storm drain flow previously diverted is recalled and routed to the next node via a storm drain element. This is also done using a table of stage/discharge/storage data similar to those for the surface flow

```
*
KK     C
KM    Off-Site flow at Navajo and First Avenue
DR POC_C
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
*
KK  HC_C
KM Combine D & C at Navajo
HC     5
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
*
KK DT_C
KM       Navajo Wash Approach
* Divert Capacity of storm drain (1,904.0 cfs).  Save temporarily as divC
* Storm Drain is RCBC B=8.0  D=8.0  nCls=3 So=.0045  n=.016  L=3449
DT divC
DI   0.0  1904.0   10000
DQ   0.0  1904.0  1904.0
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
*
KK SF_CI
KM Navajo Wash, First Ave to Stone Ave
* Half Roadway W=18.0  Sx=.030  n=.016  h=.50
* Half Parkway W=12.0  Sx=.020  n=.025
* L=3450  So=.0045
RS     1    FLOW      -1
SV     0    .421    .979   1.623   2.241   4.095   6.671   7.959   9.247
SQ     0      30      81     168     277     725    1606    2147    2748
SE     0    0.54    0.79    1.04    1.28    2.00    3.00    3.50    4.00
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
*
* Recall divC
KK DR_C
KM       Navajo Wash Approach
DR divC
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
*
* Route storm drain flow.
KK SD_CI
KM      Navajo Wash Approach
RS     1    FLOW      -1
SV     0     .63    1.27    1.90    2.53    3.17    3.80    4.43    5.07
SQ     0     129     363     644     952    1277    1613    1957    1904
SE     0    1.00    2.00    3.00    4.00    5.00    6.00    7.00    8.00
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
*
KK     I
KM  Recall N0220
DR POC_I
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
*
KK  HC_I
KM Combine with flow from C
HC     3
* -----+-------+-------+-------+-------+-------+-------+-------+-------+
```

Figure 4.4. Coding Corresponding to Figure 4.3 Diagram

elements. Use of a data table provides the ability to use multiple pipes and box culvert cells. It also accounts for storage attenuation which the kinematic wave approaches do not. Storm drain elements are designated similarly to surface flow elements except with "SD_" (SD_CI). As before, storm drain capacity is based on full flow with an increased Manning's "n" value of .016. The stage-storage-discharge tables for pipe reaches are constructed on depth increments of 1/8th the pipe diameter.

At that point, the street and storm drain flows are combined with the offsite flow at the next node and the process is repeated.

Spreadsheets used to develop this data and construct the necessary HEC-1 card sets are provided in Appendix 4e for the Navajo Wash Scenario and Appendix 4f for the First Avenue Scenario.

TSMS(6) & TSMS(7) -- Detailed Hydrologic Modeling of Scenarios

The routing elements reflecting proposed storm drains have been added to the hydrologic models for both scenarios. Those models are referred to here as TSMS(6) and TSMS(7) for the Navajo Wash Scenario and First Avenue Scenario respectively. The addition of 100-year storm drains has the effect of placing all or most of the flow in the proposed storm drain system. That changes the character of flow in the routing reach, potentially affecting the resulting discharges.

Figure 4.5 shows the modeling schematically for the Navajo Wash Scenario and the analysis results are presented in Table 4.5. Though the nature of the routing reach is substantially different, very little variation in peak discharge results is seen, and it is unnecessary to resize the storm drains based on the detailed modeling results. This is somewhat counter-intuitive as the addition of storm drain elements would be expected to increase peak discharges, much as confining all flow to the street cross-section did earlier. A representative HEC-1 printout based on PB = 3.151 corresponding to POC_K is included as Appendix 4g.

Similar results were found for the First Avenue Scenario as can be seen in Table 4.6. The modeling configuration is shown in Figure 4.6. Appendix 4h is a representative HEC-1 printout based on PB = 3.282 corresponding to POC_H.

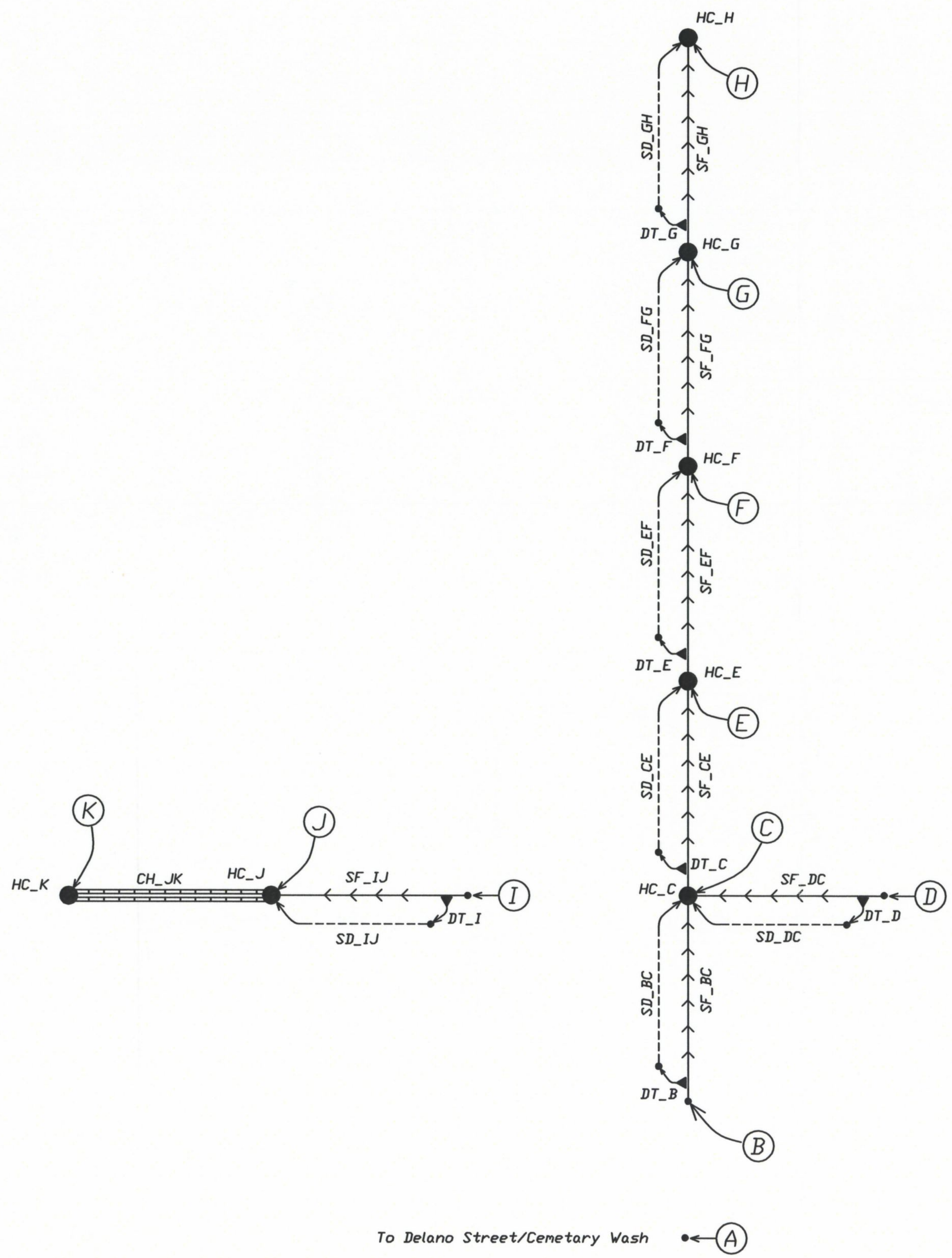
TABLE 4.5. TSMS(6) Detailed Discharges -- Navajo Wash Scenario

