DRAFT

10% Design Report

OUTER LOOP DRAINAGE SYSTEM

The Desert Greenbelt

Presented To

CITY OF SCOTTSDALE

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

Registered Professional Engineer (Civil)
CERTIFICATE NO. 30490
MARTIN P. BRESSOR
Date Signed 11-25-97
ARIZONA, U.S.A.

PENTACORE
ARIZONA
Civil Engineering • Construction Administration
Land Surveying • Planning • ADA Consulting

November 1997

Jeff Riddle

RIDER HUNT CHARTWELL
Phoenix

10% DESIGN
VALUE ANALYSIS/PARTNERING WORKSHOP AGENDA
PIMA CHANNEL & MILLER ROAD BASIN PROJECTS
CITY OF SCOTTSDALE, ARIZONA

December 16 & 17, 1997

Location:

Scottsdale Stadium
3rd Floor Press Box Conference Room
Indian School & Osborn Roads
Scottsdale, AZ

Facilitator:

Rider Hunt Chartwell
8283 Hayden Road, Suite 200
Scottsdale, AZ 85258
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December 16, 1997

8:00 a.m. **INFORMATION PHASE**
INTRODUCTION TO WORKSHOP
(by VA Team Leader, John Pucetas, RA, CVS)

Welcome & Opening Remarks
Team Member Introductions
VA Process, Workshop Organization & Agenda
Objectives of Workshop

8:30 **PROJECT BRIEFING**
(by Stantec & Pentacore, City of Scottsdale)

Project Goals
Design Overview & 10% Review Comments
Project Budget Review, Cost Model

10:00 **FORCE FIELD ANALYSIS**

Best Project Features
Weakest Project Features

11:00 **FUNCTION ANALYSIS PHASE**

Briefing on Function Analysis
Function Analysis Questions
Function Identification & Classification
Function - Cost – Relationship (Cost Distribution Graph)
Function Analysis System Technique (FAST) diagramming

12:00 p.m. **LUNCH**

1:00 **CREATIVE PHASE**

Brainstorm Large Variety of Ideas
Generate Ideas for Basic Function
Think of Ideal Solutions
Modify and Combine Ideas
No Judgment

3:00 **ANALYSIS PHASE (VA Teams)**

List Idea Advantages and Disadvantages
Establish Evaluation Criteria
Evaluate Ideas By Comparison (10 High, 1 Low)
Rank Ideas for Further Investigation
Weighted Evaluation (as appropriate)

5:00 **ADJOURN**

December 17, 1997

8:00 a.m. **REVIEW STATUS AND PROGRESS OF TEAMS**

8:30 **SELECT MOST PROMISING IDEAS FOR DEVELOPMENT (VA Teams)**

9:00 **PROPOSAL DEVELOPMENT PHASE (VA Teams)**

Review of Proposal Forms and Final Products
Team Member Proposal Development Assignments
Prepare Design Alternatives

12:00 p.m. **LUNCH**

1:00 **PROPOSAL DEVELOPMENT - Continued (VA Teams)**

Written Proposals
Cost Estimate of Alternatives
Sketches of Alternatives
Life Cycle/Energy Impacts of Alternatives

3:00 **RECOMMENDATION PHASE (Individual VE Teams)**

Finalize Written Proposals (Present, Proposed, Discussion)
Finalize Sketches of Present & Proposed Design
Update Cost Information
Complete Value Summary Sheets

4:00 **IMPLEMENTATION PLAN**

Process for Accepting/Rejecting Recommendations
Implementation Tracking Log
Develop Implementation Schedule of Events

4:30 **WORKSHOP "POST MORTEM"**
CLOSING REMARKS

5:00 **ADJOURN**

DRAFT

10% Design Report

OUTER LOOP
DRAINAGE SYSTEM

Presented To:

CITY OF SCOTTSDALE

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

By:

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APPENDIX 4 - HYDRAULICS
APPENDIX 5 - SEDIMENTATION TRANSPORT AND YIELD
APPENDIX 6 - QUANTITIES AND COST ESTIMATES

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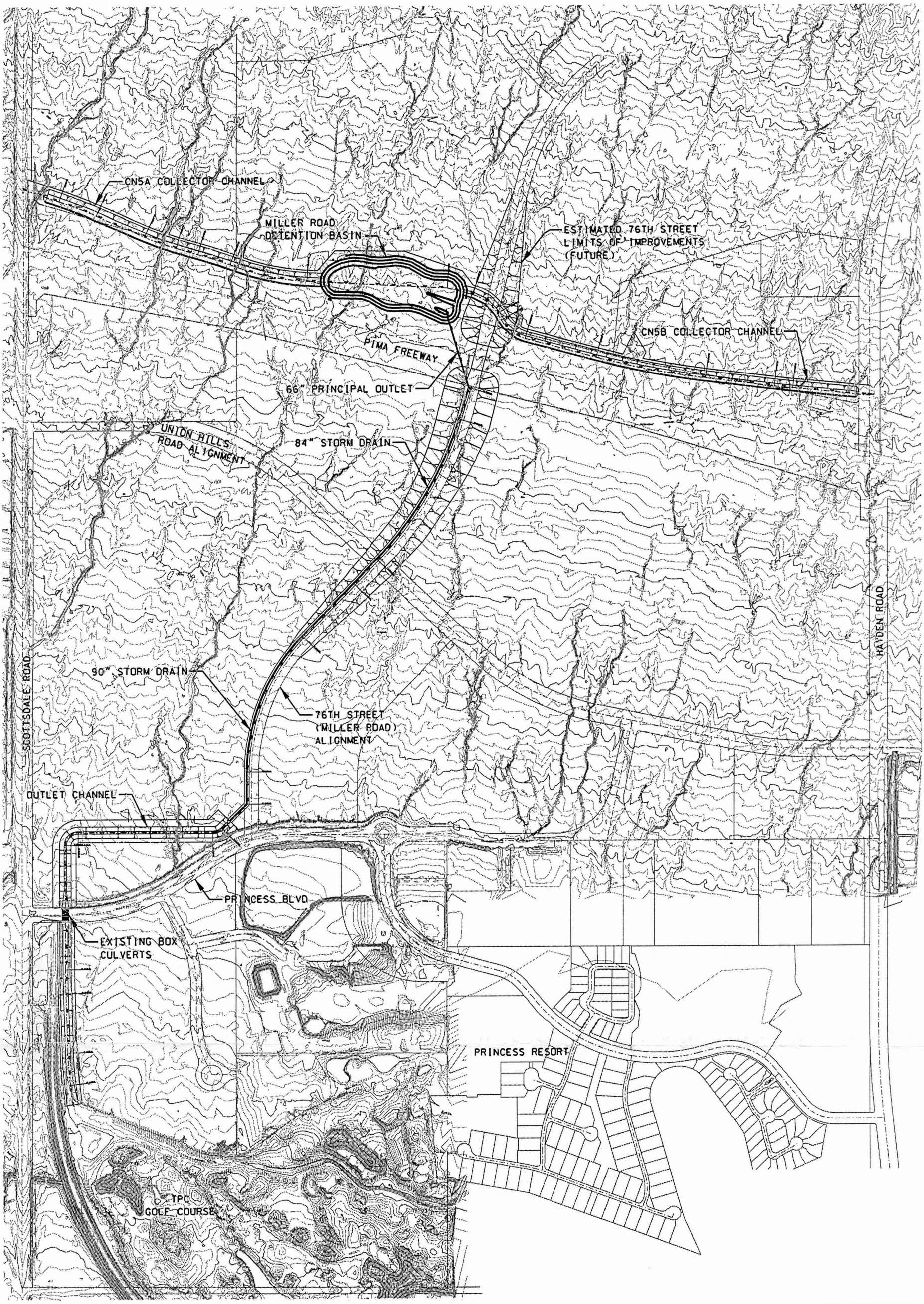
Date Signed 11-25-97
ARIZONA, U.S.A.

PROJECT DESCRIPTION

The Outer Loop Drainage System (OLD) is a component of the Scottsdale Desert Greenbelt. The primary purpose of the OLD system is to collect stormwater runoff between Pima Road and Scottsdale Road, north of Princess Boulevard, and convey it to the Bureau of Reclamation's regional detention facilities located at the Tournament Players Club of Scottsdale (TPC). The proposed Arizona Department of Transportation (ADOT) Pima Freeway Section 9A will intersect the project and design of the OLD system will be coordinated with ADOT's design of the Pima Freeway. Figure 1 presents a site plan of the project area and Appendix 1 contains preliminary plan and profile sheets of the proposed project.

The primary components of the project consist of collector channels, a detention basin, a storm drain outlet and an outlet channel. The collector channels will parallel the north side of the Pima Freeway and will be used to intercept cross drainage and convey the runoff to the detention basin. The collector channels will be lined with soil cement on the south side adjacent to the freeway and will have an unlined invert and north side slope. The detention basin located north of the Pima Freeway has two synonymous names, 38R-2 and Miller Road. The basin collects the runoff intercepted and conveyed by the collector channels. The proposed principal outlet of the basin consists of a 66-inch reinforced concrete restrictor pipe. The 66" restrictor pipe under-crosses the Pima Freeway where it increases to an 84-inch cast-in-place concrete pipe at the first junction south of the freeway. The remaining length of the storm drain system coincides with the 76th Street alignment heading south. A 90-inch cast-in-place concrete pipe replaces the 84-inch pipe at the intersection of Union Hills Road and 76th Street where an inlet to collect local runoff is planned for future development. The 90-inch storm drain conveys stormwater to an area north of Princess Boulevard and daylights into an open, unlined outlet channel north of Princess Boulevard. The outlet channel will convey the stormwater westerly towards Scottsdale Road and then southerly to Princess Boulevard where an existing six cell 6-foot wide by 3-foot high box culvert currently under-crosses Princess Boulevard. The continuation of the outlet channel from Princess Boulevard to the TPC is lined with soil cement. The stormwater will then be discharged at this point into the regional detention facility at the TPC. Discharge into the TPC will be coordinated with the City and its drainage engineer responsible for the TPC golf course. Cross-sections of the major system facilities are provided in Appendix 1.

Coordination with other agencies and groups will be a critical component of the design of this system. As already mentioned, the design will be closely coordinated with ADOT and their design engineer for the Pima Freeway. Additionally, Arizona State Land Department (ASLD) is the majority land owner within the project site area where the Core North parcel located north of the Pima Freeway and the Core South parcel located south of the Pima Freeway and north of Princess Boulevard are the two principal land parcels consisting of approximately 85% of the right-of-way requirements for the OLD project. Coordination will also be required with the Bureau of Reclamation and the Corps of Engineers as well as with various departments within the City of Scottsdale (City) and the Flood Control District of Maricopa County (FCD). Lastly, coordination with private landowners such as the Princess Resort, the TPC, Corrigan-Marley and private utility companies will also be required during the design of the project.



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CITY OF SCOTTSDALE
OUTER LOOP DRAINAGE SYSTEM
OVERALL SITE PLAN

FIGURE 1

PROJECT HISTORY AND DESIGN CONSTRAINTS

The Outer Loop Drainage System originally began as a component of the Pima Road Drainage System that was part of the City's Desert Greenbelt Project under a design contract with Greiner, Inc. As part of the Pima Road System, the OLD system in previous engineering documents is referred to as the 38R-2 Detention Basin System. Stantech Consulting Incorporated (Stantech) was awarded a contract by the City of Scottsdale to continue the design of the Pima Road System, and as part of that contract, Pentacore Arizona has been retained as a subconsultant to Stantech responsible for the development of the OLD system. Pentacore's scope of work for the OLD system essentially consists of advancing the OLD system from Greiner's 30% design to a final set of construction documents.

Pentacore's scope of work and the design parameters presented in this report are considerably different from the original scope of work. While conducting the hydrology, hydraulic and sedimentation analyses as part of the original scope, a number of improvements and cost savings to the system as a whole were discovered. As a result, many of the system components have changed from the 30% Greiner plans. Additionally, Greiner had prepared a cost study report for the 38R-2 system to evaluate various alternative system configurations different from those provided on the 30% plans for potential cost savings. The contents of the cost study report have been considered in the advancement of the design and preparation of this report.

The design parameters and project constraints presented herein are a result of a 10% level of conceptual design for many of the system components. It is expected that the efficiency of some of these components may be improved as the design proceeds. As the system continues to develop, many of the engineering models will require revision to ensure the system meets its objectives. The models and engineering revisions will be provided to qualified parties for review and comment throughout the duration of the project.

Collector Channels

The OLD system contains two collector channels that parallel the Pima Freeway. One channel, CN5A, runs from Scottsdale Road to the Miller Road Basin while the other channel, CN5B, runs from Hayden Road to the Miller Road Basin. As part of the Greiner 30% design plans, these collector channels were designed to be completely lined with soil cement. The cost study report investigated several other alternative channel configurations for the collector channels including unlined, riprap, and concrete linings, alone and in combinations. The Greiner cost study report evaluated costs, but did not consider the hydraulic feasibility of unlined channels in relation to the geometry presented in the 30% plans. However, because of the relatively flat terrain parallel to the freeway, unlined channels were considered to be a feasible alternative and the configuration of the collector channels was adjusted from the 30% plans to the configuration recommended in the cost study report.

While completely unlined channels proved to be the most cost effective configuration, coordination with ADOT required that the side slopes of the collector channels adjacent to the freeway be lined with an improved material. The most cost effective lining material for these channels was determined to be soil cement from the Greiner cost study report. The on-site soils have been determined to be good for soil cement applications. The well graded sandy material, absent of a large percentage of fine material, is well suited for soil cement. The site is undeveloped

with large open spaces, and each of the two collector channels is capable of providing long haul runs which is ideal for soil cement construction.

Pentacore analyzed the cost effectiveness of several channel cross-sections. The proposed cross-section for the two collector channels is shown in Appendix 1. A 4H:1V side slope is used for the unlined, northern slope of the channel. This slope is relatively flat enough to minimize side slope erosion and rilling as a result of water falling on the side slope, while steep enough to minimize the limits of right-of-way acquisition. Surficial runoff into the channel will be prevented with a small berm, grade ditch and controlled inlets into the collector channels. A 1H:1V side slope is used for the southern soil cement lined side slope. Pentacore compared the costs of a thinner, 4H:1V soil cement paved side slope and determined the cost was only marginally less expensive per linear foot of channel than the 1H:1V side slope section. Because of the expense of additional right-of-way, cost savings realized by using less soil cement material are offset, resulting in only a nominal cost savings. The City and Pentacore agreed the narrower channel section was more effective for this application. The cost comparison of the two sections is provided in Appendix 6.

Miller Road Basin

The detention basin of the OLD system is located at the northwest corner of 76th Street and the Pima Freeway. The detention basin has traditionally been referred to as basin 38R-2, which was the basin designation for most of the hydrology models developed for the area. The basin has recently been referred to as the Miller Road Basin, particularly by ADOT and its consultants, because of its proximity to 76th Street, which is also Miller Road. Because of the coordination that is required with ADOT, and to remain consistent with the nomenclature for other basins in the Desert Greenbelt Project, the Miller Road Basin will be used throughout this report.

The purpose of the basin is to collect and detain the stormwater runoff resulting from the 100-year, 24-hour storm event. In addition to the volume of runoff, the basin is also required to provide storage volume for sediment yield. The recommended volume of sediment yield to be stored is 5 times the annual sediment yield plus the sediment yield from the 100-year storm event. This is a common storage volume provision, and is consistent with the storage volume recommendations for other systems of the Desert Greenbelt Project. A minimum additional 1-foot freeboard is also required of the basin. The highest containment elevation of the basin used for design is 1607 which occurs at the southwestern corner of the basin. The maximum detained high water elevation within the detention basin, including the necessary dead storage volume for sediment yield, will be restricted to 1606.

Originally, the design of the basin was to be a continuation and further development of the layout and configuration as shown on the Greiner 30% plans. However, preliminary hydrology and hydraulic analyses revealed that the basin was oversized for its intended purpose. The basin sizing was adjusted as a result of these analyses described in the hydrology and hydraulic sections of this report. The areal extent of the basin was elongated from the 30% plans to take advantage of available right-of-way adjacent to the Pima Freeway. As a result, the basin configuration will be considerably different from that presented in the 30% plan.

Outlet Channel

The outlet channel has undergone the most significant changes of any component of the OLD system. Originally designed to extend from the principal outlet of the Miller Road Basin to the TPC,

the outlet channel now extends from the intersection of 76th Street (Miller Road) and Princess Boulevard to the TPC. The outlet channel from the basin to the intersection has been replaced with a storm drain to be described in the following section.