	Trib			100-year Discharge		
POC	Area	ARF	PB	TSMS(6)	TSMS(4)	Increase
A	.274	1.000	3.600	528	528	--
B	.107	1.000	3.600	188	188	--
D	3.017	.935	3.367	1,701	1,701	--
C	3.178	.931	3.350	1,800	1,828	-1.6%
I	3.359	.926	3.332	1,961	1,995	-1.7%
J	3.904	.911	3.280	2,528	2,583	-2.2%
K	5.554	.873	3.143	3,870	3,931	-1.6%
E	.192	1.000	3.600	236	236	--
F	.317	1.000	3.600	412	347	15.8%
G	.495	1.000	3.600	547	509	6.9%
H	.700	1.000	3.600	836	789	5.6%

TABLE 4.6. TSMS(7) Detailed Discharges -- First Avenue Scenario

	Trib			100-year Discharge		
POC	Area	ARF	PB	TSMS(7)	TSMS(5)	Increase
A	.274	1.000	3.600	528	528	--
B	.107	1.000	3.600	180	188	-4.4%
D	3.017	.935	3.367	1701	1,701	--
C	3.178	.931	3.350	1809	1,795	0.8%
E	3.369	.925	3.331	1944	1,935	0.5%
F	3.495	.922	3.319	2014	1,988	1.3%
G	3.672	.917	3.301	2207	2,176	1.4%
H	3.877	.912	3.282	2465	2,422	1.7%
I	.274	1.000	3.600	423	423	--
J	.274	1.000	3.600	1107	1,102	0.5%
K	1.996	.967	3.483	2851	2,849	0.1%

Figure 4.5.
TSMS(6) Final Modeling Configuration -- Navajo Wash Approach



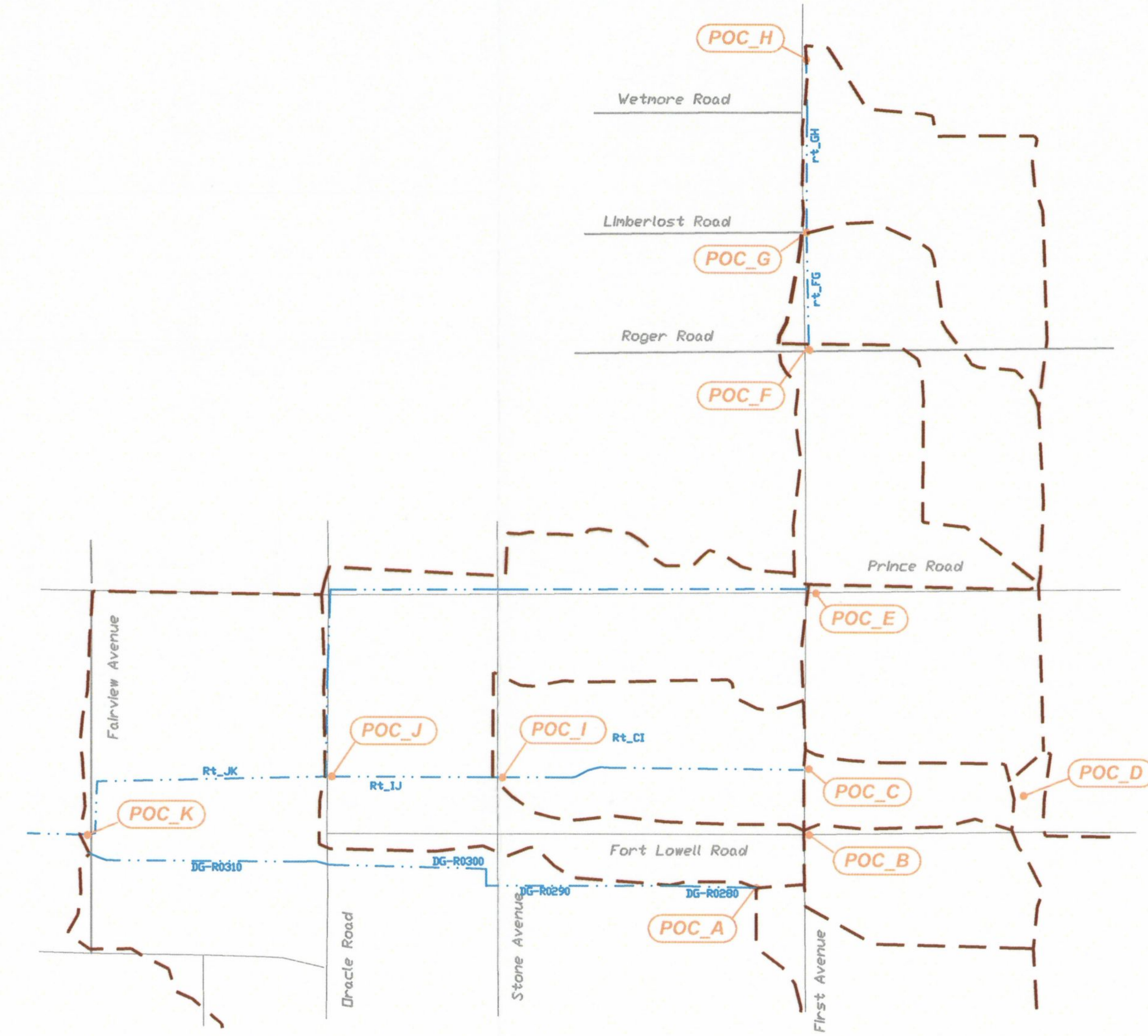


Figure 4.6
TSMS(7) Final Modeling Configuration -- First Avenue Scenario

Section 5
Conceptual Plans And Cost Estimates for Alternative Scenarios

The costs associated with the two scenarios have been evaluated through preparation of concept plans. The impacts to water and sewer facilities have been examined due to the potential for serious conflicts. Those efforts are documented in this section.

Concept Plans

The concept roadway and channel plans for future Navajo Road/Wash and First Avenue discussed in Section 3 were used to develop preliminary designs for the two scenarios. Storm drain mains are based on the information developed with the initial storm drain sizing in the previous section (TSMS(4) and TSMS(5)). The systems considered here are those required to handle the principle Navajo Wash and other TSMS flows. Catch basins and laterals associated with local drainage have not been included but would be roughly equivalent for both cases. The concept plans for the two scenarios are provided in Appendices 5a and 5b.

Water and Sewer Impacts

Because significant water and sewer impacts are anticipated with either scenario, specific estimates for relocations of those facilities are included. Sanitary sewers in First Avenue, Stone Avenue, and Oracle Road, are shallow with flat longitudinal slopes. They present a particular problem as there is little prospect of relocating them to avoid conflicting with the storm drains proposed here. It may be necessary for them to pass through the storm drains in a specially-constructed sleeve or as ductile iron pipe supported at each joint against the force of the storm drain flow. The likelihood of debris being caught is low given that all runoff will enter the storm drain system via catch basins.

Cost Evaluation

The concept plans have been used to determine preliminary cost estimates for the two scenarios. Costs for reinforced concrete channels and box culvert storm drains were derived using the engineering estimate for *I-10, Prince Road to 29th Street*. A composite cubic yard cost reflecting the cost of concrete, reinforcing steel, structural excavation, and structural backfill was developed based on structural concrete (f'c = 3,000 psi) at $400 per cubic yard; reinforcing steel @ $0.80 / lb assuming 150 lbs per CY of concrete or $120 / CY; and $80 per cubic yard to account for structural excavation and structure backfill. This resulted in a composite cost per cubic yard for concrete being $600 per cubic yard.

The channel structural section was assumed to consist of twelve-foot 12" thick retaining walls and a 15" thick bottom floor. The cubic yard quantities assumed per linear foot of box culverts is based on ADOT's standard details.

Unit costs for reinforced concrete pipe storm drains and utility relocation work were also based on the above-mentioned I-10 estimate and that of *Harrison Road, Speedway Boulevard to Old Spanish Trail.*

More discussion of unit costs and determination of quantity estimates are provided in Appendix 5c, and the results summarized in Table 5.1.

Conclusion and Recommendation Regarding Choice of Scenario

It can be seen from Table 5.1 that the First Avenue Scenario would cost approximately $8 million more than the Navajo Wash Scenario based on the 100-year design. For that reason, the Navajo Wash Scenario was selected to be carried forward. Subsequent evaluations of level of improvement and the impact of metering flow in the existing Mountain Avenue storm drain are limited to the Navajo Wash Scenario.