The outlet channel, originally designed to be completely lined with soil cement, has been revised to an unlined channel with lining improvements at specific locations. The existing six cell 6-foot wide by 3-foot high box culverts under Princess Boulevard are considered part of the outlet channel. The location of the improvements are shown on the plan and profile sheets in Appendix 1.

The design capacity of the outlet channel was limited to 930 cfs which was the restricted flowrate agreed upon by the City and ASLD in previous discussions. The flowrate of 930 cfs was determined to be the hydraulic capacity of the existing box culverts by an independent consultant for ASLD without overtopping Princess Boulevard. The hydraulic capacity of the culverts was determined without the proposed OLD improvements in place. It is Pentacore's opinion that the OLD improvements will significantly increase the hydraulic capacity of the existing culverts. The hydraulic calculations for the basis of this opinion are presented later in this report.

The outlet channel begins just north of the intersection of 76th Street and Princess Boulevard. At this point, the channel receives flow from the basin outlet and storm drain. An existing unlined channel is located parallel, and north of Princess which collects and conveys flow from the area east of 76th Street. The purpose of this unlined channel is to convey runoff to the existing box culverts previously described rather than routing the flow through the Princess Hotel Resort. There are a number of outstanding issues pertaining to the future accommodations of this existing channel that have yet to be resolved. The ultimate design and final configuration of the OLD outlet channel will depend on the future functionality of this existing channel.

The outlet channel will also intercept surface runoff from the Core South area west of 76th Street to Scottsdale Road, and south of the Pima Freeway. Like the collector channels, surficial flow directly into the channel will be prevented. Improved inlets into the channel will be provided where the channel intercepts existing washes.

The outlet channel discharges just upstream of the TPC golf course. The discharge point is consistent with the point shown on the Greiner 30% plans. The impacts to the TPC have yet to be evaluated and coordination with the TPC may be required as the design proceeds.

Principal Outlet and Storm Drain

The principal outlet from the Miller Road Basin is also changed from the 30% design plans. Originally proposed as a 10-foot wide by 6-foot high box culvert, the principal outlet has been revised to a single concrete culvert. The revised principal outlet is extended south under the proposed Pima Freeway where it connects to a larger storm drain. This storm drain then continues south along 76th Street to the outlet channel at the intersection of 76th Street and Princess Boulevard. For purposes of clarification, the "principal outlet" term will be used to describe the 66" RCP restrictor pipe, while the "storm drain" term will be used to describe the cast-in-place piping from the junction structure south of the Pima Freeway to the outlet channel. The piping from the Miller Road Basin to the outlet channel will be continuous, but local flow from the Core South area introduced to the piping system at Union Hills Road will result in an increased pipe size at that location. During the engineering evaluation of the proposed outlet system, Pentacore found that a storm drain could be used more effectively to drain the detention basin and at a cost savings to the

City. The comparative cost analyses between the open channel system and the storm drain piping system are provided on Table A6-1 in Appendix 6. The proposed layout of the system is presented on the plan and profile sheets in Appendix 1.

The design constraints of the principal outlet and storm drain piping system are governed by the City of Scottsdale Design Standards and agreements between the City and ASLD that were established based on drainage engineering analyses that were completed for the project area by others. The agreed upon discharge flowrate from the detention basin is limited to 720 cfs. The purpose of restricting the flow to 720 cfs was to ensure the maximum discharge rate at the existing box culverts at Princess Boulevard was less than 930 cfs, which was determined by the other drainage analyses to be the maximum flowrate that could be conveyed through the existing box culverts without overtopping Princess Boulevard. As will be discuss later in this report, the hydraulic capacity of the existing box culverts is believed to be greater than 930 cfs with the improvements of the OLD system in place. Additionally, the discharge rate through the principal outlet must be sufficiently large enough to ensure the Miller Road Basin will drain within 36 hours.

The proposed alignment of the outlet system, from the detention basin to the outlet channel at Princess Boulevard, has also changed considerably from the 30% design plans. Where the open channel presented in the 30% design plans was required to be offset from the 76th Street alignment, the storm drain alternative allows a dual purpose use of the 76th Street right-of-way. The proposed piping alignment is shown in the Appendix 1.

Overall System Design

The overall system design has been greatly changed from the system originally proposed in the 30% plans prepared by Greiner. In addition to the modifications to the system components just described, the engineering analyses were also revised to include the Core South area. The Core South area significantly impacts the OLD system and these impacts have been considered in the proposed improvements.

HYDROLOGY

Assumptions and Constraints

There exists an extensive history behind the hydrology of the OLD system. The hydrology for the area was originally developed by the Bureau of Reclamation for the design of the Reach 11 Dikes of the Hayden/Rhodes Aqueduct for the Central Arizona Project. In 1987, Simons, Li and Associates (SLA) performed a more site specific hydrology analysis for this area for the State Trust Lands as part of the Outer Loop Highway design from 32nd Street to Pima Road[1]. This hydrology was later incorporated in 1988 into the General Drainage Plan for North Scottsdale prepared by Water Resources and Associates (WRA)[2]. In 1992, WRA further developed the hydrology for the area in The Upper Indian Bend Wash Regional Drainage and Flood Control Plan[3]. In 1994, as part of their development work for the Core North State Parcel, Gilbertson Associates revised the hydrology prepared by WRA by incorporating drainage improvements within Core North[4]. Lastly, Robert Ward[5] conducted a hydrology study in 1995 for the City of Scottsdale examining development conditions for the year 1996 and the year 2002. Each of these models were developed for the Core North area, and the 2002 hydrology models were used as the basis for this report and future design of the OLD system. The hydrology is established for the 100-year/24-hour storm frequency event.

The final hydrology models established by Robert Ward for Core North have been reviewed, agreed to, and accepted by the various agencies interested in the drainage of north Scottsdale; namely the City, ASLD, and FCD. This hydrology has been established using the City's land-use mapping and coordinates the most recent conditions and planned development by various developers in the area. These models coordinate all information to develop the most likely post-development scenario for Core North in the year 2002.

The Core North hydrology was merged with Core South hydrology, established by Pentacore based on the drainage analysis prepared by SLA[6](1987), to develop the overall hydrology model that is used for the design of the OLD system. The Core South hydrology is described later in this report. The overall post-development hydrology used by Pentacore for the design of the OLD system melds the following:

- Core North - Robert Ward (1995)
- Core South - Pentacore (1997)
- OLD system improvements - Pentacore (1997)

In addition to the post-development conditions, Pentacore also analyzed the hydrology for existing conditions in Core South. This hydrology was used to verify the agreed to maximum discharge rate from the Miller Road Basin (720 cfs) and to verify the impacts at the Princess Boulevard culverts.

Several figures are provided in the appendix to depict the hydrology modeling described. Figure A3-1 reflects the Core South delineation used to analyze existing conditions. Figure A3-2 depicts the post-development subbasin delineation and routing for Core North and Core South. Figure A3-3 reflects the City's mapping for parcels and zoned land-use.

Core South Post-Development Conditions

The hydrology for Core South developed by Pentacore was established using the City of Scottsdale's land-use zoning and the routing configuration reflected by SLA (June 1987) in accordance with the City's General Plan. The final hydrology effectively coordinates the OLD system improvements with the post-development conditions for Core North and Core South.

The Core South development area is slightly more than 0.5 square miles which includes the area south of the Pima Freeway, north of Princess Boulevard, west of Hayden Road, and east of Scottsdale Road. This area was omitted from the Ward 1995 model for Core North and from the Greiner model for the 38R-2 Detention Basin. The hydrology modeling developed for the OLD system includes the Core South area and was developed using the following assumptions:

a) the post-development subbasins were delineated in accordance with the conceptual drainage report prepared by SLA as part of their drainage analysis of state trust lands;
    - subbasin areas were determined from the digital mapping;
    - parameters used for the intra-basin kinematic wave routing were obtained from the digital terrain model (DTM);
    - roughness coefficients for the kinematic wave routing were consistent with the values used for the Core North area where a Manning's value of 0.045 was used for overland flow and a value of 0.035 was used for routing within naturally formed washes;

b) the directional routing between the subbasins is consistent with the routing schematic also provided by the SLA conceptual drainage plan;

c) an SCS curve number of 77 was used for all Core South subbasins based on the City's Design Standards Manual[7] (Figure 2.2-19) and the following:

- the SCS soil survey map for the area indicates the area is predominately classified in the "B" hydrologic soil group;
- the area is a western desert urban area with natural desert landscaping;

d) the subbasin hydrology characteristics are shown in Table A3-1 in the appendix;

e) the channel configuration used for routing between the Core South subbasins assumes the following:

- unlined, trapezoidal channels with a 10-foot bottom invert and 2H:1V side slopes;
- Manning's roughness coefficient of 0.025 used for sandy, unlined channels;
- the slope of the channels is estimated to be consistent with the natural grade slope as determined from the DTM for the area;
- the channel depths were adjusted to ensure complete flow containment within the channels.

Miller Road Basin

The configuration of the Miller Road Basin has undergone significant changes from the parameters used in the hydrology models by Gilbertson, Ward and Greiner. The areal extent of the basin has been altered from a "square" format to a "rectangular" format. As previously discussed, the basin no longer uses a 10-foot wide by 6-foot high box culvert as a principal outlet, but rather a 66-inch concrete restrictor pipe. The 66-inch pipe connects to a larger storm drain which is routed to the intersection of 76th Street and Union Hills Road where the stormwater runoff from the local area is combined with the basin discharge flow.

The elevation stage, volumetric basin storage and outlet discharge for the basin required revisions. Stage-storage-discharge curves for the revised basin were developed by Pentacore as a result of a hydraulic analysis for the revised outlet system. The volume data was obtained from the digital model of the detention basin. The hydraulic parameters presented in Table 1 and Figure 2 below were developed using Haestad's StormCAD software program for storm drains.

The basin spillway elevation is 1607. The basin spillway length (600 feet), spillway coefficient of discharge (2.9) and head exponent (1.5) are consistent with the values used for the other models.

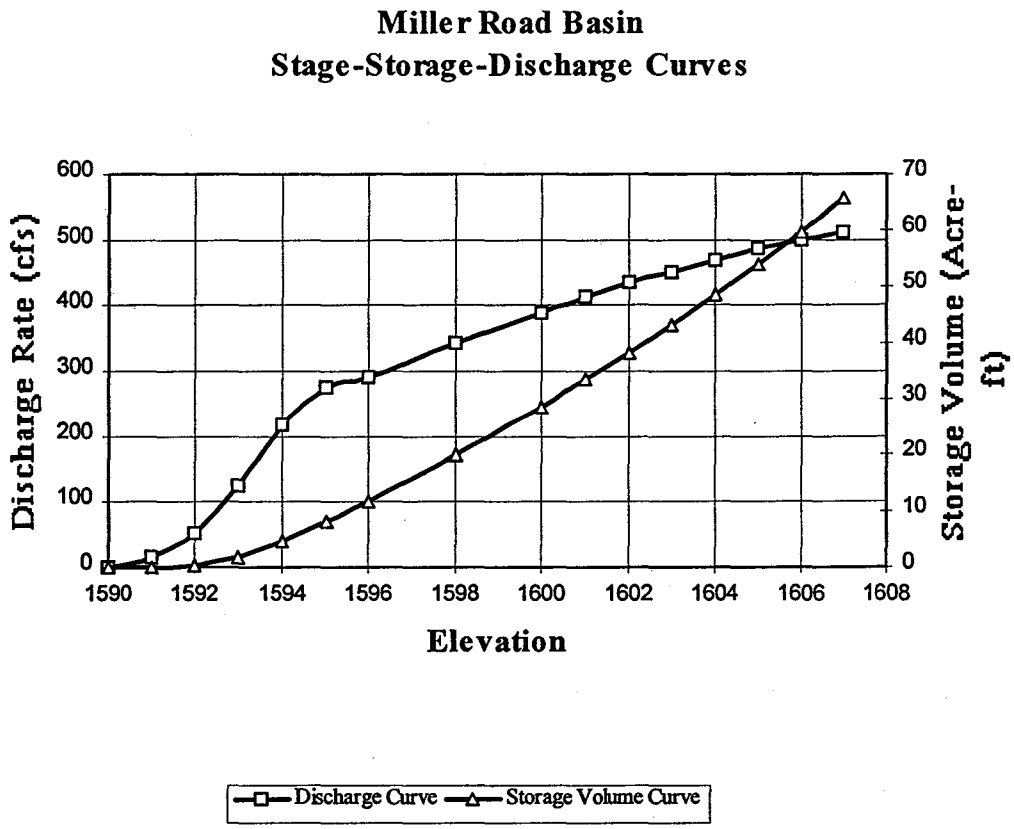

Figure 2 - Miller Road Basin Stage-Storage-Discharge Relationships

Elevation	Volume (acre-ft)	Q (cfs)
1590	0.0	0
1591	0.1	16
1592	0.4	52
1593	1.9	127
1594	4.8	216
1595	8.2	274
1596	11.9	291
1598	19.9	342
1600	28.7	390
1601	33.4	412
1602	38.3	435
1603	43.3	451
1604	48.6	468
1605	54.0	486
1606	59.7	500
1607	65.6	512

Table 1 - Miller Road Basin Stage Storage Volume

HEC-1 Modeling

Three distinct hydrology models have been developed by Pentacore for the OLD system: OLD.12i, OLD.13i, and OLD.14i. Models 12 and 13 were developed for post-development conditions whereas model 14 was developed for existing conditions in Core South. Model 12 was developed for post-development conditions in according to the configuration previously described and as depicted on Figure A3-2. Model 13 modifies Model 12 where drainage from the Core South area, east of the 76th Street alignment, is routed directly southward into the CAP regional detention facility. Model 14 represents existing conditions for the Core South area as depicted on Figure A3-1. The HEC-1 models are presented in the appendix. This model was developed for the 100-year/6-hour storm event.

The two post-development models presented in this study are based on the assumptions and approximations previously described. In addition to those parameters, each model has the following in common:

1. The 84-inch storm drain which is fed by the 66" principal outlet combines with local storm water runoff from the local area north of Union Hills Road (CS6 and CS10) at Union Hills Road. A 90-inch concrete storm drain continues from the junction to the outlet channel near Princess Boulevard.
2. Most of the stormwater runoff from subbasins west of 76th Street (CS4, CS3, CS2, CS1, CS1A, and CS2A) are routed south, through the Princess Resort.
3. Runoff from subbasins CS7 and CS8 is routed into the outlet channel.
4. Runoff from subbasins CS11 and CS 12 is combined with the flow in the outlet channel.

Miller Road Basin Optimization

An initial evaluation of the existing hydrology model used by Greiner for development of the 30% plans revealed that the detention basin was considerably oversized for the stormwater volume it was required to detain. While a reason for the basin oversize was not specifically determined, two factors impacted the performance of that system: the principal outlet and the open channel outlet.

The principal outlet used for that system was a 6-foot by 10-foot box culvert that had been used consistently throughout years for the hydrology assessment of the area. The hydraulic capacity of an outlet of this size may be considerably greater than 720 cfs given the appropriate storage head in the basin and outfall slope of the culvert. The maximum permissible discharge rate for the principal outlet is 720 cfs per agreements between the City and ASLD. Additionally, the depth of the detention basin and the discharge rate may affect the configuration of the downstream outlet channel. The hydraulic performance of open channels has been a significant concern throughout the development of the Desert Greenbelt project and the outlet of this system is in an area where the existing natural grade is steep, at approximately 1%. By lowering the detention basin invert, the slope of the outlet channel could be reduced, thereby reducing the high velocity and improving the hydraulics in the open channel outlet.

Pentacore explored the advantages of reducing the size of the principal outlet to afford more storage depth within the detention basin without exceeding a discharge rate of 720 cfs. To do so, 4 basin configurations were analyzed in conjunction with 3 principal outlet culvert sizes. The 4 basins included the existing basin configuration (approximate bottom invert 450 feet by 450 feet), and square basin configurations using bottom basin extents 400 feet, 300 feet and 200 feet wide. Each basin configuration used 4H:1V side slopes. For each basin configuration, three principal outlet sizes were analyzed: 60-inch, 72-inch and 90-inch concrete culverts. The volumes for each of the basins were determined from the DTM for the project. The basin outlet characteristics were determined from a hydraulic analysis similar to the analysis previously described for the Miller Road Basin final configuration.

Many HEC-1 models were developed for each feasible configuration of the basin/outlet size. Of the models, eight were considered to be hydraulically feasible. Feasibility was determined to be configurations that had the ability to store and convey stormwater without overtopping the basin, complete gravity drain within 36 hours and without exceeding 720 cfs discharge rate. Once the feasible basin and outlet sizes were determined, Pentacore proceeded to assess the cost advantages of using a storm drain outlet piping system in lieu of the open channel outlet system. As will be described later in this report, there were considerable advantages to using a storm drain outlet.. Of the feasible alternatives, the optimal configuration resulted in a 300-foot basin invert with an outlet culvert diameter size ranging between 60 and 72 inches.