TABLE 5.1. Cost Comparison of Navajo Wash and First Avenue Scenarios

	Navajo Wash Scenario	First Avenue Scenario
1. Navajo Wash Downstream Channel Improvements	4.86	4.33
2. Navajo Wash Primary Storm Drain	17.73	3.56
3. Navajo Road Reconstruction	1.52	1.52
4. Arterial Crossing Reconstruction	0.44	0.44
5. Navajo Road Water & Sewer Relocation	1.40	0.61
6. First Avenue Primary Storm Drain	3.14	25.96
7. First Avenue Water & Sewer Relocation	0.36	1.09
	29.44	37.50
Costs are shown in $millions	Difference	8.06

SECTION 6
ALTERNATIVE LEVELS OF IMPROVEMENT

The storm drain proposals to this point have been sized such that all-weather access be provided in a 100-year storm. That requires that most of the flow be carried in the storm drain system as the capacity of the streets is relatively low in comparison to the 100-year discharges to be conveyed. The data in Table 5.1 indicates the cost of handling the 100-year storm in this manner will be on the order of $30 million.

The purpose of this section is to determine the cost savings that would be realized by designing to a lesser storm. This section makes that determination using 10-year and 2-year criteria.

TABLE 6.1. Precipitation Information for Range of Return Frequencies

		Trib		PB for					
POC	Reach	Area	ARF	100-yr	50-yr	25-yr	10-yr	5-yr	2-yr
	Non-Reduced PB -- 1-hr:			3.00	2.70	2.30	1.90	1.50	1.10
	*Non-Reduced PB -- 3-hr:			3.60	3.24	2.76	2.28	1.80	1.32
A	--	.274	1.000	3.600	3.240	2.760	2.280	1.800	1.320
B	BC	.107	1.000	3.600	3.240	2.760	2.280	1.800	1.320
D	DC	3.017	.935	3.367	3.030	2.581	2.133	1.684	1.235
C	CI	3.178	.931	3.350	3.015	2.569	2.122	1.675	1.228
I	IJ	3.359	.926	3.332	2.999	2.555	2.110	1.666	1.222
J	JK	3.904	.911	3.280	2.952	2.514	2.077	1.640	1.203
K	--	5.554	.873	3.143	2.829	2.410	1.991	1.572	1.153
E	EF	.192	1.000	3.600	3.240	2.760	2.280	1.800	1.320
F	FG	.125	1.000	3.600	3.240	2.760	2.280	1.800	1.320
G	GH	.317	1.000	3.600	3.240	2.760	2.280	1.800	1.320
H	--	.495	1.000	3.600	3.240	2.760	2.280	1.800	1.320

*1.2 time the one-hour duration value.

PRECIPITATION

Table 6.1 provides the precipitation information needed for evaluating the Navajo Wash Scenario for a range of return frequencies. Only the 10-year and 2-year are actually used in this study.

10-YEAR CRITERION

The hydrologic model TSMS(8) represents the 10-year storm drain system. It has been derived from TSMS(6) but with the applicable precipitation values. Initially this model was run with the 100-year storm drain in place to get preliminary discharges from which new storm drain sizes were determined. The storm drain routing elements were then updated to reflect the smaller capacities and the discharges recomputed. The resulting discharges changed very little and it was unnecessary to further adjust the storm drain sizes. The storm drain here fully contains the 10-year storm rather than the combined roadway/storm drain capacity as was done for the 100-year storm.

Table 6.2 summarizes the resulting discharges. It can be seen that the 10-year peaks range from 40% to 50% of the 100-year discharge which is in line with the guidelines of Reference 3 for suburban to highly urban settings.

It should be noted that while the value of PB is areally reduced for the 10-year event, the CN values have not. This results in a slightly conservative discharge results.

TABLE 6.2. -- TSMS(8) 10-Year Discharges

				100-yr		10-yr		
POC	Reach	Trib Area	ARF	PB	Q	PB	Q	Prcnt of 100-yr
A	--	.274	1.000	3.600	528	2.280	284	54%
B	BC	.107	1.000	3.600	188	2.280	91	48%
D	DC	3.017	.935	3.367	1,701	2.133	720	42%
C	CI	3.178	.931	3.350	1,800	2.122	772	43%
I	IJ	3.359	.926	3.332	1,961	2.110	869	44%
J	JK	3.904	.911	3.280	2,528	2.077	1,144	45%
K	--	5.554	.873	3.143	3,870	1.991	1,851	48%

Table 6.3 summarizes the storm drain sizing. The sizing calculations are provided in Appendix 6a. The corresponding storm drain routing data is calculated in Appendix 6b and the HEC-1 run for POC_K in Appendix 6c. Note that 100-year capacity is retained in the downstream channel and under drains.

TABLE 6.3. -- TSMS(8) 10-Year Storm Drain Sizing

Reach	Location	Q100	Strt/ Chnl Cpcty	Cpcty Needed	Cnvync Type	nCells	B	D	So	n	L	Qcap	Excess Cpcty
	-- Navajo Wash --												
JK	Fairview to Oracle	1,144	--	1,144	Channel	--	15.0	8.0	.0025	.016	--	1,378	+234
IJ	Oracle to Stone	869	--	869	RCBC	2	8.0	7.0	.0040	.016	1,874	1,000	+131
CI	Stone to First Ave	772	--	772	RCBC	2	8.0	6.0	.0050	.016	3,449	905	+133
DC	First Ave to Ft. Lowell	720	--	720	RCBC	2	8.0	5.0	.0060	.016	2,786	769	+49
	-- First Avenue --												
BC	Navajo to Ft. Lowell	91	--	91	RCP	1	--	5.0	.0050	.016	650	150	+59

2-YEAR CRITERION

This same process was repeated for the 2-year storm. The precipitations and storm drain sizing of TSMS(8) were adjusted to create TSMS(9). The resulting discharges are seen in Table 6.4.

TABLE 6.4 TSMS(9) 2-Year Discharges

POC	Reach	Trib Area	ARF	100-yr PB	100-yr Q	2-yr PB	2-yr Q	2-yr Prcnt of 100-yr
A	--	.274	1.000	3.600	528	1.320	122	23%
B	BC	.107	1.000	3.600	188	1.320	39	21%
D	DC	3.017	.935	3.367	1,701	1.235	49	3%
C	CI	3.178	.931	3.350	1,800	1.228	86	5%
I	IJ	3.359	.926	3.332	1,961	1.222	155	8%
J	JK	3.904	.911	3.280	2,528	1.203	299	12%
K	--	5.554	.873	3.143	3,870	1.153	697	18%

In this case, the 2-year discharges for POC_D through POC_I are less than 10% of the 100-year discharges, somewhat lower than that suggested by Reference 3. This is attributed to the flow intercepted by the existing storm drains upstream, particularly the Mountain Avenue storm drain, whose effects are more pronounced in smaller storms.

The results of the storm drain sizing based on these discharges are seen in Table 6.5. It is possible to convey these flows entirely with RCPs ranging in size from 48" to 72". Two 72" pipes are needed for reach IJ (Stone to Oracle) due to the very flat slope that exists there. The storm drain sizing calculations are provided in Appendix 6d with the storm drain routing data in Appendix 6e and the representative HEC-1 run for POC_K in Appendix 6f.

TABLE 6.5. TSMS(9) 2-Year Storm Drain Sizing

Reach	Location	Q100	Strt/ Chnl Cpcty	Cpcty Needed	Cnvync Type	nCells	B	D	So	n	L	Qcap	Excess Cpcty
	-- Navajo Wash --												
JK	Fairview to Oracle	713	--	713	Channel	--	25.0	8.0	.0025	.016	--	2,679	--
IJ	Oracle to Stone	313	--	313	RCP	2	--	6.0	.0040	.016	1,874	436	+123
CI	Stone to First Ave	163	--	163	RCP	1	--	6.0	.0050	.016	3,449	244	+81
DC	First Ave to Ft. Lowell	88	--	88	RCP	1	--	5.0	.0060	.016	2,786	164	+76
	-- First Avenue --												
BC	Navajo to Ft. Lowell	39	--	39	RCP	1	--	4.0	.0050	.016	650	83	+44

UNDER DRAINS

One of the goals of this project is to provide 100-year all-weather access for the major roadways crossing Navajo Wash. While all-weather access doesn't strictly require that the entire flow be underground, past experience has shown it to be extremely difficult to carry a traversable amount of flow -- say at one foot in depth -- across a roadway without adversely affecting the 100-year flow profile upstream. For this reason, under drains capable of conveying a 100-year flow fully under the roadway have been assumed here. These have been sized preliminarily using the same assumptions and procedures used for preliminary storm drain sizing. The longitudinal slope of the downstream storm drain reach has been taken to be the energy gradient.

Under drains consist of a culvert section functioning as an inverted siphon. Grate inlets upstream of the roadway crossing capture any flow not in the storm drain. Similar openings on the downstream side of the roadway allow flow exceeding the capacity of the downstream storm drain to return to the surface and avoid backing up storm drain and surface flow upstream. These structures are sized for the full 100-year discharge regardless of the return frequency for which the storm drain is sized. The preliminary under drain sizing calculations are summarized in Table 6.6 The storm drain slope downstream of the crossing has been used in these calculations.

TABLE 6.6. UNDER DRAIN SIZING CALCULATIONS

Under-Drain	Q	nCls	B	D	So	A	P	R	Qcap	Excess Cap
Oracle Road	2,731	4	12.00	7.00	.0030	336.0	152.0	2.21	2,908	+177
Stone Avenue	2,172	3	12.00	7.00	.0040	252.0	114.0	2.21	2,519	+347
First Avenue	1,982	3	10.00	7.00	.0050	210.0	102.0	2.06	2,238	+256
Ft. Lowell Road	1,879	3	10.00	7.00	.0060	210.0	102.0	2.06	2,452	+573

COST EVALUATION

Table 6.7 shows the cost of the 100-year Navajo Wash Scenario storm drain system developed previously with the under drains added in. Tables 6.8 and 6.9 show the corresponding costs of storm drains for the 10-year and 2-year criteria respectively. Under drains are included in this table. The cost breakdown between under drains and the connecting reaches of storm drain is indicated.

TABLE 6.7. Cost of 100-Year Criterion

Reach	Sta	to Sta	L	TSMS(6) -- 100-year Storm Drain Type	Conc. per LF	Cost per LF	Cost
Oracle Under-Drain	99+42	101+90	248	4 - 12' x 7' RCBC	6.342	$3,805	$943,690
IJ - Oracle to Stone	101+90	117+98	1,608	3 - 10' x 7' RCBC	3.394	$2,036	$3,274,531
Stone Under-Drain	117+98	119+68	170	3 - 12' x 7' RCBC	4.543	$2,726	$463,386
CI - Stone to First Ave	119+68	152+08	3,240	3 - 8' x 7' RCBC	2.579	$1,547	$5,013,576
First Avenue Under-Drain	152+08	154+38	230	3 - 10' x 7' RCBC	3.394	$2,036	$468,372
DC - First Ave to Ft. Lowell	154+38	180+41	2,603	3 - 8' x 7' RCBC	2.579	$1,547	$4,027,882
Ft. Lowell Road Under-Drain	180+41	182+21	180	3 - 10' x 7' RCBC	3.394	$2,036	$366,552
DC - Ft. Lowell to Hedrick	182+21	187+66	545	3 - 8' x 7' RCBC	2.579	$1,547	$843,333
BC -- Ft. Lowell to Navajo			650	1 - 60" RCP	--	$300	$195,000
						Subtotal:	$15,596,322
						Plus 20% Contingency & Misc.:	$18,715,586
						Storm Drain:	$16,025,187
						Under Drains:	$2,690,400

TABLE 6.8. Cost of 10-year Criterion

Reach	Sta	to Sta	L	TSMS(8) -- 10-Year: Storm Drain Type	Conc. per LF	Cost per LF	Cost
Oracle Under-Drain	99+42	101+90	248	4 - 12' x 7' RCBC	6.342	$3,805	$943,690
IJ - Oracle to Stone	101+90	117+98	1,608	2 - 8' x 7' RCBC	1.808	$1,085	$1,744,358
Stone Under-Drain	117+98	119+68	170	3 - 12' x 7' RCBC	4.543	$2,726	$463,386
CI - Stone to First Ave	119+68	152+08	3,240	2 - 8' x 6' RCBC	1.725	$1,035	$3,353,400
First Avenue Under-Drain	152+08	154+38	230	3 - 10' x 7' RCBC	3.394	$2,036	$468,372
DC - First Ave to Ft. Lowell	154+38	180+41	2,603	2 - 8' x 5' RCBC	1.642	$985	$2,564,476
Ft. Lowell Road Under-Drain	180+41	182+21	180	3 - 10' x 7' RCBC	3.394	$2,036	$366,552
DC - Ft. Lowell to Hedrick	182+21	187+66	545	2 - 8' x 5' RCBC	1.642	$985	$536,934
BC -- Ft. Lowell to Navajo			650	1 - 60" RCP	--	$300	$195,000
						Subtotal:	$10,636,168
						Plus 20% Contingency & Misc.:	$12,763,401
						Storm Drain:	$10,073,002
						Under Drains:	$2,690,400

TABLE 6.9. Cost of 2-year Criterion

Reach	Sta	to Sta	L	TSMS(9) -- 2-year: Storm Drain Type	Conc. per LF	Cost per LF	Cost
Oracle Under-Drain	99+42	101+90	248	4 - 12' x 7' RCBC	6.342	$3,805	$943,690
IJ - Oracle to Stone	101+90	117+98	1,608	2 - 72" RCP	--	$400	$643,200
Stone Under-Drain	117+98	119+68	170	3 - 12' x 7' RCBC	4.543	$2,726	$463,386
CI - Stone to First Ave	119+68	152+08	3,240	1 - 72" RCP	--	$400	$1,296,000
First Avenue Under-Drain	152+08	154+38	230	3 - 10' x 7' RCBC	3.394	$2,036	$468,372
DC - First Ave to Ft. Lowell	154+38	180+41	2,603	1 - 60" RCP	--	$300	$780,900
Ft. Lowell Road Under-Drain	180+41	182+21	180	3 - 10' x 7' RCBC	3.394	$2,036	$366,552
DC - Ft. Lowell to Hedrick	182+21	187+66	545	1 - 48" RCP	--	$200	$109,000
BC -- Ft. Lowell to Navajo			650	1 - 48" RCP	--	$200	$130,000
						Subtotal:	$5,201,100
						Plus 20% Contingency & Misc.:	$6,241,320
						Storm Drain:	$3,550,920
						Under Drains:	$2,690,400

A comparison of costs of the 100-year, 10-year and 2-year systems is shown in Table 6.10. The cost determined earlier for the 100-year downstream channel is assumed in each case. Design for a lesser storm can be considered during final design.

While likely that it will be found necessary to carry the entire 100-year storm under the arterial crossings to avoid increasing the upstream floodplain elevation, this also should be examined more closely during final design.

The 100-year costs for water and sewer relocation and arterial construction have been assumed as well. The cost differential shown here is for only the storm drain mains themselves. It can be seen a savings of about $6 million would be realized by designing for a 10-year storm, and $12 to $13 million by designing for a 2-year storm.