As discussed in the previous section, the Miller Road Basin has been elongated to take advantage of existing right-of-way adjacent to the Pima Freeway. The characteristics of the rectangular basin are similar to those for the optimal square configuration. The basin stage-storage-discharge relationship presented in the previous section is based on this final configuration as shown in Appendix 1 and subsequently used for all hydrology models described herein.

Core South Routing

The pertinent issue that remains unresolved regarding the Core South development area pertains to the ultimate drainage plan for the area east of the 76th Street alignment. Figure A3-2 shows the anticipated post-development subbasin delineation and routing scheme for Core South as presented in the SLA report and in accordance with the general plan for the area. The issue that remains in question pertains to the drainage conveyance and routing of subbasins CS5, CS4A, and CS3A. At present, the runoff from these subbasins is collected by an unlined channel adjacent to Princess Boulevard, and conveyed west, parallel to Princess, to the existing box culverts located near Scottsdale Road. This is consistent with the general plan developed for North Scottsdale. However, parties interested in the development of the OLD system question whether this runoff should be conveyed to the west, to the existing box culverts, or whether the drainage plan should be revised to convey the runoff from these subbasins directly south into the Princess Hotel Resort. Models were developed for the two conditions:

- OLD.12i - General Plan routing
- OLD.13i - Revised Core South routing

Hydrology Results and Design Values

The results from the OLD.14i hydrology model indicate the peak runoff flowrate at the Princess Boulevard culverts from this area, under existing conditions is 418 cfs. As Pentacore can best

determine, it appears this peak runoff rate is substantially greater than the value determined by others and the value used as the bases to establish the agreements between the City and ASLD. The peak runoff value from the Core South area reported by SLA[1] for undeveloped conditions from Core South is 66 cfs. The peak runoff rate for developed conditions was 209 cfs. As a result of the hydrology for existing conditions prepared by Pentacore, the peak discharge rate from the detention basin was limited to 512 cfs.

The results from the two post-development hydrology models prepared for the OLD system are presented in Appendix 3. Table A3-3 presents a comparative summary of the results. At this time, the results and runoff flowrates from the OLD.12i model have been used for the design of the OLD system. This model replicates the approved general drainage plan for North Scottsdale and represents the best estimated post-development conditions for the Core North and Core South areas. The drainage values as a result of this hydrology model do not meet the criteria established by the agreements between the City and ASLD. While the maximum discharge rate from the Miller Road Basin ("38R-2") is 491 cfs, less than 512 cfs; the peak flow at the existing box culverts at Princess Boulevard ("PRNCSS") is 1361 cfs, 46-percent greater than 930 cfs. The OLD.13i model improves the situation at the Princess Boulevard culverts, reducing the peak flowrate from 1361 cfs to 1048 cfs, but the flowrate remains 13-percent greater than 930 cfs. At this time, the issues of post-development retention and the impacts of development within the Core South parcel remain uncertain and must be addressed by the agencies with interests in the project. The peak discharge rate used for the design of the outlet channel remained at 930 cfs.

The high water elevation detained in the Miller Road Basin is 1605.38. Since the maximum water surface elevation permitted in the basin is 1606.0, the basin will be raised in the subsequent design phases to provide 1-foot of freeboard and ensure the basin is optimally designed.

HYDRAULICS

Hydraulic analyses were conducted for various components of the OLD system including the collector channels, the basin principal outlet, the storm drain, the outlet channel and the existing culverts at Princess Boulevard. As with the hydrology, the hydraulic analysis that has been conducted for the OLD system is considerably different from the original hydraulic analysis that was expected for this project because of the changes made to the components of the system. As a result, the hydraulics presented herein are on par with a 10% conceptual level of design. Pentacore will consistently update the hydraulics of the system as the design proceeds to completion and make necessary adjustments throughout the design process to ensure the efficiency of the system and cost benefit to the City and FCD is optimized.

The hydraulic parameters for the OLD system are established from the results of the hydrology, presented above, based on the peak flow discharge rates associated with the 100-year/24-hour storm frequency event. These peak flowrates are obtained from the results of the HEC-1 OLD.12i hydrology model. The peak flowrates presented in the table below will be used for the design of the channels of the OLD system. As issues surrounding the design of the OLD system are resolved, the hydraulics may undergo change as well as the hydrology. The peak design flowrates for the channels of the system are:

Channel	Design Flowrate (cfs)
Outlet Channel	930
CN5A Collector Channel	1086
CN5B Collector Channel	695

Table 2 - Design Flowrates for the OLD System

Principal Outlet – Miller Road Basin

The principal outlet for the Miller Road Basin has undergone the most significant change of the components of the OLD system. The principal outlet is now proposed to be a 66-inch concrete restrictor pipe, revised from the 10-foot by 6-foot high box culvert proposed in the 30% design plans. The purpose for adjusting the principal outlet was to improve the effectiveness of the detention basin while reducing the potential cost of the OLD system by replacing the open channel outlet with a piped outlet.

The hydraulic parameters of the principal outlet were analyzed for a number of principal outlet sizes. As described in the hydrology section of this report, a number of basin configurations and principal outlet sizes were analyzed for potential cost savings for the OLD system. A 66-inch principal outlet was determined to be the optimal outlet size, in conjunction with the detention basin sizing, to provide cost savings while ensuring an acceptable level of hydraulic performance for the system.

The hydraulic calculations and assessment for the basin and the principal outlet were accomplished using Haestad's StormCAD modeling as presented in Table A4-1 in Appendix 4. The principal outlet system is modeled as a storm drain. The StormCAD model analyzes the storm sewer system hydraulics via a backwater analysis starting at the downstream discharge point and proceeding upstream. StormCAD performs a gradually varied flow analysis in free surface flow conditions and pressure flow computations under full flow conditions. The current storm drain system proposed will operate under pressure in the 66″ restrictor pipe and under gravity in the 84″ and 90″ sections. It was determined that a future collection point will be provided at the northwest corner of the 76th Street/Union Hills intersection, consistent with the Concept Drainage Plan for the Core South Parcel. This will require an increase in pipe size diameter from 84-inch to 90-inch at this junction.

The culvert is proposed to be a cast-in-place concrete pipe, and for this reason, a Manning's roughness coefficient of 0.015 was used for the outlet culvert. The slope of the culvert is similar to the existing grade as shown on the plan and profile sheets in the appendix.

Permissible Flow Velocities

Where the channels of the 30% plans were designed to be completely lined with soil cement, the channels of the OLD system have been revised for unlined channel inverts. While the channels are not completely unlined, the inverts are designed to be unlined except in specific areas where the hydraulics mandate lining to prevent scour and erosion. The FCD Design Manual[8] recommends that the velocity of flow within unlined, earthen channels be limited to 6 fps and the BOR recommends calculating the permissible flow velocity within an unlined channel using a procedure

based on the calculated unit tractive forces of the flow[9]. Calculations for permissible velocities within the channels of the OLD system are contained in Appendix 4. The targeted flow velocity within each channel for design is to be 4 fps. HEC-RAS models were developed for each collector channel and the hydraulic characteristics for each are presented in the appendix.

Flow rates for the collector channels were opened from the OLD.12i hydrology model. The Manning's roughness coefficients used for soil cement and natural soil material were 0.019 and 0.022, respectively. The values are consistent with recommendations provided in local design guidelines.

Outlet Channel

The most significant change to the outlet channel from the 30% design plans is its reduction in length. The channel no longer extends from the Miller Road Basin, but from the intersection of Princess boulevard and 76th Street. The channel is unlined from the intersection to the existing box culverts at Princess Road and soil cement lined from the culverts to the outlet at the TPC as seen on the plan and profile sheets provided in Appendix 1. The existing slope downstream from the existing box culverts prohibits the use of an unlined channel due to erosion and scour. The hydraulics for this channel were calculated using normal flow calculations discussed above.

Collector Channels

The collector channels have been revised from soil cement lined channels shown on the 30% plans to unlined channels with soil cement side slope protection adjacent to the Pima Freeway as previously described and shown on the plan and profile sheets in Appendix 1. The channel configurations were not expected to change significantly from the 30% design plans, and normal flow calculations were used to confirm the channel configurations were feasible for the change to unlined channels. Normal depth calculations based on the permissible velocity calculations were preliminarily used to determine the hydraulics of the collector channels. Subsequently, HEC-RAS models were developed for the two collector channels using the water surface elevation in the Miller Road Basin as a basis for backwater calculations. The results from the HEC-RAS modeling for the two collector channels are presented in Tables A4-2 and A4-3 in Appendix 4.

Existing Culverts – Princess Boulevard

The existing six cell, 6-foot wide by 3-foot high box culverts at Princess Boulevard limit the hydraulic capacity of the OLD system and may cause stormwater to backup, potentially overtopping the road. ASLD and the City have agreed that the OLD system should not deliver a peak discharge rate greater than the hydraulic capacity of the box culverts. On the basis of work performed by others, the hydraulic capacity of the box culverts was determined to be 930 cfs.

Pentacore has performed an independent hydraulic analysis to assess the hydraulic capacity of the box culverts with the OLD system improvements in place. Figures A4-1 and A4-2 illustrate the configuration of the existing culverts. The upstream invert elevation is 1545.44 as determined by a field survey. The elevation at which impounded water will begin to overtop Princess Boulevard is 1550.96. Therefore, the maximum upstream headwater depth for the culverts is approximately 5.5 feet. FHWA's HDS-5[10] (HY-8) manual and HEC-RAS were used to estimate the hydraulic capacity of the culverts.

Charts from HDS-5 used to estimate the hydraulic capacity of the culverts are provided in the appendix. The charts for inlet control were used. The first chart is based on a headwater depth of 5.4 feet and an approach velocity of approximately 15 fps. The approach velocity was determined from a HEC-RAS model of the upstream outlet channel. The total energy HW depth is 8.9 feet, resulting in a hydraulic capacity for the existing culverts that ranges between 1260 and 1440 cfs depending upon the chosen wingwall configuration constraint. The second chart represents a back analysis to determine the hydraulic parameters used to arrive at 930 cfs. Again depending on the chosen wingwall constraint, the headwater depth was determined to range between 4.8 and 5.7 feet. Since these charts are based on total energy head, the estimated capacity of the culverts at 930 cfs appears to be low given the OLD system improvements. With the OLD system improvements in place, it is Pentacore's opinion that the hydraulic capacity of the existing box culverts will be greater than 930 cfs.

Pentacore has also developed on number of HEC-RAS models using various techniques to estimate the capacity of the culverts. The results of those models are summarized in Table A4-4. The values reported in this table are based on mixed flow regimes. Under supercritical flow conditions, the hydraulic capacity was significantly greater. Referring to the summary table, the results using bridge techniques indicate the hydraulic capacity of the culverts is approximately 1250 cfs. Special culvert analyses were less, but still greater than 930 cfs.

The bridge calculation techniques and the results from supercritical flow indicate the capacity of these culverts is substantially greater than the current capacity used as a design constraint. Pentacore has performed hand calculations to substantiate the results from both HEC-RAS and HDS-5. Given the critical impact these culverts have on the overall design of the OLD system, it is Pentacore's recommendation that the design constraint governing the hydraulic capacity of the existing box culvert be increased from 930 cfs for future design considerations.

SEDIMENTATION YIELD

The sedimentation yield for the OLD system had not been calculated for the 30% design, but was considered by Greiner in the cost study report. Runoff sediment yield is used for the design of detention basins to estimate additional dead storage volume required to collect the sediment deposition. The dead storage volume for other Desert Greenbelt detention basins is based on the 5 times the mean annual sediment yield plus the sediment yield resulting from a 100-year storm frequency event. The recommended dead storage volume provided in the Greiner Cost Study Report was 3 acre-feet/square mile cited from the work performed by George V. Sabol Consulting Engineers (GVSCE) for the review of the Pima Road drainage system. Upon review of the GVSCE report, values were reported for three detention basins associated with the Pima Road section of the Scottsdale Desert Greenbelt Project. The recommend dead storage volume for these basins ranges between 1.4 and 3.4 acre/feet per square mile of contributing area. The Miller Road Basin has most often been compared to the Union Hills Basin of the Pima Road section with regard to the upstream contributing area where the estimated dead storage volume would be 1.4 acre-feet per square mile, approximately half the value reported in the Greiner report. Pentacore performed an independent analysis to estimate the sediment yield for comparison with other values reported.

Assumptions and Constraints

A number of methods may be used to quantify the volume of sediment yield of runoff from a contributing watershed. Methods commonly used include: 1) soil loss relationships based on the

characteristics and land use within the watershed such as the Uniform Soil Loss Equation and the Modified Uniform Soil Loss Equation; 2) average annual methods based on estimated sediment delivery such as the BOR method and the PSIAC method; and 3) sediment transport methods based on sediment transport equations such as the Zeller-Fullerton equations and Yang's Streampower relationships. Of these methods, the sediment transport methodology is commonly believed to provide the best estimates for sediment yield[11].

The hydrology and hydraulics from the Core North area were used as the basis of developing the sediment transport relationships. Hydrographs from the OLD.12i HEC-1 hydrology model were used for the analysis. The channel routing configurations used in the HEC-1 model were also used for the sediment analysis and hydraulic characteristics were developed based on normal flow.

The sediment characteristics used for this analysis are based on 17 near surface soil samples obtained by Greiner and sieve analyses performed on those samples by AGRA Earth and Environmental as part of their work on the Pima Road drainage system. Near surface soil samples were compared and the statistical gradation that resulted in a well-graded sand used for this analysis is provided in Table A5-1. Yang's Streampower relationship[12], was used to estimate sediment yield using the gradation provided in the appendix as well as the d_{50} size (1.1 mm) alone. The analysis based on the d_{50} grain size was performed for comparative purposes, only. Average annual methods typically based on the average sediment grain size can significantly underestimate the total sediment yield as will be discussed in the results.

Several assumptions were made to determine the final sediment yield values. First, the quantity of sediment transport is based on the carrying potential of the routing channel to the detention basin. The primary channels of subbasins CN5A and CN5B are used to convey sediment to the detention basin and therefore the carrying potential of those channels, with the geometric configurations used for the hydrology modeling, was the basis for this analysis. Secondly, an assumption was made that sediment is readily available to meet the transport capacity. While this may not necessarily be true for post-development conditions, it provides an upper bound for the sediment yield. Lastly, the transport analyses are based on routing hydrographs through the channels rather than using the peak flowrates to estimate sediment yield for various storm frequencies. Sediment transport analyses are often based on the peak flowrate for various storm frequencies and apply an empirical reduction factor to the results because of the difficulty associated with estimating the sediment yield for a hydrograph. However, using a hydrograph will produce more accurate results of the overall sediment yield.

Results

The results from the sediment yield analysis are provided in Table A5-2 for varying degrees of trap efficiency of wash load. Wash load represents the most significant contribution to the overall sediment yield. Depending on the trap efficiency of the basin, the dead storage volume provided for sediment yield may range between 1.2 and 4.7 acre-feet. The resulting mean annual yield for the Miller Road Basin is comparable to the value report by GVSCE for the Union Hills Road Basin when wash load is neglected.

Since the dead storage requirement is sensitive to the trap efficiency of the basin, Pentacore performed a separate analysis to estimate the trap efficiency for wash load. The results are provided in Table A5-3. Hydrographs from the OLD.12i HEC-1 model were used to estimate hydraulic retention times and trap efficiency for the basin. As a result, the estimated trap efficiency, for wash

load, is estimated to be approximately 33%. Based on the calculated trap efficiency, the sediment yield characteristics for the Miller Road Basin are presented below with comparisons to the values reported for other basins of the Desert Greenbelt Project.

	CN5A	CN5B	Miller Road DB	Union Hills DB	Deer Valley DB	Happy Valley DB
Annualized Sediment Yield (Acre-feet)	0.06	0.10	0.17	0.33	0.70	0.70
100-year Sediment Yield (Acre-feet)	0.57	0.96	1.52	6.72	9.65	7.76
Subbasin Area (square-mile)	0.30	0.65	0.95	4.95	3.90	2.38
Mean Average Sediment Yield (Acre-ft/sq. Mile)	0.21	0.16	0.18	0.07	0.18	0.29
Total Sediment Storage Capacity (Acre-feet)	0.88	1.48	2.36	8.40	13.20	11.30
Average Storage Capacity (Acre-feet/sq.mile)	0.88	1.48	2.48	1.70	3.38	4.75

Table 3 - Sediment Yield

The sediment yield values provided above for the Miller Road detention basin represent an upper limit of sediment yield because of the assumption made that sediment was abundantly available within the contributing watershed. This assumption is applicable for the undeveloped conditions that currently exist, but may be too conservative for post-development conditions where the planned land-use for approximately 40% of the contributing watershed is expected to be industrial/commercial. At this time, the basin will be designed to provide 2.3 acre-feet of dead storage volume for sediment deposition, which is approximately 3.6% of the total storage capacity of the basin.