TABLE 6.10. Cost of Various Return Frequencies
Costs are shown in $millions

	100-Year	10-Year	2-Year
1. Downstream Channel	$4.64	$4.64	$4.64
2. Primary Storm Drain	16.03	10.07	3.55
3. Under Drains	2.69	2.69	2.69
4. Navajo Road Reconstruction	1.52	1.52	1.52
5. Arterial Reconstruction	0.44	0.44	0.44
6. Water & Sewer Relocation	1.40	1.40	1.40
Total:	$26.72	$20.76	$14.24
Savings Over 100-Year Criterion:	--	$5.95	$12.47
Percent:	--	22.3%	46.7%

SECTION 7. IMPACT OF METERING INFLOW TO EXISTING MOUNTAIN AVENUE STORM DRAIN

The possibility of eliminating a new storm drain currently planned for the Mountain Avenue construction is investigated in this section. This would be accomplished by reducing the amount of flow being intercepted by the existing 84" Mountain Avenue storm drain leaving enough capacity for that project's drainage. This would both reduce the cost and simplify construction of Mountain Avenue.

The study of Reference 1 found that reducing the intercepted amount from 355 cfs to about 225 cfs would accomplish that. To provide a degree of safety, the metered amount has been limited here to 175 cfs.

This will add to the amount of flow at the head of Navajo Wash and potentially increase the size of the Navajo Wash storm drain. The purpose of this section is to determine the size increases needed for the 100-year, 10-year, and 2-year proposals and the corresponding increase in cost. The cost savings to the Mountain Avenue project realized by eliminating the new Mountain Avenue storm drain is first determined to assess the overall cost effectiveness of the metering proposal.

UPDATED COST OF MOUNTAIN AVENUE STORM DRAIN

The cost estimate for ***Mountain Avenue, Roger Road to Ft. Lowell Road*** has been updated several times to reflect the inflation in construction cost. The total project cost, first estimated to be $5.7 in 2002, had increased to $7.8 million by November 2005. The most recent estimate prepared in June 2006 was $11.7. The June 2006 estimate was based on actual bids received from two contractors for ***Harrison Road, Speedway Boulevard to Old Spanish Trail*** which at over $14 million were within $100,000 or less than one percent of each other.

The estimate of cost associated with the project storm drain increased from $2.3 million in 2002 to $5.3 million in June 2006, nearly half the total project cost. Costs associated with that storm drain includes RCP as large as 78", a 331' length of 8' x 6' RCBC (where shallow cover requires), extending the storm drain 2,200' downstream of the project to a location just south of Prospect Lane, and considerable relocation of water and sewer including a 24" sanitary sewer where flow bypass during construction would be needed. The applicable portions of this cost estimate are provided in Appendix 7a.

TSMS(10) -- 100-YEAR DESIGN WITH METERING

To determine the effect on a 100-year design, the maximum diversion at the 84" storm drain inlet in the TSMS(6) model was reduced from 355 to 175 cfs. Storm drains were resized based on the resulting discharges and the DT card sequences representing storm drain routing elements updated accordingly. Running the model again resulted in only slightly different discharges and it was not necessary to further adjust the storm drain sizing.

The resulting discharges are shown in Table 7.1. It can be seen that the increase in discharges are on the order of 200 cfs. Note that the discharge increase immediately downstream of the diversion would be 180 cfs (the difference of 355 and 175 cfs).

TABLE 7.1. TSMS(10) -- 100-year Metered Conditions Discharges

		Trib			100-year Discharge			
POC	Reach	Area	ARF	PB	TSMS(10)	TSMS(6)	Incrs	Prcnt
A	--	.274	1.000	3.600	528	528	--	--
B	BC	.107	1.000	3.600	188	188	--	--
D	DC	3.017	.935	3.367	1,879	1,701	178	10.5%
C	CI	3.178	.931	3.350	1,982	1,800	182	10.1%
I	IJ	3.359	.926	3.332	2,172	1,961	211	10.8%
J	JK	3.904	.911	3.280	2,731	2,528	203	8.0%
K	--	5.554	.873	3.143	4,050	3,870	180	4.7%

The HEC-1 printout corresponding to POC_K is provided as Appendix 7b. Only the input data and results differing from TSMS(6) are included. The storm drain resizing calculations are provided in Appendix 7c and summarized in Table 7.2. The higher discharges required increasing the barrel width between Oracle and Stone from 8' to 10'.

Table 7.3 compares the sizing for the 100-year storm drain with and without metering. The additional cost of metering is about $1.9 million, $3.4 million less than the $5.3 million cost of the Mountain Avenue storm drain.

TABLE 7.2. TSMS(10) -- 100-Year Storm Drain Sizing for Metered Conditions

Reach	Location	Q100	Strt/ Chnl Cpcty	Cpcty Needed	Cnvync Type	nCells	B	D	So	n	L	Qcap	Excess Cpcty
	-- Navajo Wash --												
JK	Fairview to Oracle	2,731	--	2,731	Channel	--	25.0	10.0	.0025	.016	--	3,651	--
IJ	Oracle to Stone	2,172	251	1,921	RCBC	3	10.0	7.0	.0040	.016	1,874	2,002	+81
CI	Stone to First Ave	1,982	277	1,705	RCBC	3	10.0	7.0	.0050	.016	3,449	2,238	+533
DC	1st Ave to Ft. Lowell	1,879	292	1,587	RCBC	3	8.0	7.0	.0060	.016	2,786	1,837	+250
	-- First Avenue --												
BC	Navajo to Ft. Lowell	188	59	129	RCBC	1	--	5.0	.0050	.016	650	150	+21

TABLE 7.3. 100-Year Cost Comparison of Metered vs. Un-Metered Conditions

		TSMS(10) -- 100-year Metered Proposal				TSMS(6) -- 100-year Un-Metered	
Reach	L	Storm Drain Type	Conc. per LF	Cost per LF	Cost	Storm Drain Type	Cost
IJ - Oracle to Stone	1,608	3 - 10' x 7' RCBC	3.394	$2,036	$3,274,531	3 -10' x 7' RCBC	$3,274,531
CI - Stone to First Ave	3,240	3 - 10' x 7' RCBC	3.394	$2,036	6,597,936	3 - 8' x 7' RCBC	5,013,576
DC - First Ave to Ft. Lowell	2,603	3 - 8' x 7' RCBC	2.579	$1,547	4,027,882	3 - 8' x 7' RCBC	4,027,882
DC - Ft. Lowell to Hedrick	545	3 - 8' x 7' RCBC	2.579	$1,547	843,333	3 - 8' x 7' RCBC	843,333
BC -- Ft. Lowell to Navajo	650	1 - 60" RCP	--	$300	195,000	1 - 60" RCP	195,000
				Total:	$14,938,682		$13,354,322
		Plus 20% Contingency/Miscellaneous:			$17,926,419		$16,025,187
				Increase:	$1,901,232		
				Percent:	11.9%		

TSMS(11) -- 10-YEAR DESIGN WITH METERING

Repeating this exercise for the ten-year storm produces a similar but more pronounced result, particularly at the upper end of the storm drain system. The resulting discharges are shown in Table 7.4. While the magnitude of the increase is roughly equivalent, the percent of increase is larger due to the lesser total flow of the ten-year storm. The HEC-1 printout corresponding to POC_K is provided as Appendix 7d.

TABLE 7.4. TSMS(11) -- 10-year Metered Conditions Discharges

POC	Reach	Trib Area	ARF	PB	10-year Discharge TSMS(11)	TSMS(8)	Incrs	Prcnt
A	--	.274	1.000	2.280	284	284	--	--
B	BC	.107	1.000	2.280	91	91	--	--
D	DC	3.017	.935	2.133	893	720	173	24.0%
C	CI	3.178	.931	2.122	952	772	180	23.3%
I	IJ	3.359	.926	2.110	1,090	869	221	25.4%
J	JK	3.904	.911	2.077	1,328	1,144	184	16.1%
K	--	5.554	.873	1.991	2,036	1,851	185	10.0%

The change in storm drain sizing is shown in Table 7.5. The capacity of the reach from Oracle to Stone has been increased by widening the cells. The reaches from Stone to First, and from First to Fort Lowell have been increased by deepening the structure. The storm drain resizing calculations are provided in Appendix 7e

TABLE 7.5. TSMS(11) -- 10-Year Storm Drain Sizing for Metered Conditions

Reach	Location	Q100	Strt/ Chnl Cpcty	Cpcty Needed	Cnvync Type	nCells	B	D	So	n	L	Qcap	Excess Cpcty
	-- Navajo Wash --												
JK	Fairview to Oracle	1,328	--	1,328	Channel	--	25.0	10.0	.0025	.016	--	3,651	--
IJ	Oracle to Stone	1,090	--	1,090	RCBC	2	10.0	7.0	.0040	.016	1,874	1,334	+244
CI	Stone to First Ave	952	--	952	RCBC	2	8.0	7.0	.0050	.016	3,449	1,118	+166
DC	First Ave to Ft. Lowell	893	--	893	RCBC	2	8.0	6.0	.0060	.016	2,786	992	+99
	-- First Avenue --												
BC	Navajo to Ft. Lowell	91	--	91	RCBC	1	--	5.0	.0050	.016	650	150	+59

A cost comparison of the 10-year metered and un-metered conditions is shown in Table 7.6. The cost increase of the metered condition is $1.0 million, about 10%.