GEOMETRIC LAYOUT

Collector Channels

The collector channels parallel to the Pima Freeway are configured to take full advantage of the available ADOT right-of-way not used for the freeway improvements. The stipulation for the horizontal placement of the collector channels requires the top of bank for the channels not be located closer than 16 feet from the toe of embankment/cut slope of the freeway. The collector channels are shown on plan and profile sheets 11 through 16 in Appendix 1. The centerline of the channels was established in conjunction with the proposed finished grade of the collector channel and the grade established by the geometric control for the Pima Freeway[13]. Tables A1-1 and A1-2 in Appendix 1 provide indicative data regarding the horizontal location of the collector channels. Collector channel CN5A is approximately 1,820 linear feet extending from Scottsdale Road (Sta. 11+50) to the Miller Road Basin (Sta. 29+70). Collector channel CN5B is approximately 2,600 linear feet extending from Hayden Road (Sta. 64+50) to the Miller Road Basin (Sta. 38+43).

Cross sections of the two channels are also provided in the appendix. A grade ditch and small berm will be constructed on the upstream side of the channel to prevent surficial runoff into the channel and prevent erosion and rilling of the unlined side slope. The side slope adjacent to the Pima Freeway will be protected with soil cement bank protection. Two feet toe down is provided for the soil cement bank protection. The unlined invert and side slope will be hydroseeded for aesthetic improvements.

Improved inlet structures will be provided at areas along the channels where the channel intercepts natural washes. The improvements are expected to consist of soil cement lining, but more extensive improvements may be required depending upon the hydraulics and final design configuration.

The inlets into the Miller Road Basin are expected to be lined with soil cement, and grouted riprap protection within the basin will be provided to prevent local scour at the inlets. Again, depending upon the hydraulics, these improvements may be required to be reinforced concrete. Also, energy dissipation into the basin may be required, but is not expected at this time.

Miller Road Detention Basin

The Miller Road Detention basin has undergone considerable geometric changes from the 30% design plans. The areal extent of the basin has been altered to take advantage of the available ADOT right-of-way and is governed by the ADOT freeway and the improvement of the future 76th Street alignment. ADOT requires the top bank of the basin to be a minimum of 20 feet from the toe of the freeway embankment/cut slope. A minimum 20-foot offset was also used adjacent to the 76th Street embankment.

As part of this contract, Pentacore was required to estimate the future vertical profile for the 76th Street crossing (bridge over-pass) of the ADOT freeway to establish the location of the Miller Road Basin. Pentacore coordinated with ADOT's consultant to determine the minimum clearance that will be required for the future bridge, and coordinated with TY Lin International, the structural engineer for the Pima Road Drainage System, to estimate the span and depth of the bridge deck and other properties of the bridge. The allowable approach grade from the City's Design Standards is 6-percent, however, after coordination with the City, a 3-percent approach grade was conservatively used to establish the location of the basin. The estimated toe of slope for 76th Street is previously shown in Figure 1.

The basin's top of bank extends approximately 750 feet, east to west, and 350 feet north to south. The maximum steepness of the unlined side slopes of the basin are 4H:1V. The collector channel inlets will be improved as discussed above and other improvements within the basin include the principal outlet and a sedimentation control berm. The 66-inch concrete principal outlet will be grated and protected by reinforced concrete headworks. The sedimentation control berm will be constructed of dumped riprap approximately 3-feet high. The toe of the berm will be protected with soil cement. The toe improvements are required to prevent the berm from being damaged or altered during normal maintenance activity.

Principal Outlet and Storm Drain

The basin principal outlet and storm drain are shown on plan and profile sheets 6 through 10 in Appendix 1. The 66-inch reinforced concrete restrictor pipe serving as the principal outlet is approximately 457 linear feet and extends from the Miller Road Basin (Sta. 73+73) to a drainage structure south of the Pima Freeway. At this structure the 66" pipe is enlarged to an 84" cast-in-

place concrete pipe which extends approximately 1200 linear feet to Union Hills Road (Sta. 57+16). From Union Hills Road, the 84" storm drain connects by means of a junction structure, to a 90-inch cast-in-place concrete pipe. This 90" storm drain is approximately 2,000 liner feet extending from Union Hills to the outlet channel of the OLD system (Sta. 37+20).

The horizontal alignment of both the principal outlet and the storm drain follows the 76th Street alignment and the majority of the pipe will be contained within the limits of the proposed right-of-way for the road. Since the right-of-way for 76th Street has not yet to be obtained by the City, a 30-foot width right-of-way strip will be required to construct and maintain the pipe. The principal outlet is skewed from the detention basin at an angle that will intersect with the 76th Street alignment just downstream from the ADOT right-of-way as shown on sheet 10. The storm drain departs from the 76th Street right-of-way just upstream of Princess Boulevard to drain into the outlet channel.

The vertical profile for the principal outlet and storm drain was established to provide a minimum 2-foot cover below an estimated finished 76th Street grade. A minimum 2-foot clearance from bottom of base course (ABC, select etc.) to top of pipe was also provided under the proposed Pima Freeway as coordinated with ADOT's design consultant.

The piping is expected to be cast-in-place concrete for the majority of the length. Improved reinforced concrete headworks will be provided at the inlet and outlet of the piping system. A pipe stubout will be provided at the manhole junction of the 84-inch and 90-inch pipe. The stubout will be provided for future use when local stormwater runoff will be collected at the intersection of 76th Street and Union Hills Road. Manholes are provided at 500-foot spacing intervals.

Outlet Channel

The outlet channel is shown on plan and profile sheets 4 through 6 in Appendix 1. The channel is approximately 2,720 linear feet extending from the northern property line of the TPC Golf Course (Sta. 10+00) to the 84-inch storm drain outlet (Sta. 37+20). The channel is proposed to be unlined between the storm drain outlet and the existing box culverts at Princess Boulevard (Sta. 22+44) except where the channel intercepts natural washes and at the 90° bend just upstream of the existing box culverts as shown on sheet 5. A typical cross section for the outlet channel is provided in the Appendix.

The horizontal channel alignment parallels an existing parcel property line from 76th Street to Scottsdale Road. The channel bends 90° to the south to flow perpendicular to the existing box culverts at Princess Boulevard. The channel then extends from the box culverts to the TPC following the alignment previously established by the 30% design plans.

Inlet improvements at the intercepted washes similar to those described for the collector channels will be provided. Reinforced concrete headworks will be provided for the 84-inch pipe outlet. Energy dissipation within the channel may be required at the 84-inch pipe inlet and at the TPC outlet. At this time, improvements near the existing natural wash near the intersection of 76th Street and Princess Boulevard (Sta. 36+00) are uncertain. Coordination with ASLD will be required to determine the improvements at this location. The channel will be lined with soil cement at the 90° bend and downstream from the existing box culverts. The channel lining may be determined to also be required from the 90° bend to the box culverts depending on hydraulics.

QUANTITIES AND COST ESTIMATES

Quantities and cost estimates for the OLD system are provided in Appendix 6. Table A6-1 summarizes a total cost of the OLD system. The total estimated cost of the OLD system, including a 15-percent contingency item, but excluding right-of-way acquisition costs is approximately $3.75 million.

Itemized costs are presented in Table A6-1 for the OLD system. Table A6-2 delineates the anticipated right-of-way areas required from each parcel within the project limits. The total amount of right-of-way necessary at this stage of the design is approximately 22.9 acres. A cost estimate for the purchase of this right-of-way is excluded from this report. Future value engineering sessions are expected to yield a more accurate estimate for right-of-way acquisition costs.

Table A6-3 presents a cost comparison between the OLD system and the 30% design based on Greiner's cost study report. The unit costs in Table A6-3 reflect the unit costs used by Pentacore and those used by Greiner. For both estimates, the cost of right-of-way acquisition has been excluded due to the uncertainty of an accurate price per acre for this real estate. In terms of right-of-way required for the project, the Greiner 30% report estimates approximately 29.5 acres required compared to the 22.9 acres required based on this report. This difference may translate into a substantial dollar amount once realistic real estate prices are targeted. Without considering right-of-way, the cost estimate prepared for this report is approximately $600,000 higher than the Greiner 30% cost estimate. Much of this difference can be traced to the soil cement lining proposed for the outlet channel ($463,905), and the landscaping proposed around the Miller Road basin perimeter ($68,800). These costs, as well as the other costs however, are expected to be refined based on developments from the value engineering meetings scheduled before the final 10% submittal.

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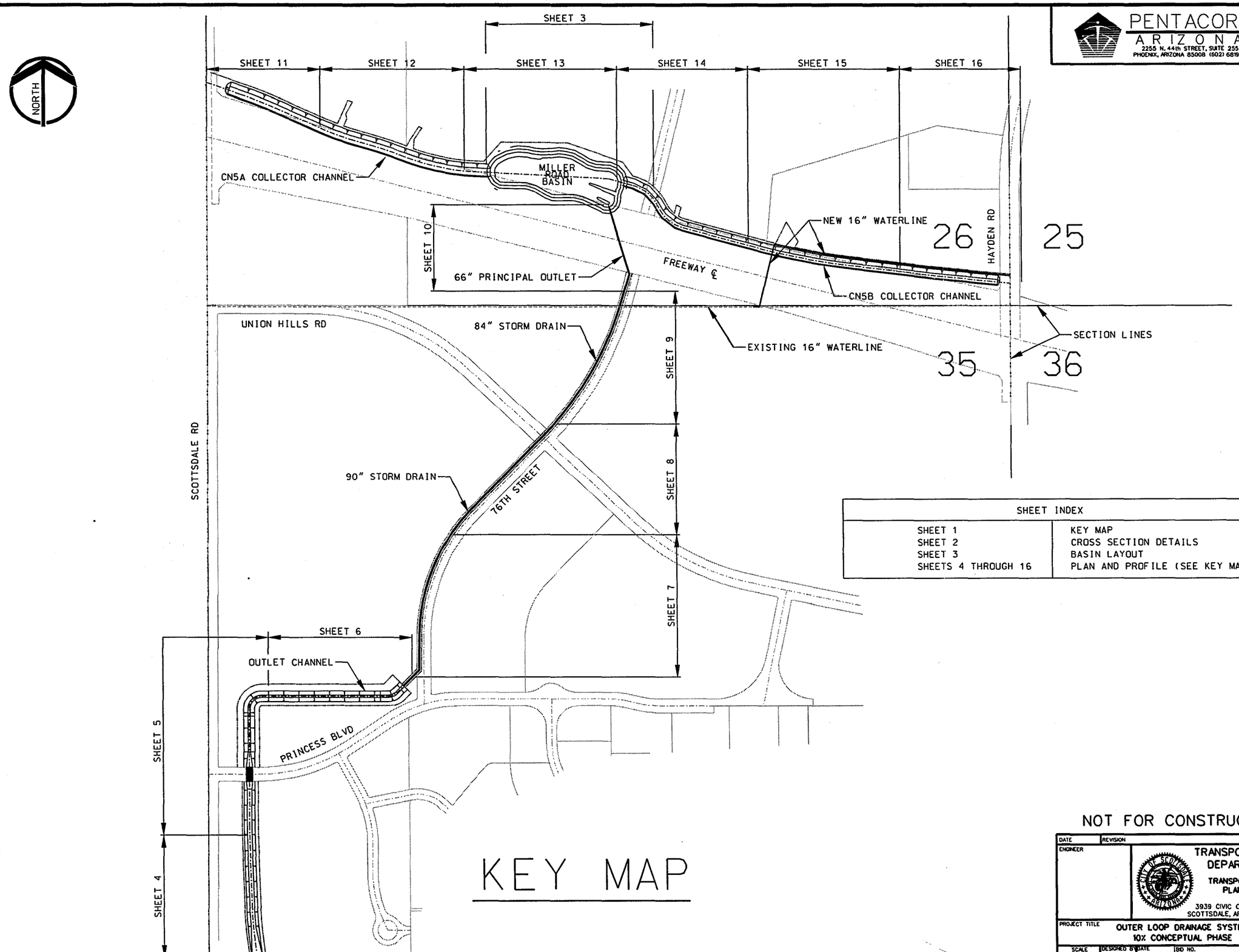
[3] Water Resources and Associates (Robert Ward), "Upper Indian Bend Wash Regional Drainage and Flood Control Plan," July 1992.
[4] Gilbertson Associates (Thomas Klimek), "Final Report Outer Loop Freeway Crossings Between Scottsdale and Pima Roads," October 1994.
[5] Ward, Memorandum and hydrology models delivered to William Erickson, City of Scottsdale, October, 1995.
[6] Simons, Li and Associates (Robert Ward), "Final Report, Drainage Analysis for State Trust Lands, Outer Loop Highway, Scottsdale Road to Pima Road," June 1987.
[7] City of Scottsdale, "Design Standards and Policies Manual," July 1996.
[8] Flood Control District of Maricopa County, "Drainage Design Manual for Maricopa County, Arizona, Volume II, Hydraulics."
[9] Bureau of Reclamation, "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Monograph No. 25, 1984.
[10] Federal Highway Administration, "Hydraulic Design of Highway Culverts", Hydraulic Design Series No. 5, September, 1985.
[11] Fuller, "Estimating Sediment Storage Requirements for On-line Detention Basins," Arizona Stormwater Storage Symposium, June 1997.
[12] Yang, *Sediment Transport Theory and Practice*, 1996.
[13] Arizona Department of Transportation, "Plan and Profile of Proposed Pima Freeway - Section 9A - Interim Roadway - Scottsdale Road - New Pima Road", ADOT Project RAM-600-1-335, March, 1996.



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SHEET INDEX	
SHEET 1	KEY MAP
SHEET 2	CROSS SECTION DETAILS
SHEET 3	BASIN LAYOUT
SHEETS 4 THROUGH 16	PLAN AND PROFILE (SEE KEY MAP)

NOT FOR CONSTRUCTION

DATE	REVISION	BY
ENGINEER		

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
1" = 600'	MPB	11/20/97		
	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	1 OF 16

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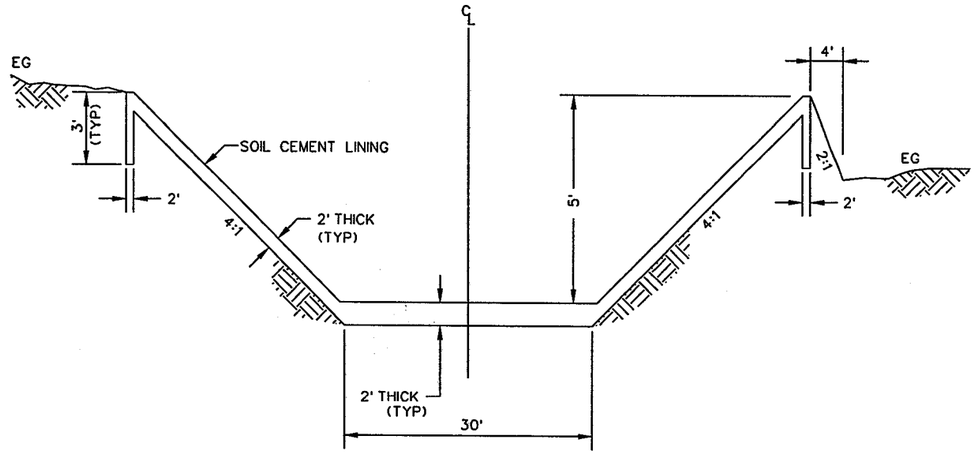
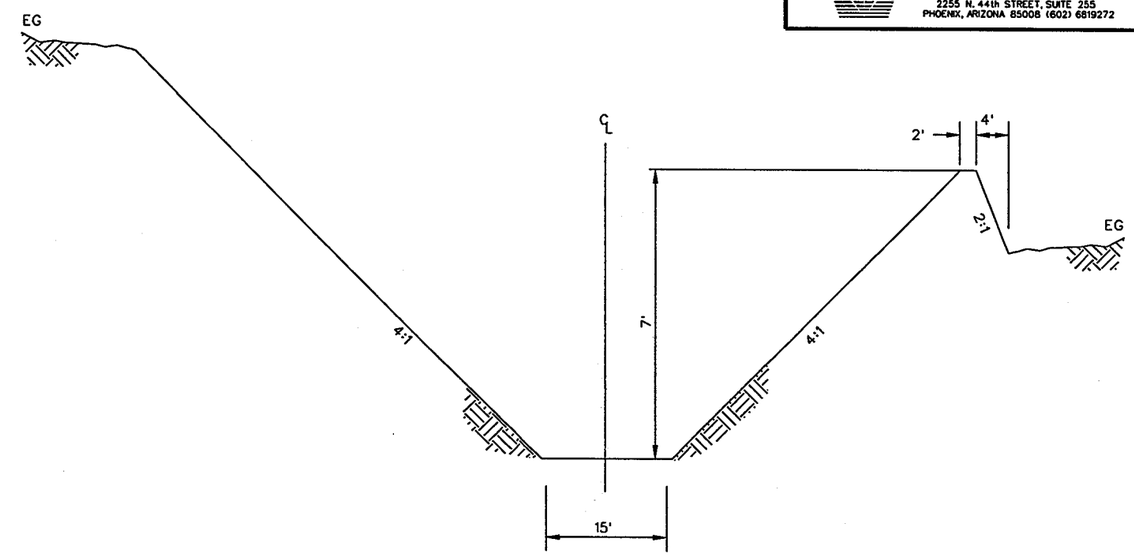
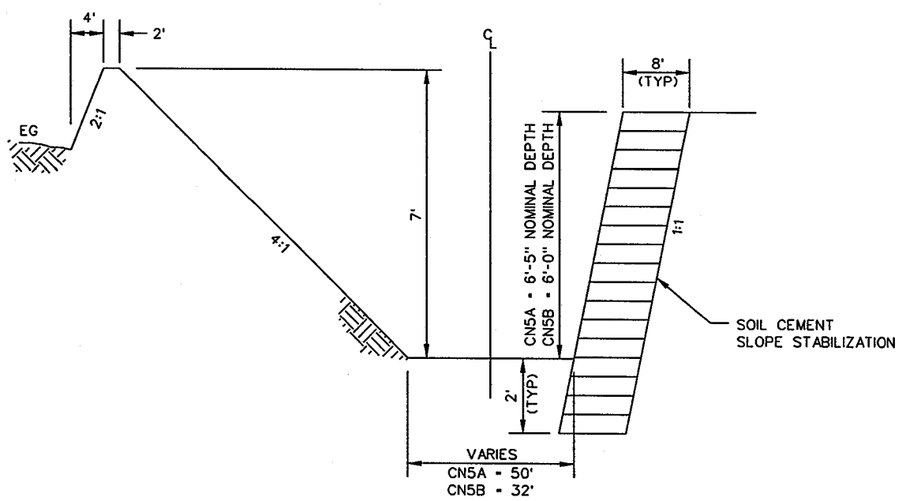


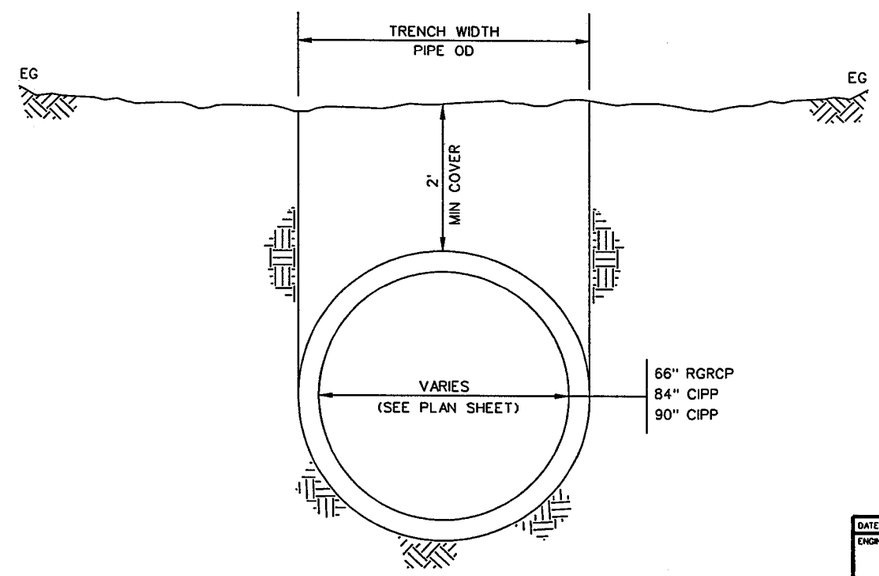
DETAIL A
IMPROVED OUTLET CHANNEL
NTS



DETAIL B
UNLINED OUTLET CHANNEL
NTS



DETAIL C
COLLECTOR CHANNELS CN5A AND CN5B
NTS



DETAIL D
STORM DRAIN OUTLET
NTS

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DATE	REVISION	BY

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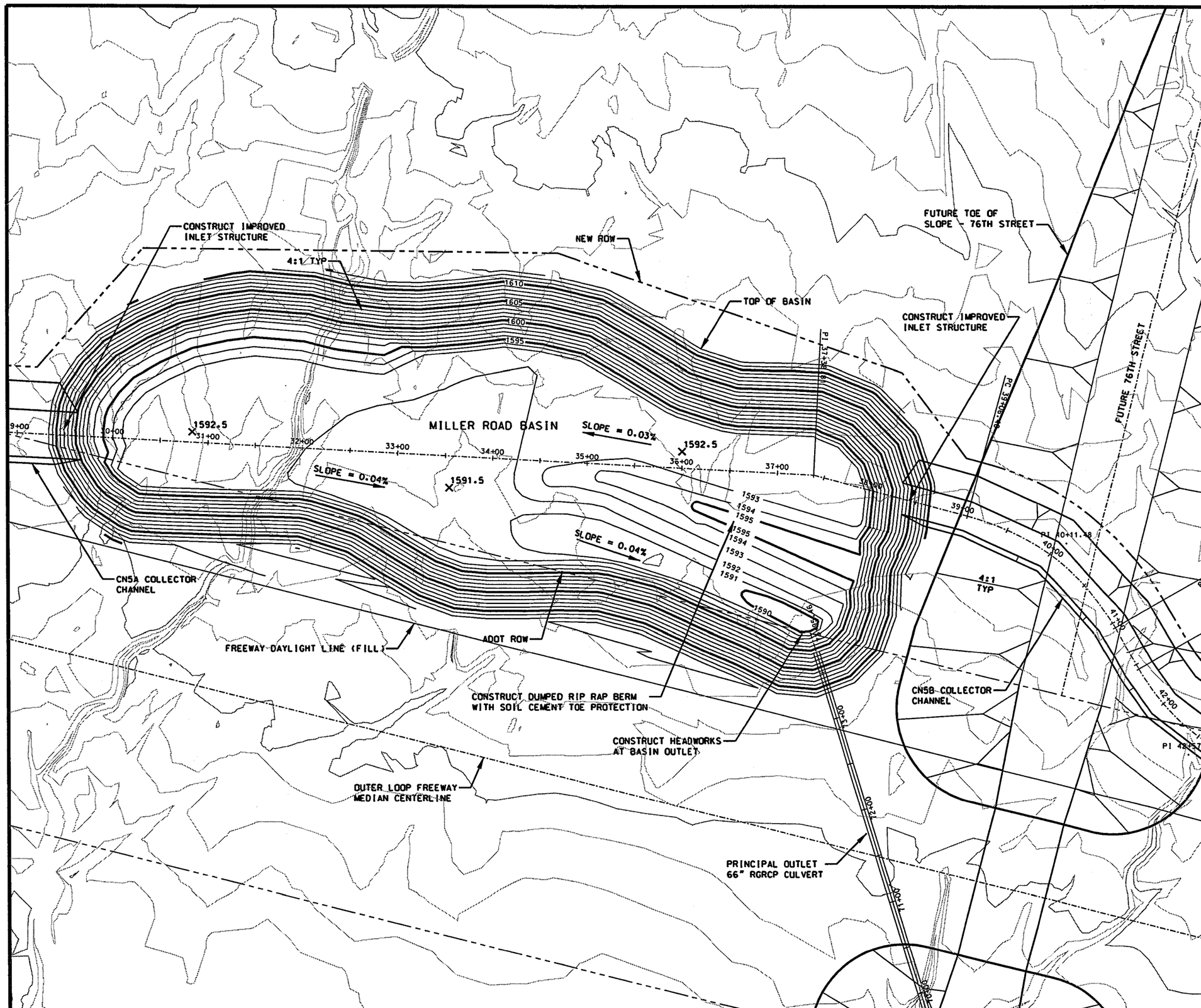
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
N.T.S.	MPB	11/20/97		
	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	2 OF 16



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REMOVAL NOTES

☐ Description	Unit	Quan.

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DATE	REVISION	BY

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TRANSPORTATION PLANNING
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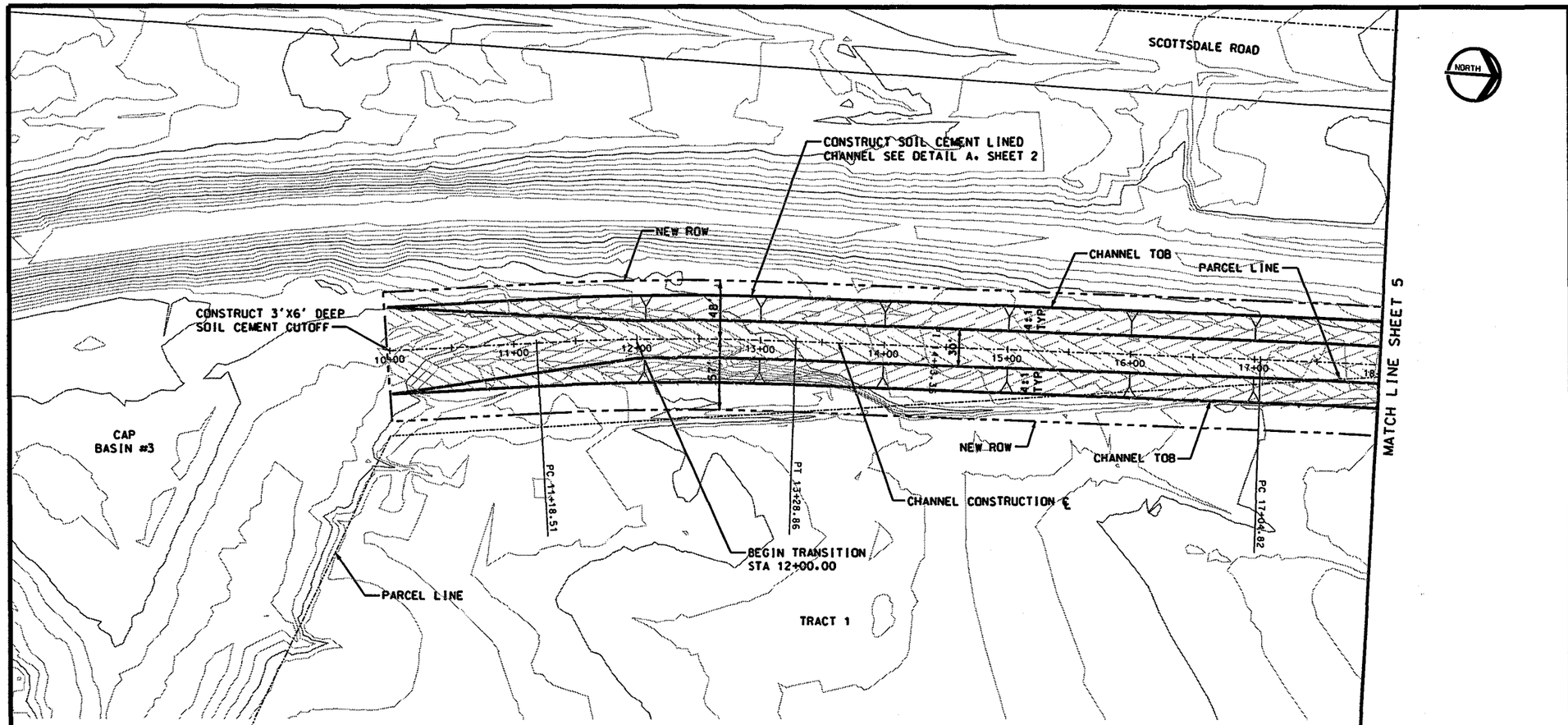
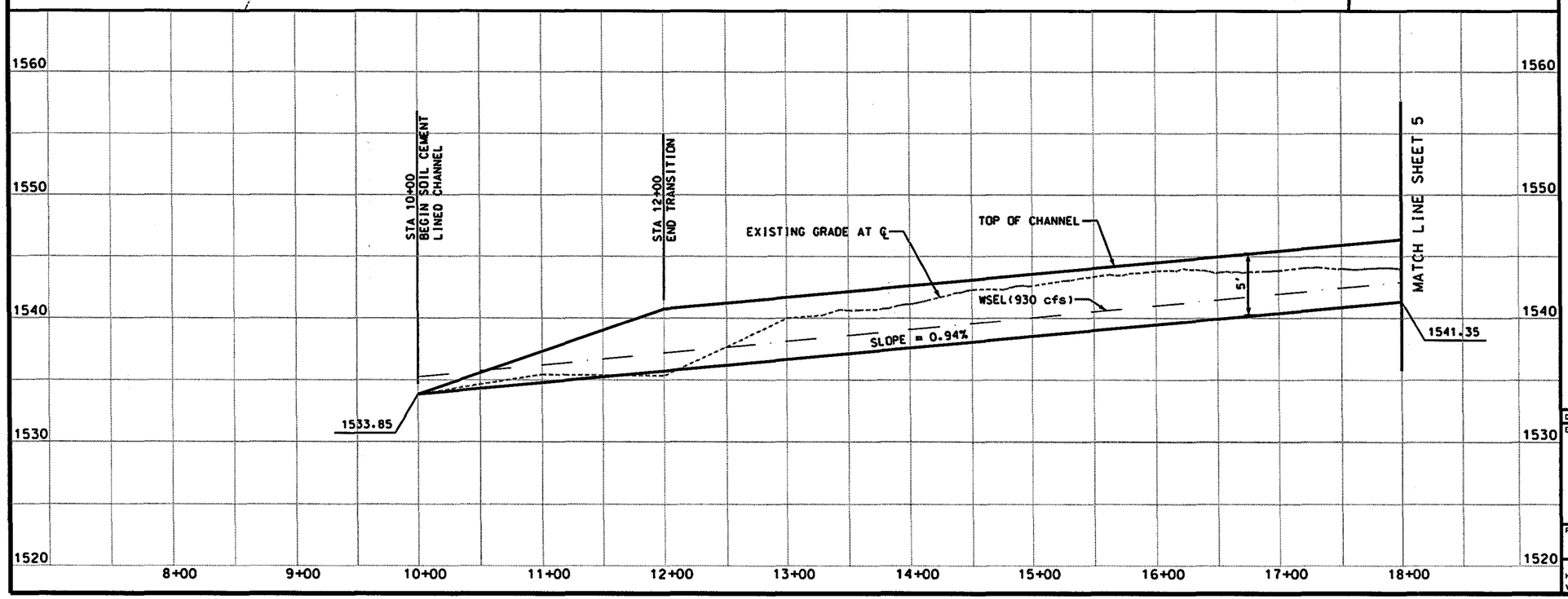
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE
OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	3 OF 16