TABLE 7.6. TSMS(11) -- 10-Year Cost Comparison of Metered vs. Un-Metered Conditions

		TSMS(11) -- 10-year Metered Proposal				TSMS(8) -- 10-year Un-Metered	
Reach	L	Storm Drain Type	Conc. per LF	Cost per LF	Cost	Storm Drain Type	Cost
IJ - Oracle to Stone	1,608	2 - 10' x 7' RCBC	2.372	$1,423	$2,288,506	2 - 8' x 7' RCBC	$1,744,358
CI - Stone to First Ave	3,240	2 - 8' x 7' RCBC	1.808	$1,085	3,514,752	2 - 8' x 6' RCBC	3,353,400
DC - First Ave to Ft. Lowell	2,603	2 - 8' x 6' RCBC	1.725	$1,035	2,694,105	2 - 8' x 5' RCBC	2,564,476
DC - Ft. Lowell to Hedrick	545	2 - 8' x 6' RCBC	1.725	$1,035	564,075	2 - 8' x 5' RCBC	536,934
BC -- Ft. Lowell to Navajo	650	1 - 60" RCP	--	$300	195,000	1 - 60" RCP	195,000
				Total:	$9,256,438		$8,394,168
		Plus 20% Contingency/Miscellaneous:			$11,107,725		$10,073,002
				Increase:	$1,034,724		
				Percent:	10.3%		

TSMS(12) -- 2-YEAR DESIGN WITH METERING

The results of the metering proposal on discharge for a two year storm are presented in Tables 7.7. The increase in magnitude of discharge is less than with the 100 or 10-year storms simply because there is less overall flow. Similarly, the percentage increase is high, particularly at the upper end, because the 84" storm drain intercepts a larger portion of the total flow. The HEC-1 printout for the 2-year metered conditions is found in Appendix 7f.

TABLE 7.7. TSMS(12) -- 2-year Metered Conditions Discharges

POC	Reach	Trib Area	ARF	PB	2-year Discharge TSMS(12)	TSMS(9)	Incrs	Prcnt
A	--	.274	1.000	1.320	122	122	--	--
B	BC	.107	1.000	1.320	39	39	--	--
D	DC	3.017	.935	1.235	199	49	150	306%
C	CI	3.178	.931	1.228	236	86	150	174%
I	IJ	3.359	.926	1.222	305	155	150	96.8%
J	JK	3.904	.911	1.203	435	299	136	45.5%
K	--	5.554	.873	1.153	735	697	38	5.5%

The determination of corresponding storm drain sizing is summarized in Table 7.9. The calculations are provided in Appendix 7g.

A comparison of sizing and cost of the two approaches is provided in Table 7.9. It can be seen that it is necessary to increase the size of the upper reaches of RCP, raising the cost about $450,000 or 12.5%.

TABLE 7.8. TSMS(12) -- 2-Year Storm Drain Sizing for Metered Conditions

Reach	Location	Q100	Strt/ Chnl Cpcty	Cpcty Needed	Cnvync Type	nCells	B	D	So	n	L	Qcap	Excess Cpcty
	-- Navajo Wash --												
JK	Fairview to Oracle	435	--	435	Channel	--	25.0	10.0	.0025	.016	--	3,651	--
IJ	Oracle to Stone	305	--	305	RCBC	2	--	6.0	.0040	.016	1,874	436	+131
CI	Stone to First Ave	236	--	236	RCBC	1	--	6.0	.0050	.016	3,449	244	+8
DC	First Ave to Ft. Lowell	199	--	199	RCBC	1	--	6.0	.0060	.016	2,786	267	+68
	-- First Avenue --												
BC	Navajo to Ft. Lowell	39	--	39	RCBC	1	--	4.0	.0050	.016	650	83	+44

TABLE 7.9. TSMS(12) -- 2-Year Cost Comparison of Metered vs. Un-Metered Conditions

		TSMS(12) -- 2-year Metered Conditions				TSMS(9) -- 2-year Un-Metered	
Reach	L	Storm Drain Type	Conc per LF	Cost per LF	Cost	Storm Drain Type	Cost
IJ - Oracle to Stone	1,608	2 - 72" RCP	--	$400	643,200	2 - 72" RCP	$643,200
CI - Stone to First Ave	3,240	1 - 72" RCP	--	$400	1,296,000	1 - 72" RCP	$1,296,000
DC - First Ave to Ft. Lowell	2,603	1 - 72" RCP	--	$400	1,041,200	1 - 60" RCP	$780,900
DC - Ft. Lowell to Hedrick	545	1 - 72" RCP	--	$400	218,000	1 - 48" RCP	$109,000
BC -- Ft. Lowell to Navajo	650	1 - 48" RCP	--	$200	130,000	1 - 48" RCP	$130,000
				Subtotal:	$3,328,400		$2,959,100
		Plus 20% Contingency / Misc.:			$3,994,080		$3,550,920
				Increase:	$443,160		
				Percent:	12.5%		

SUMMARY OF METERING COSTS FOR VARIOUS RETURN FREQUENCIES

The costs of the metered and un-metered proposals for each return frequency are summarized in Table 7.10 for convenience.

TABLE 7.10. Summary of Total Costs for Metered vs. Un-Metered Conditions

	100-Year		10-Year		2-Year	
	Un-Metered	Metered	Un-Metered	Metered	Un-Metered	Metered
1. Downstream Channel	$4.64	$4.64	$4.64	$4.64	$4.64	$4.64
2. Primary Storm Drain	16.03	17.93	10.07	11.11	3.55	3.99
3. Under Drains	2.69	2.69	2.69	2.69	2.69	2.69
4. Navajo Road Reconstruction	1.52	1.52	1.52	1.52	1.52	1.52
5. Arterial Reconstruction	0.44	0.44	0.44	0.44	0.44	0.44
6. Water & Sewer Relocation	1.40	1.40	1.40	1.40	1.40	1.40
Total:	$26.72	$28.62	$20.76	$21.80	$14.24	$14.68
Increased Cost of Metering:		$1.90		$1.03		$0.44
Less Mountain Avenue Savings:		5.35		5.35		5.35
Overall Net Savings:		$3.45		$4.32		$4.91

SECTION 8. HYDRAULIC EVALUATION OF IMPROVEMENT LEVELS

This section describes the process and presents the results of the hydraulic evaluation of surface flow associated under various conditions. The purpose is to provide a sense of the flooding mitigation that would be realized under various levels of improvement.

DISCHARGES

The discharges corresponding to the metered conditions for the 100, 10, and 2-year storms, taken from Tables 7.1, 7.4, and 7.7 respectively, are presented in Table 8.1. Also shown are the differences in the 100-year and 10 and 2-year storms which are the amounts of flow remaining on the surface during those storms with the corresponding storm drain system in place.

HYDRAULIC MODEL

The hydraulic model used for this effort is derived from the FEMA HEC-2 model that is currently in effect. As with the TSMS hydrologic modeling, the FEMA model has been refined.

In particular, the FEMA cross-sections have been updated based on the more detailed survey, project mapping, and 2005 PAG mapping. This also adjusts the floodplain elevation to the project datum. The special bridge (SB) routine modeling of the existing culverts at Fairview and Oracle have been replaced with special culvert modeling (SC). Discharges have also been adjusted per Table 8.1 where the FEMA 100-year discharge is also shown for comparison.