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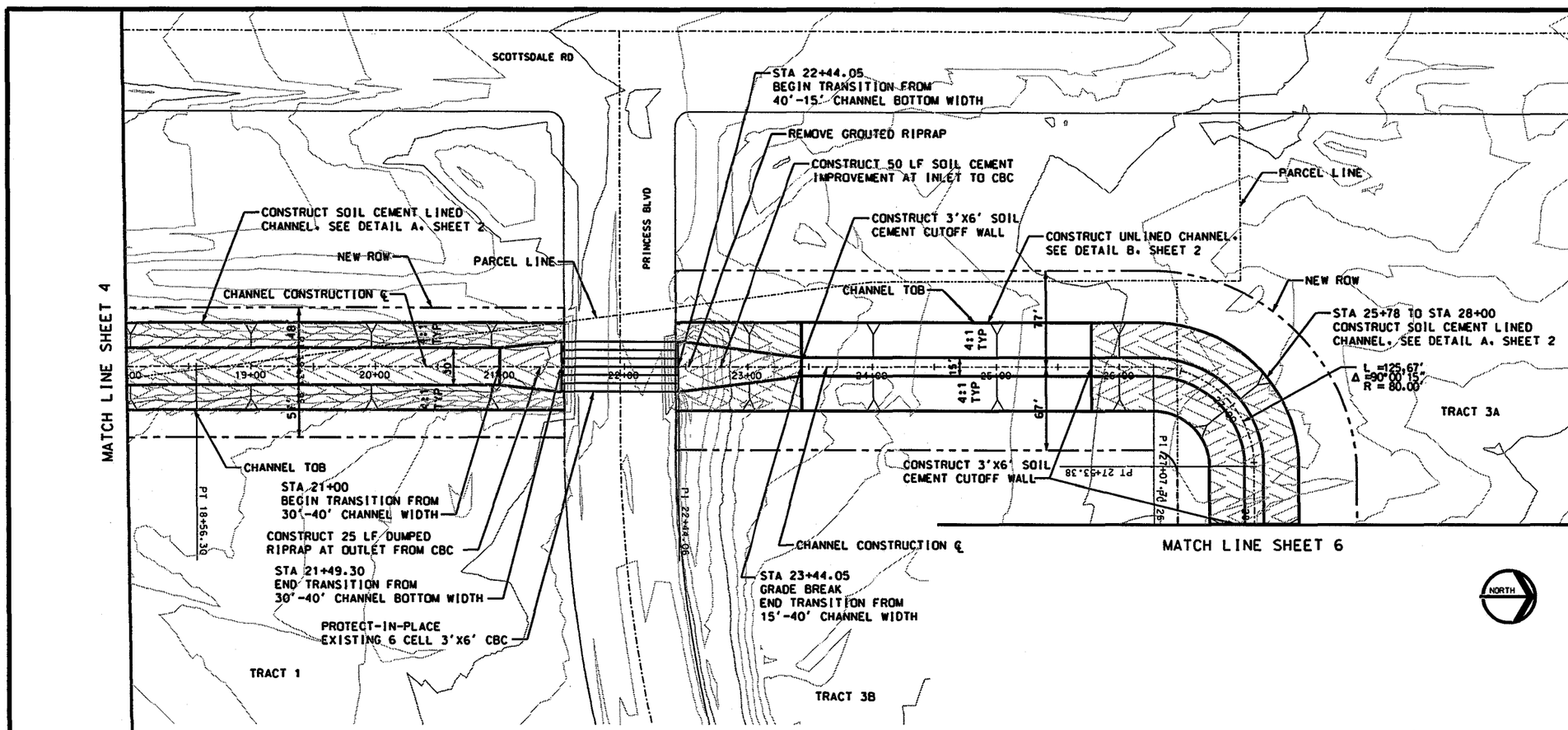
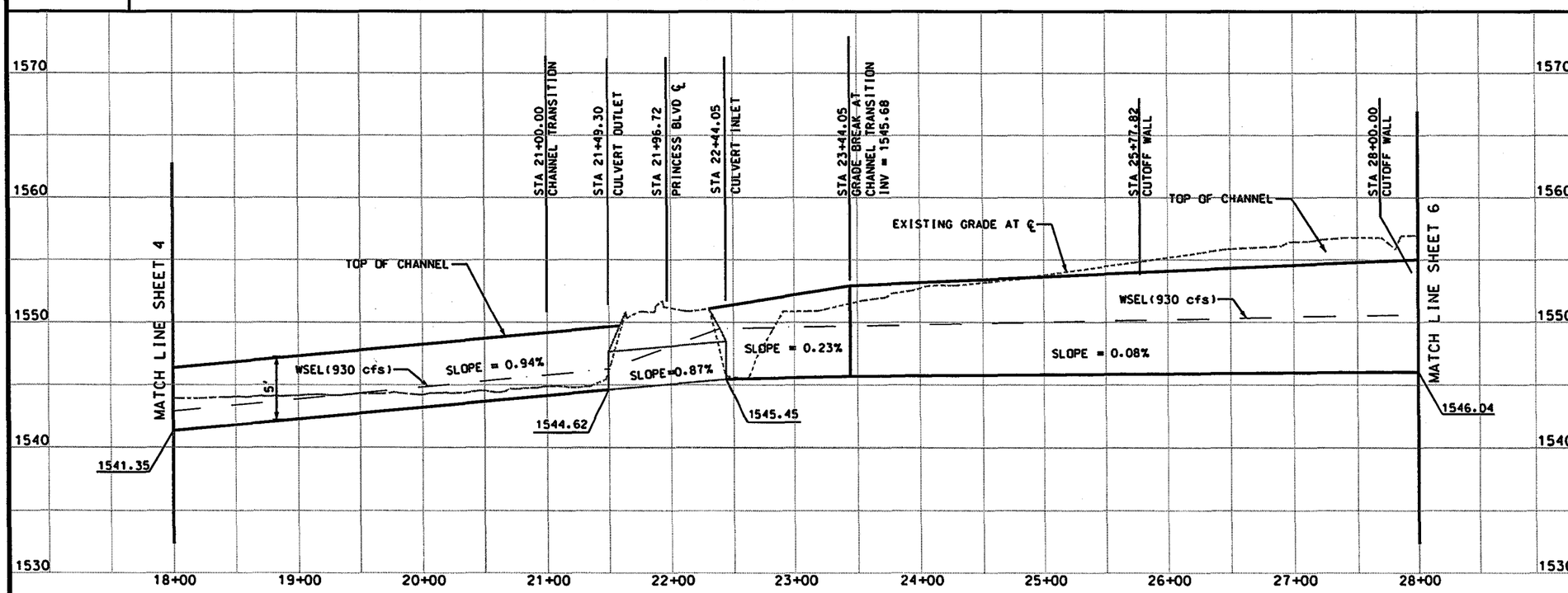
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	4 OF 16





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REMOVAL NOTES

☐ Description	Unit	Quan.

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DATE	REVISION	BY

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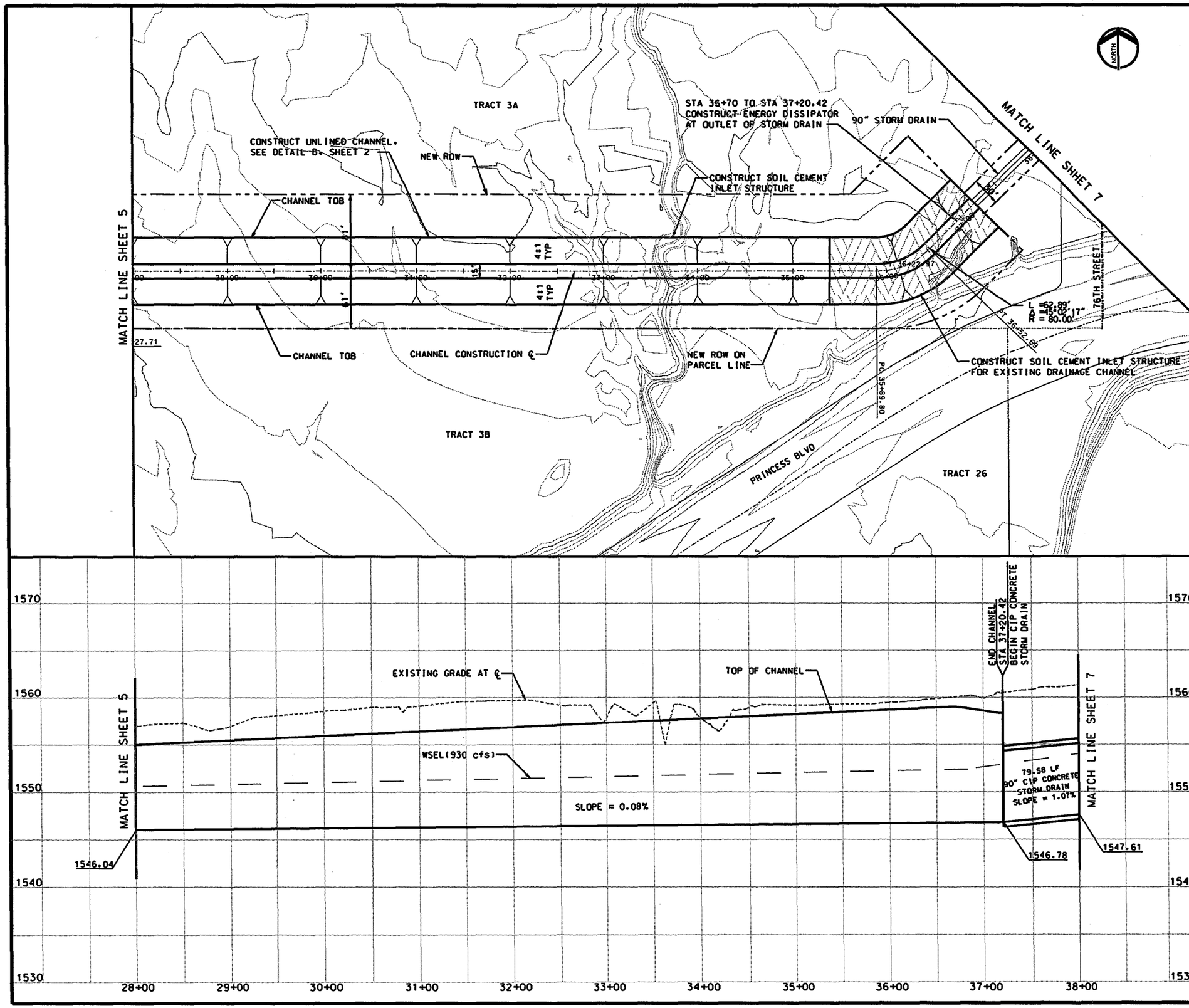
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	5 OF 16



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REMOVAL NOTES

☐ Description	Unit	Quan.

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DATE	REVISION	BY

ENGINEER

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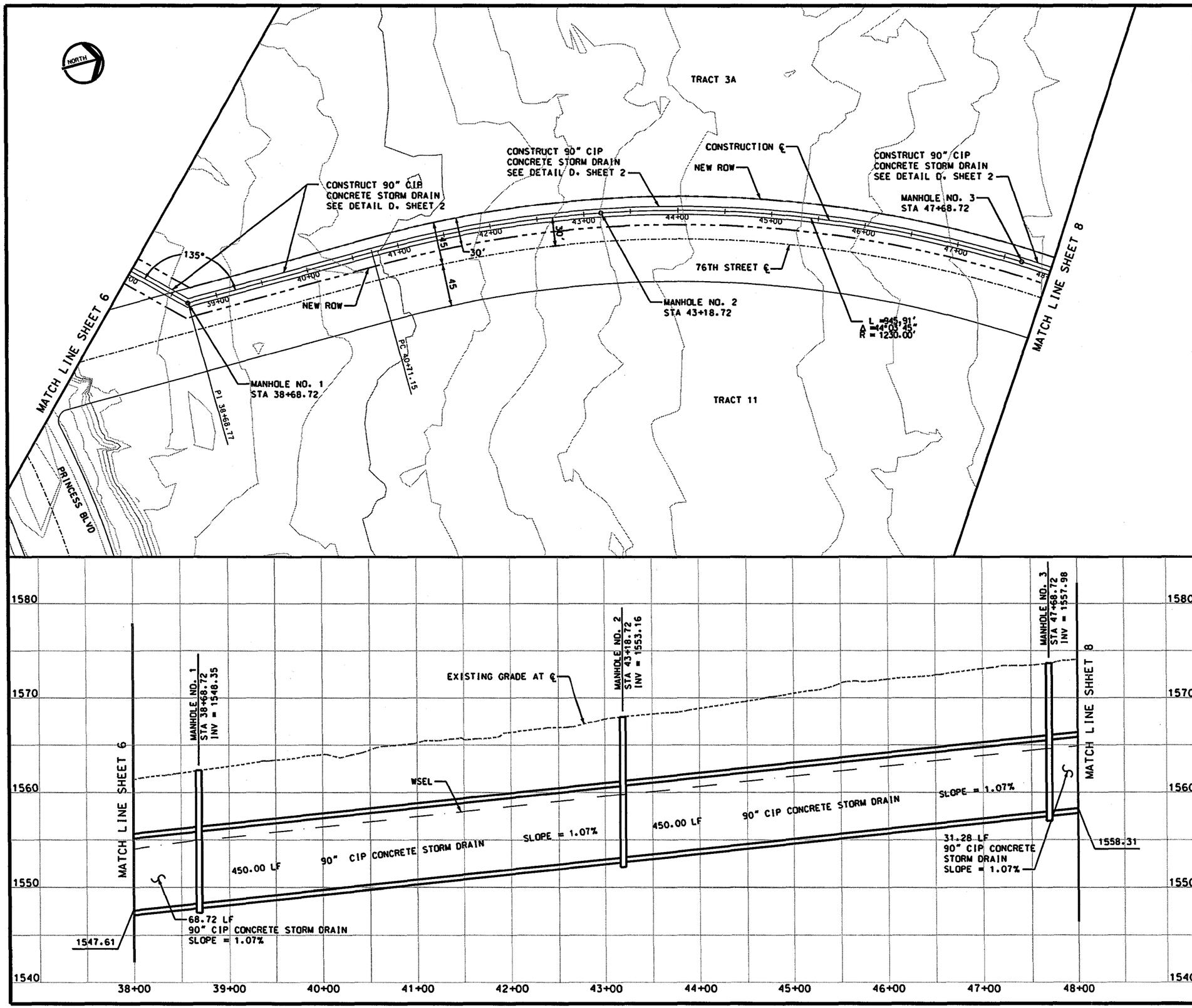
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	6 OF 16



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REMOVAL NOTES

☐ Description	Unit	Quan.

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DATE	REVISION	BY

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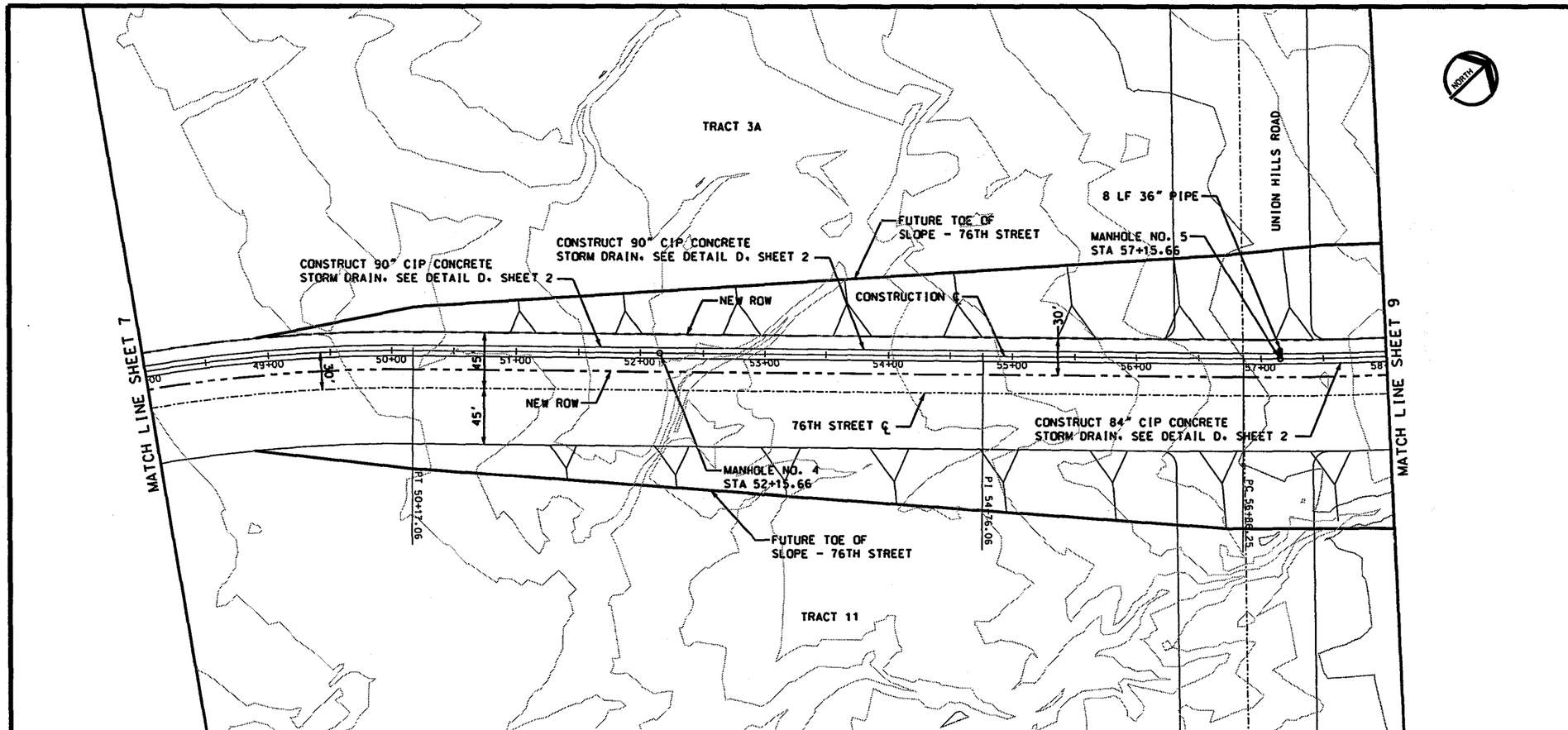
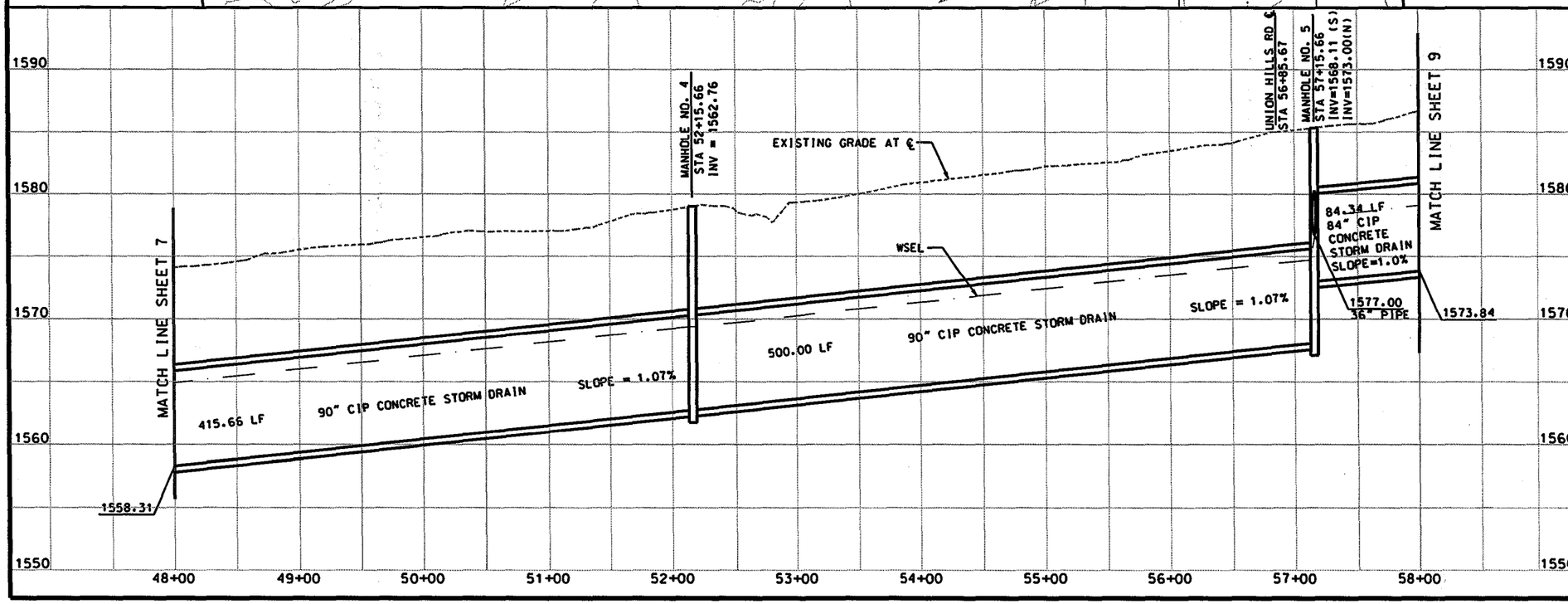
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY T.C.	AS-BUILT	PROJECT NO. 5001.0007	7 OF 16





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REMOVAL NOTES

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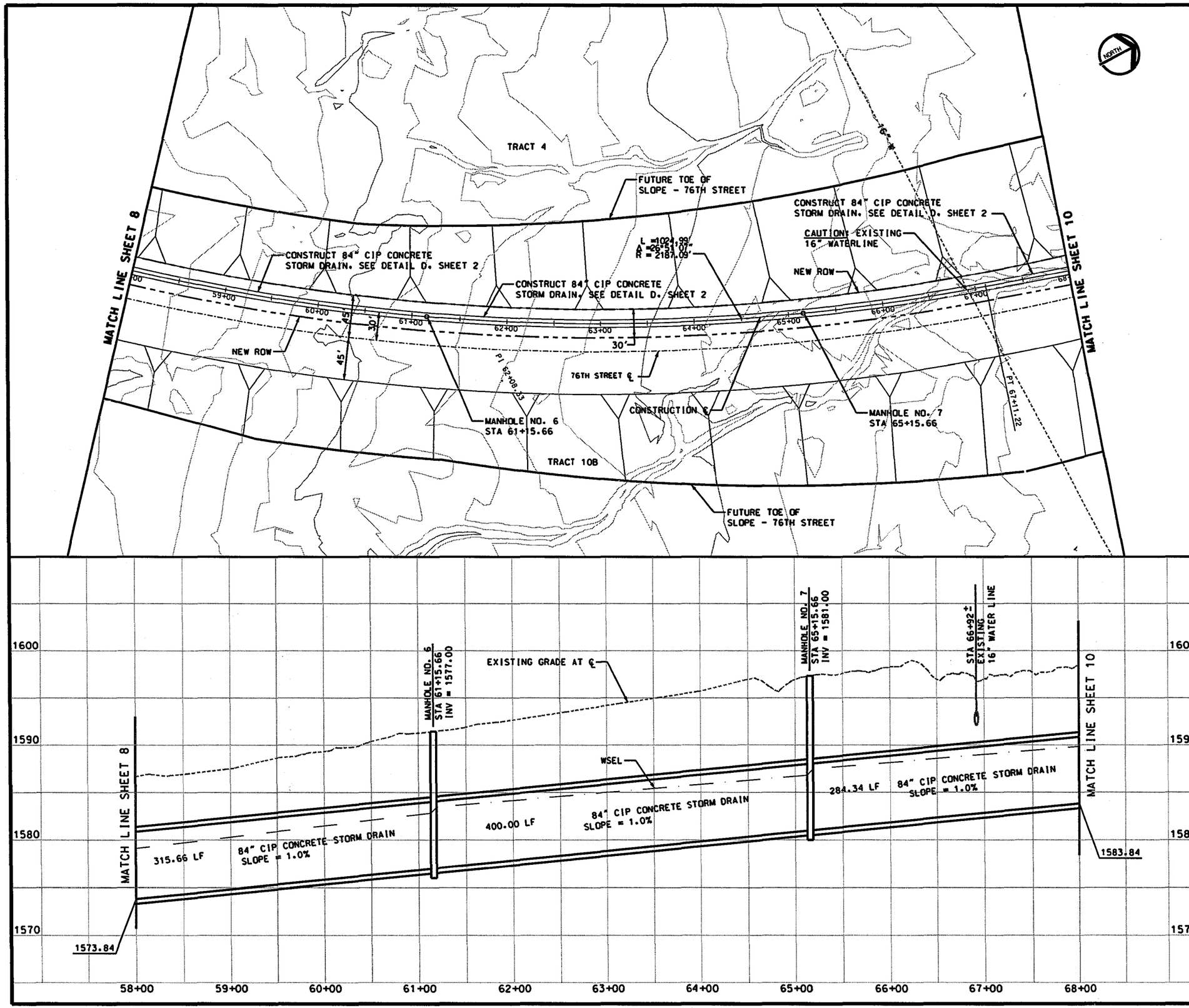
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	8 OF 16



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REMOVAL NOTES

☐ Description	Unit	Quan.

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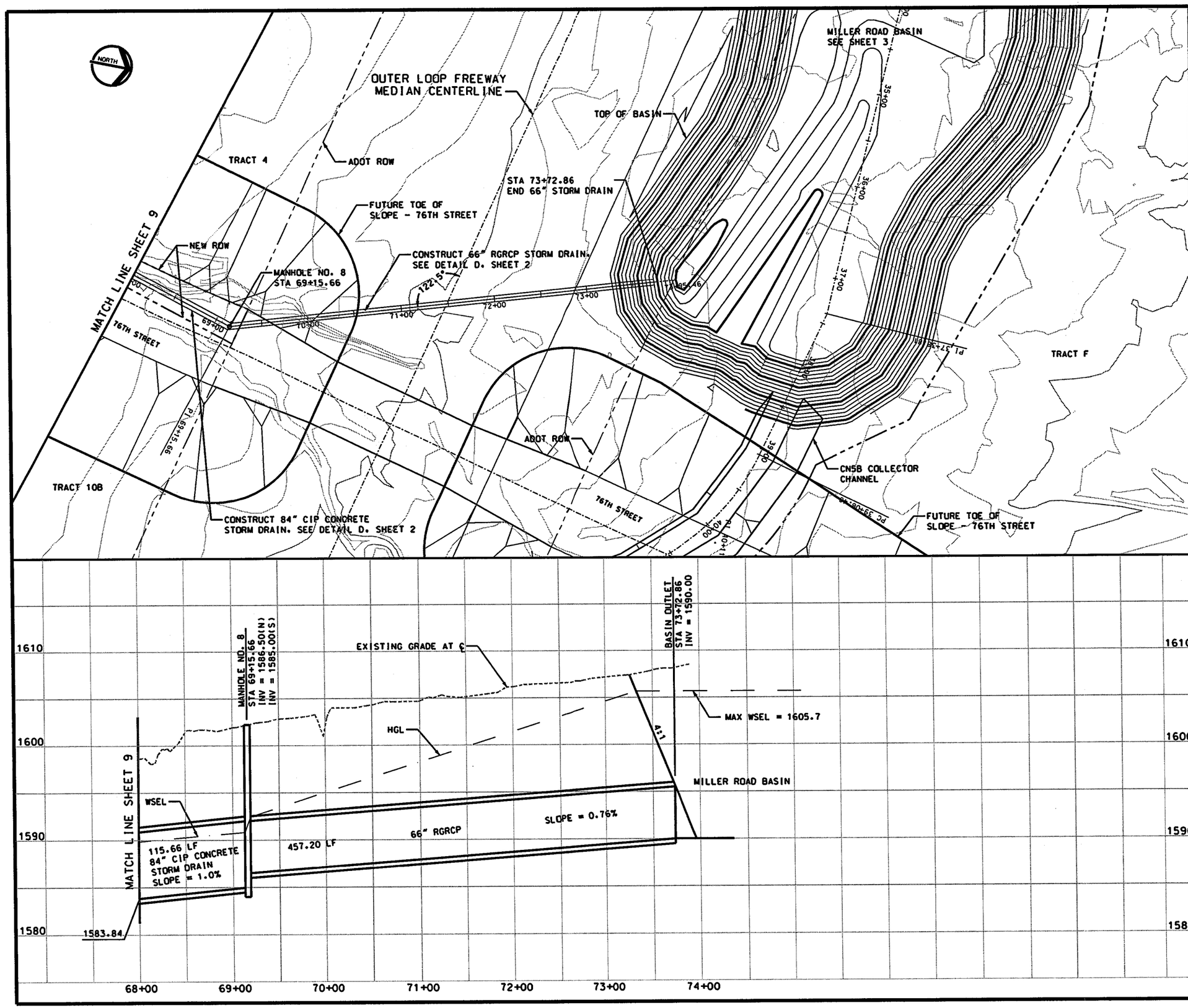
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	9 OF 16



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REMOVAL NOTES

☐ Description	Unit	Quan.

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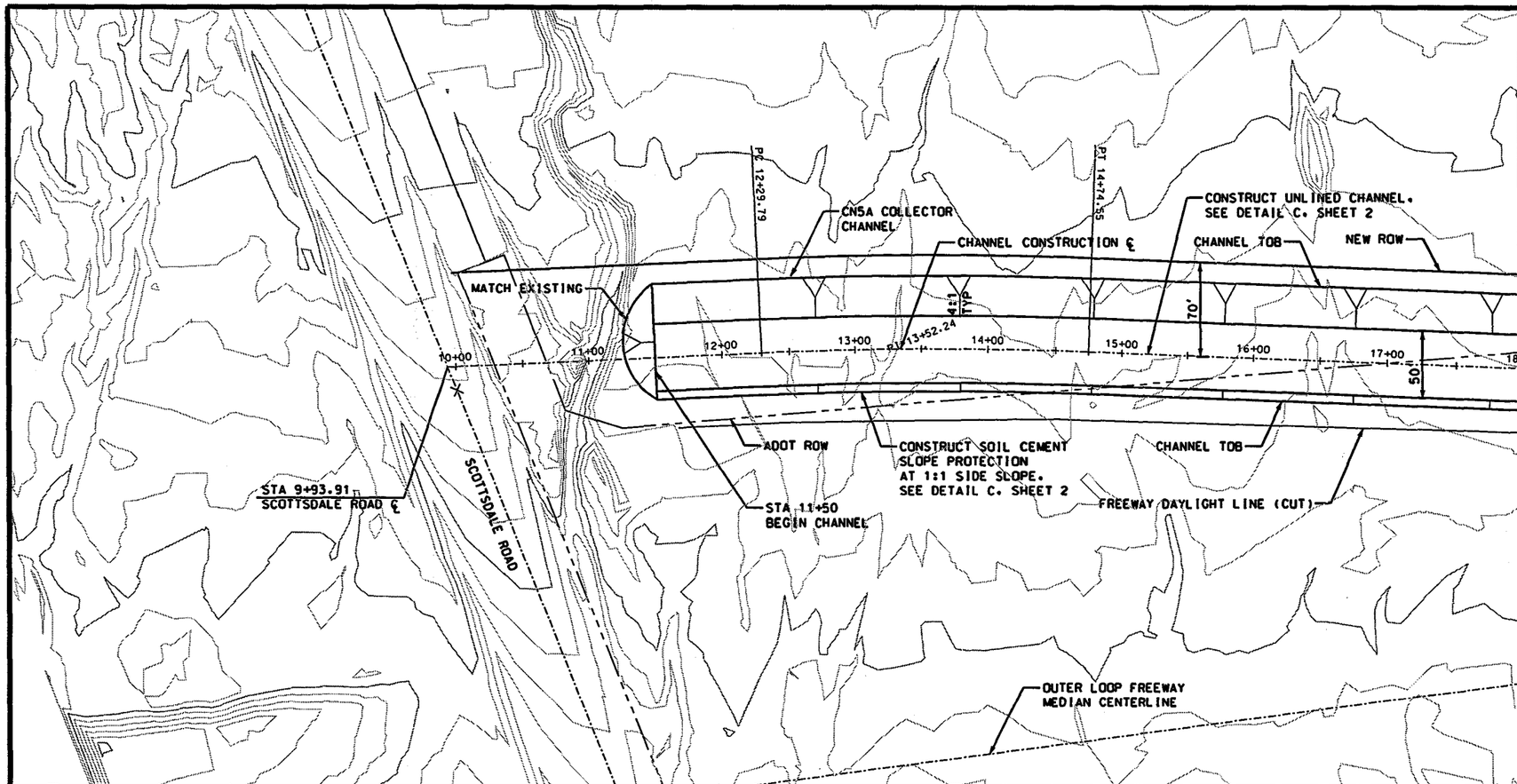
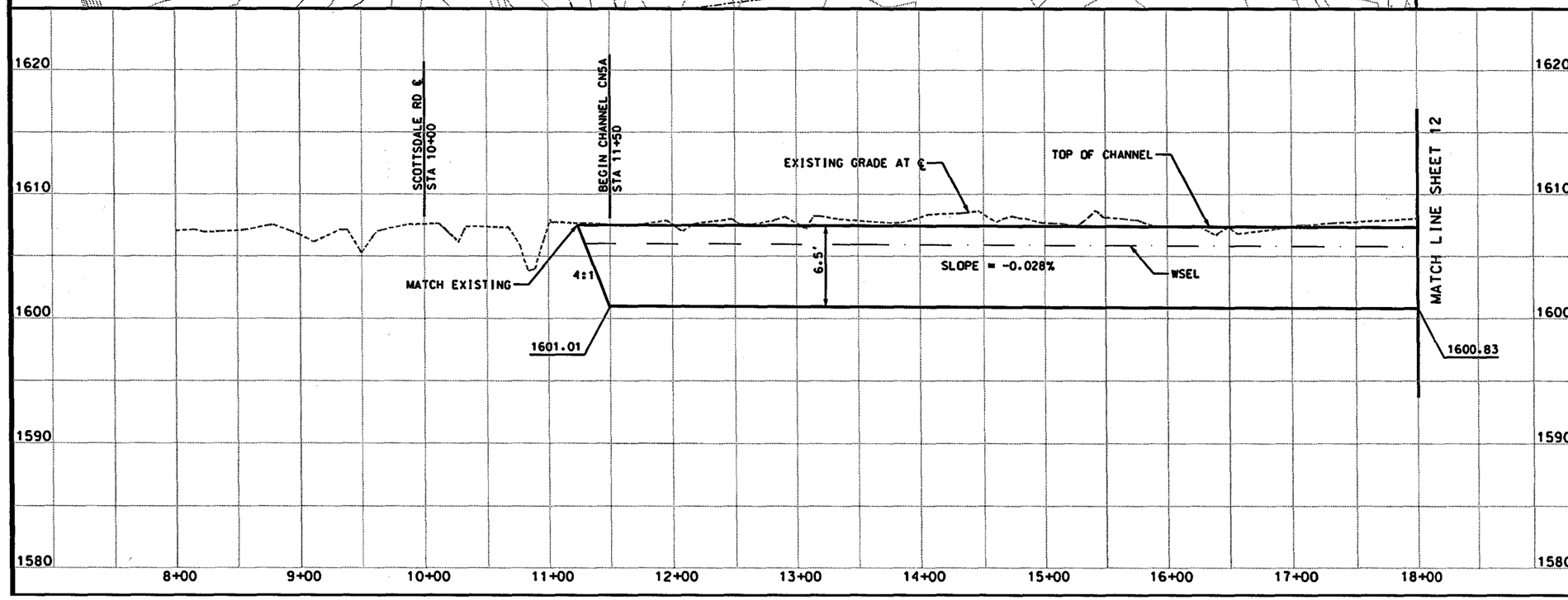
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	10 OF 16





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REMOVAL NOTES

☐ Description	Unit	Quan.