TABLE 8.1. Surface Discharges

POC	Reach	HEC-2 Cross-Sections	100-yr FEMA	100-yr TSMS(10)	10-yr TSMS(11)	2-yr TSMS(12)	(100-yr) - (10-yr)	(100-yr) - (2-yr)
K	--	12-14	3,000	4,050	2,036	735	2,014	3,315
J	JK	15-24	2,021	2,731	1,328	435	1,403	2,296
I	CI	25-29	2,100	1,982	952	236	1,030	1,746
C	IJ	30-35	2,100	2,173	1,090	305	1,083	1,868
D	DC	36-40	2,100	1,879	893	199	986	1,680

The location of cross-section and the Manning's "n" roughness coefficients have not been changed. The presence of structures in the overbank is still reflected by high "n" values rather than actual removal of ineffective flow areas. Also unchanged is the assumption of subcritical flow for the purpose of establishing the floodplain depth and width. The implications of retaining these elements of the FEMA model are discussed below.

DEVELOPMENT OF REFINED SECTIONS

The refined cross-sections were developed with InRoads using the merged surfaces of the manually-surveyed cross-sections and the photogrammetry (described in Section 3). Where flooding was found to extend beyond the project mapping, supplement points were taken from the PAG mapping.

The updated sections were exported from InRoads into an AutoCAD drawing from which they were subsequently transferred into a spreadsheet where GR cards and other applicable data cards were constructed.

The spreadsheet was also used to plot the new sections on top of the FEMA cross-sections such that their differences can be readily viewed. The FEMA sections have been raised 2.2' to adjust them to the project datum. They have also been translated varying distances left or right to line up the centerlines of Navajo Wash.

The data and plots of these sections are provided in Appendix 8a. The locations of the sections are shown on the map of Appendix 8b superimposed on the project and PAG mapping.

RESULTS

Table 8.2 summarizes depths for various discharge conditions. The FEMA results, based on original cross-sections and discharges are also shown for comparison. Starred depths indicate critical flow. Average depths of flow in the Navajo roadway -- reflected by Sections 28 through 40 -- are also shown.

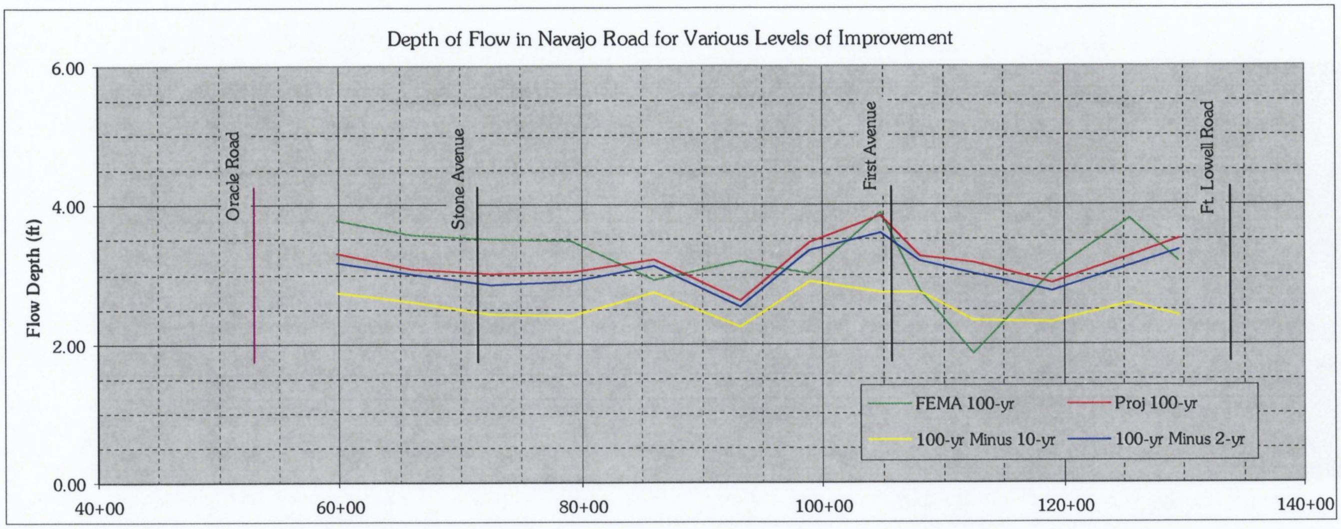
It can be seen that under existing conditions the average depth of flow in Navajo Road (as determined from Sections 28-40) is about 3.2' in a 100-year storm. The addition of a ten-year storm drain would reduce the depth of flow by about 0.6' or about 20%. The addition of a two-year storm drain would reduce the 100-year flow depth only about 0.2' or 4%. A plot of depths in the roadway is provided as Figure 8.1.

Figure 8.2 is a map of the project area showing the extent of flooding that would be expected in a 100-year storm under existing conditions and with a ten-year storm drain in place.. These are indicated by solid red and blue lines respectively. The approximate location of the current FEMA 100-year floodplain is also shown for reference. Affected buildings and parcels are shown to indicate the number and size of structures impacted in each case.

TABLE 8.2. FLOW DEPTHS FOR VARIOUS LEVELS OF IMPROVEMENT

Sec	Sta	FEMA 100-yr	Proj 100-yr Minus 10-yr	Proj 100-yr Minus 2-yr	Proj 100-yr
	Downstream of POC_K: Q=	3,000	2,014	3,315	4,050
12	0+00	7.47*	5.92*	7.96*	8.93*
13	4+75	7.45*	6.21*	8.18*	9.12*
14	8+25	7.24*	5.91*	7.73*	8.61*
	K to J: Q=	2,021	1,403	2,296	2,731
15	9+75	6.96	4.81*	7.30	8.65
16	10+15	6.37*	4.97*	6.85	8.16
17	10+77	9.23	8.93	8.61	7.53
18	11+27	9.57	8.80	9.30	9.90
19	15+77	8.58*	7.40	7.74*	8.55*
20	20+77	7.74	6.28*	8.15*	8.82*
21	26+77	9.77	8.30	10.19	10.98
22	32+77	9.29*	7.70	9.51	9.67
23	38+77	9.16*	8.15	9.07	9.46
24	41+77	8.10	7.48	8.12	8.67*
	J to I: Q=	2,100	1,030	1,746	1,982
25	42+27	6.10	6.91	7.66	7.55
26	43+55	7.12	4.42	6.72	7.33
27	44+05	5.98*	3.86*	4.68*	5.52
28	49+82	3.77	2.74*	3.18*	3.31*
29	55+96	3.58*	2.61*	3.01*	3.08*
	I to C: Q=	2,100	1,083	1,868	2,173
30	62+46	3.51*	2.42*	2.85*	3.01*
31	69+16	3.48*	2.41*	2.90*	3.03*
32	76+07	2.92*	2.74*	3.12*	3.22*
33	83+15	3.19*	2.24*	2.53*	2.62*
34	88+87	3.01*	2.90*	3.36*	3.47*
35	94+77	3.90*	2.74*	3.60*	3.84*
	C to D: Q=	2,100	986	1,680	1,879
36	98+11	2.77*	2.73*	3.20*	3.25*
37	102+52	1.85	2.33*	3.02*	3.18*
38	109+02	3.03*	2.32*	2.77*	2.87*
39	115+52	3.80*	2.57*	3.13*	3.25*
40	119+55	3.19*	2.41*	3.35*	3.51*
	Average Depth in Navajo Road (Sections 28-40):	3.23	2.55	3.08	3.20
	Percent of 100-year Depth:		79.6%	96.1%	100%

FIGURE 8.1. PLOT OF FLOW DEPTHS IN NAVAJO ROAD UNDER VARIOUS IMPROVEMENT SCENARIOS



It can be seen that in terms of flooding, the improvement afforded by the ten-year storm drain would be relatively modest. It is noted, however, that the ten-year scenario will provide all-weather access on the crossings at Fort Lowell, First Avenue, Stone and Oracle Road. There would also be a substantial improvement to access along Navajo Road itself in smaller storms.

The limits of flooding with a two-year storm drain in place has not been shown since the reduction in flow depth is so minimal.

BREAKOUT OF FLOW

FEMA typically does not consider the effects of flow breaking away from the main water course. There are in this case, however, locations where the topography does not fully contain the full flow. In such cases the highest point along the cross section has been taken to be the extent of flooding. Areas of anticipated breakout are indicated by dashed lines in Figure 8.2.

Flow breaking out northward upstream of First Avenue is captured by First Avenue and carried northward out of the Navajo Wash watershed. To evaluate this condition more accurately, breakout flow should be determined using HEC-2's (or HEC-RAS's) split flow capability, and discharges at downstream sections adjusted accordingly.

Flow breaking out northward downstream of First Avenue flows parallel to and independently of that in Navajo Road in Mohave, Yavapai, and possibly Prince Road until reaching Oracle Road where it is carried southward back to Navajo Wash. A split flow analysis would be applied here as well if accurate flow depths are needed. In this case, the flow would be returned to the main channel at Oracle Road.

Flow breaking out to the south in Castro and Balboa Avenues enters Cemetery Wash. The flows of Navajo Wash and Cemetery Wash merge at Oracle, a fact which is not included in current TSMS modeling or the project modeling used here. If a less-than-100-year design is adopted, these flows should be combined and split using rating data reflecting the combined existing and proposed conditions along Oracle Road.

Figure 8.2. Map of Flooding Impact of Various Improvement Proposals

FLOW REGIME

It was noted that the HEC-2 analysis resulted in critical depth all along Navajo Road, suggesting that flow might in fact be supercritical. To test this, the HEC-2 data was exported to HEC-RAS and a mixed flow profile determined. It was found that the flow in Navajo Road is in fact supercritical, but the resulting depths are only slightly less than critical depth. Table 8.3 summarizes these results. The actual and critical flow profiles are in plotted in Figure 8.3.

Table 8.3. Results of HEC-RAS Analysis

		Chnl	Critical WS		Computed WS		+/-
SECNO	Q	Invert	Elev	Depth	Elev	Depth	Crit
28	1,982	2329.1	2332.27	3.17	2332.09	2.99	-0.18
29	1,982	2332.3	2335.42	3.12	2335.32	3.02	-0.10
30	2,173	2334.6	2337.72	3.12	2337.51	2.91	-0.21
31	2,173	2337.3	2340.35	3.05	2340.01	2.71	-0.34
32	2,173	2340.3	2343.57	3.27	2343.36	3.06	-0.21
33	2,173	2344.7	2347.33	2.63	2347.10	2.40	-0.23
34	2,173	2347.6	2351.18	3.58	2350.59	2.99	-0.59
35	2,173	2350.6	2354.45	3.85	2354.22	3.62	-0.23
36	1,879	2353.1	2356.41	3.31	2355.81	2.71	-0.60
37	1,879	2356.5	2359.83	3.33	2359.56	3.06	-0.27
38	1,879	2359.8	2362.70	2.90	2362.61	2.81	-0.09
39	1,879	2362.9	2366.16	3.26	2365.53	2.63	-0.63
40	1,879	2365.1	2368.59	3.49	2368.59	3.49	--

DETAILED ANALYSIS

Should it be necessary to re-perform the HEC-2 analysis, the cross-sections should be revised to actually block flow rather than using an increased "n" to reflect the presence of walls and structures. Since flow is critical, "n" has no effect on depth of flow and the resistance to flow presented by the flooded development is not considered. This would be necessary only if less than a 100-year storm drain is installed.

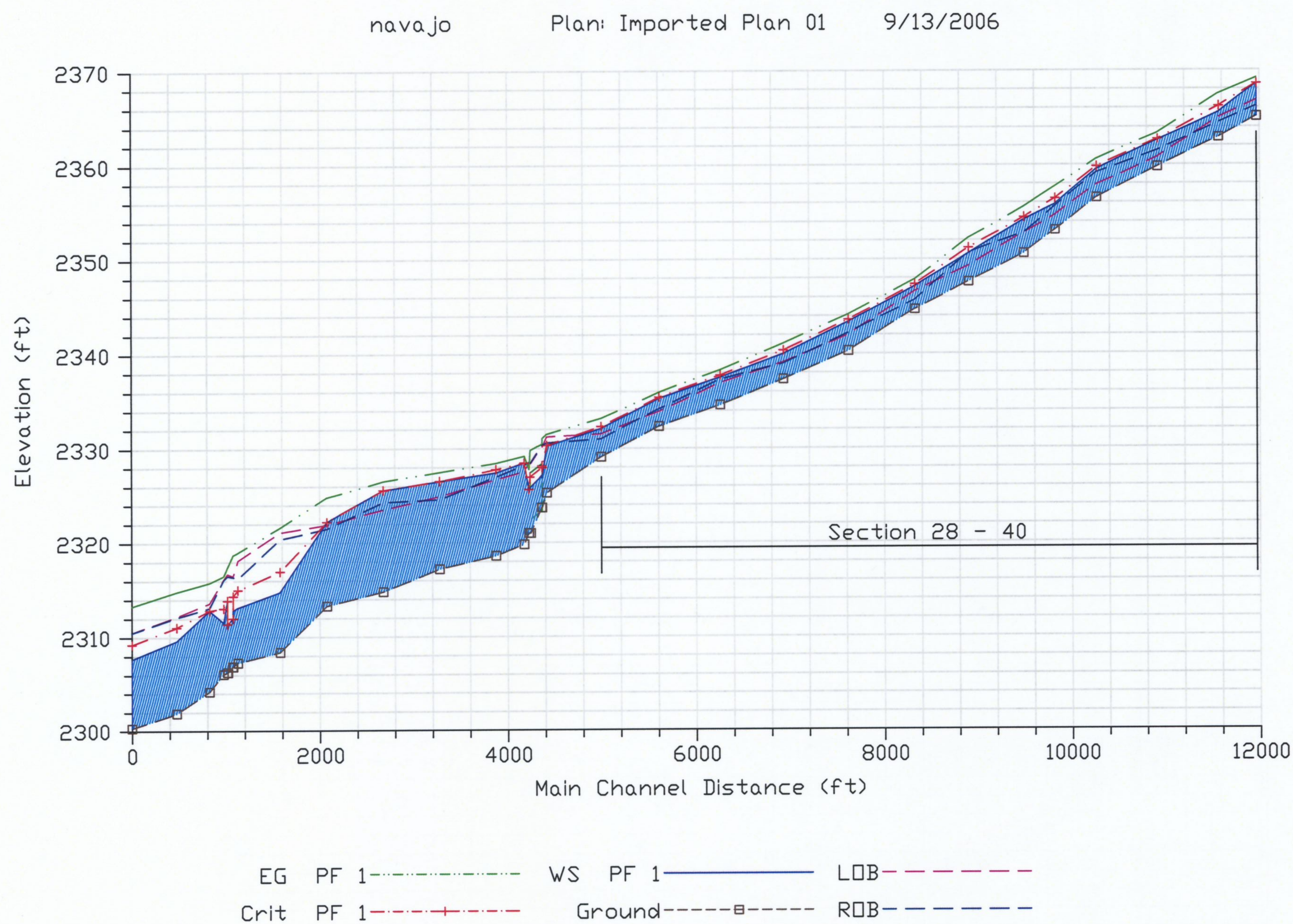


Figure 8.3. Results of HEC-RAS Evaluation

Section 9.
Selected Alternative

[This section would under the original contract summarize the results of the study and identify the chosen course of action. Conceptual plans would have been provided reflecting the selected concept, and the hydrology and hydraulic modeling updated accordingly. Because funding for construction is no longer available, the work under this section was terminated prior to its completion.

Prior to termination, work was started on a set of preliminary plans for a 100-year storm drain. Progress prints are included at the end of the appendices for future reference]