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PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
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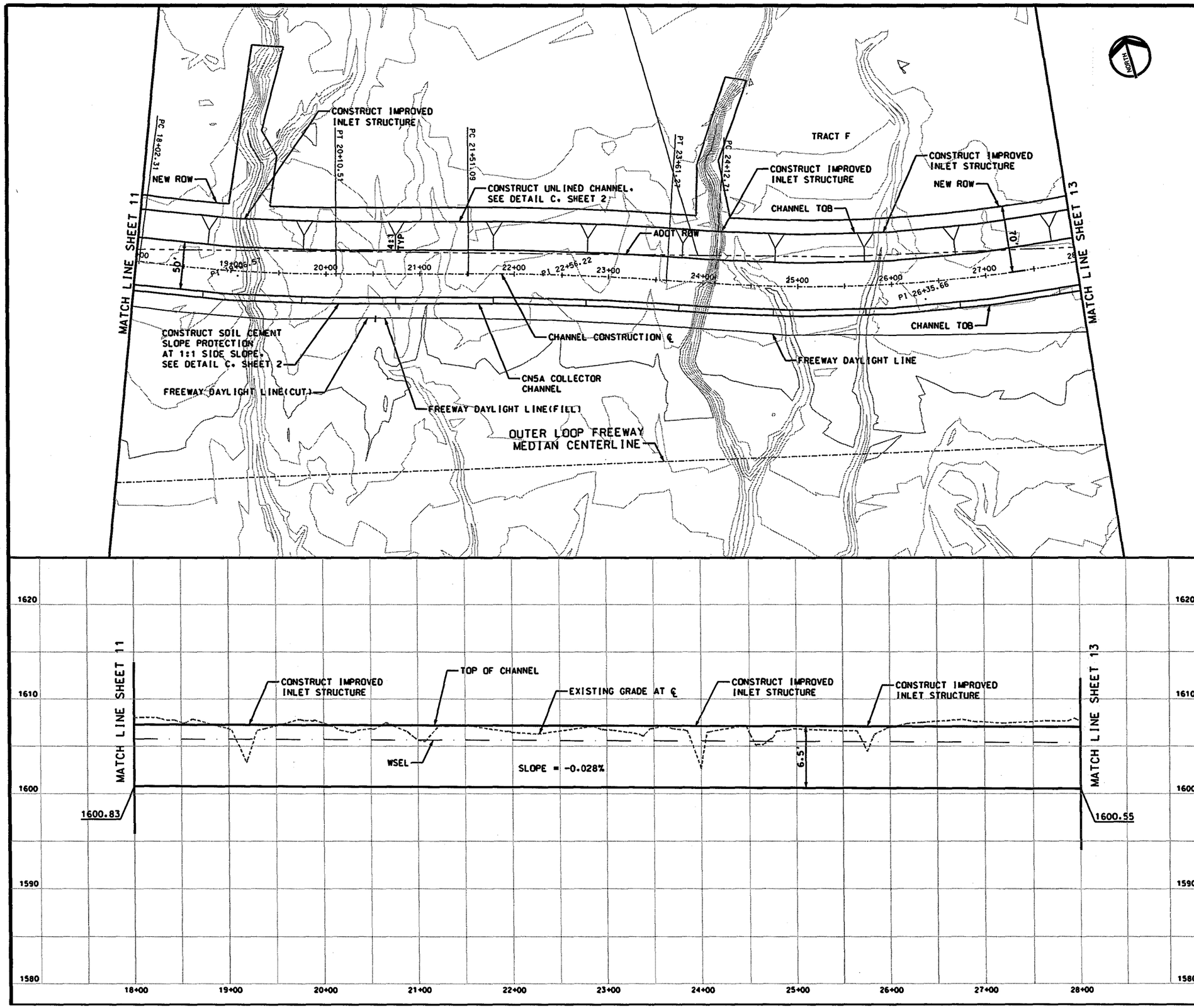
SCALE	DESIGNED BY	DATE	BD NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	11 OF 16



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REMOVAL NOTES

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PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
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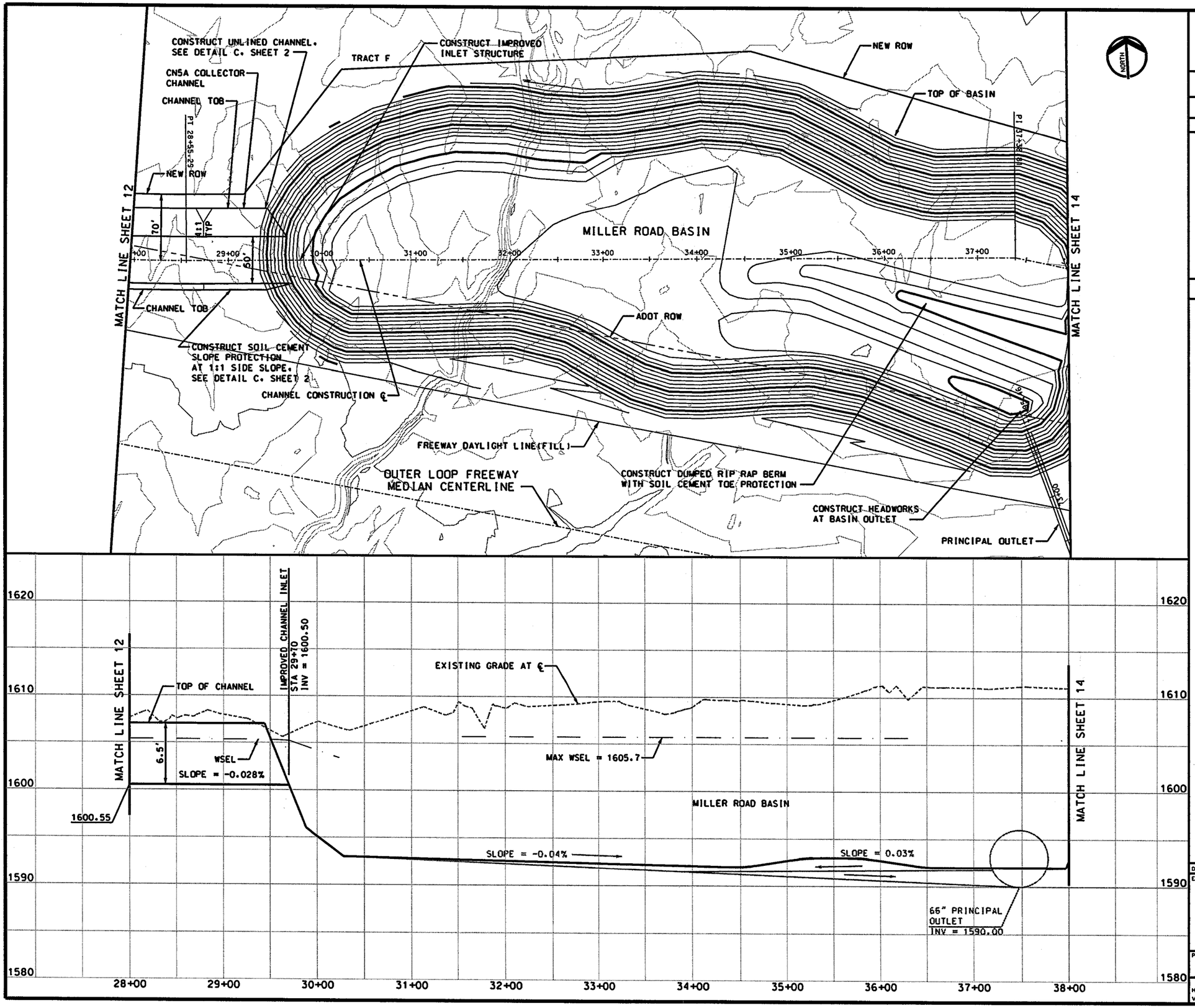
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	12 OF 16



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REMOVAL NOTES

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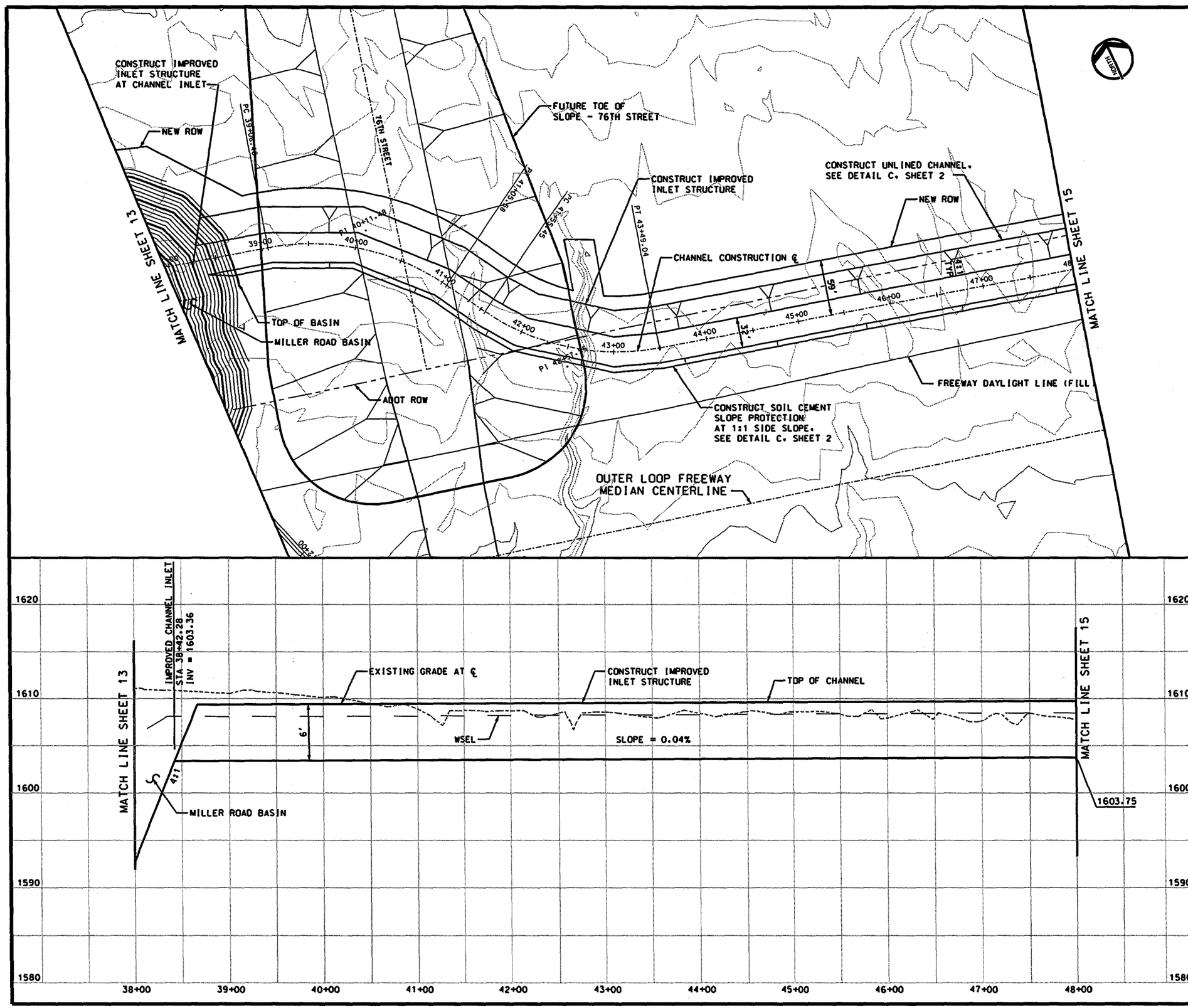
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	13 OF 16



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REMOVAL NOTES

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PROJECT TITLE: OUTER LOOP DRAINAGE SYSTEM
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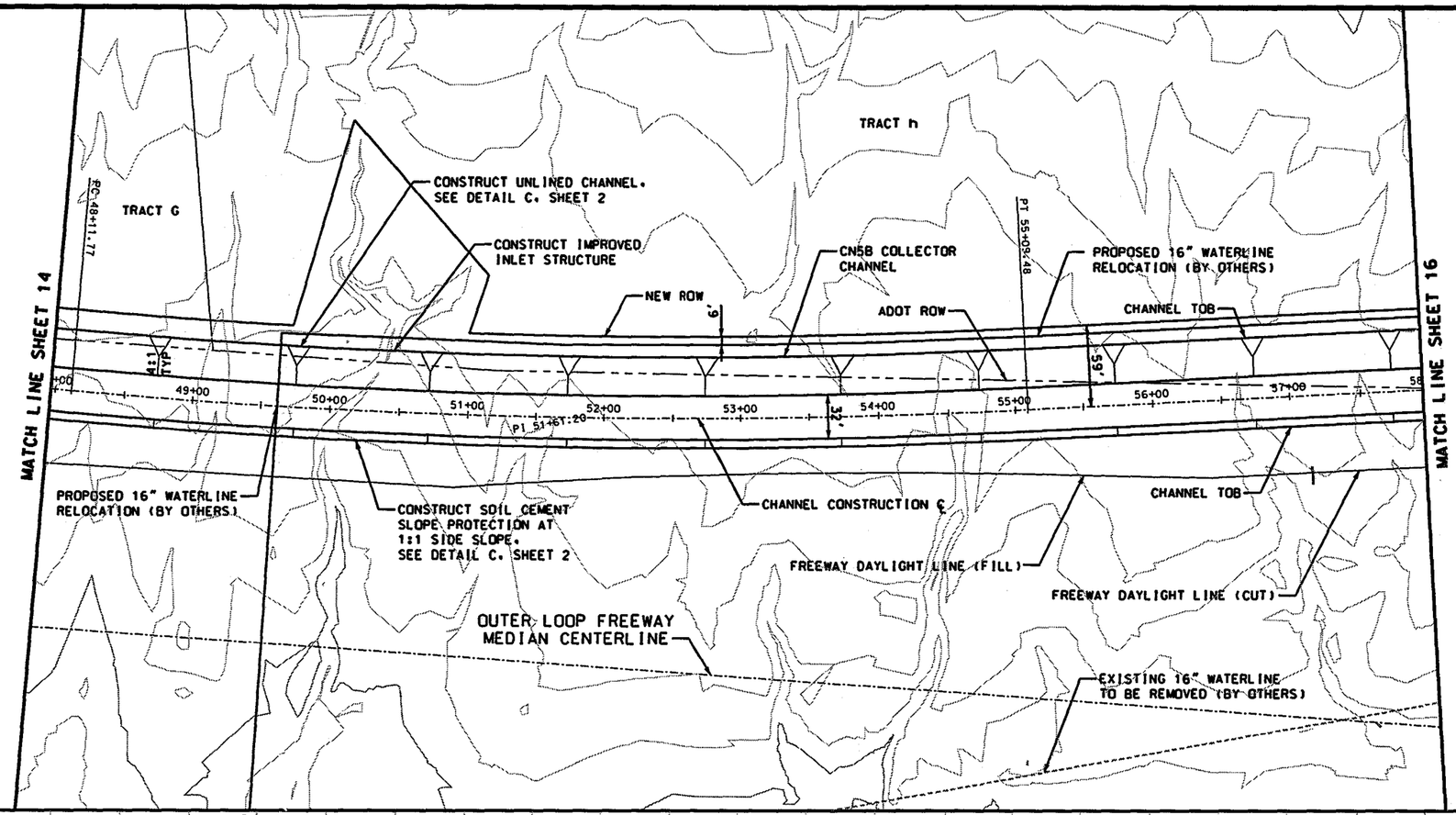
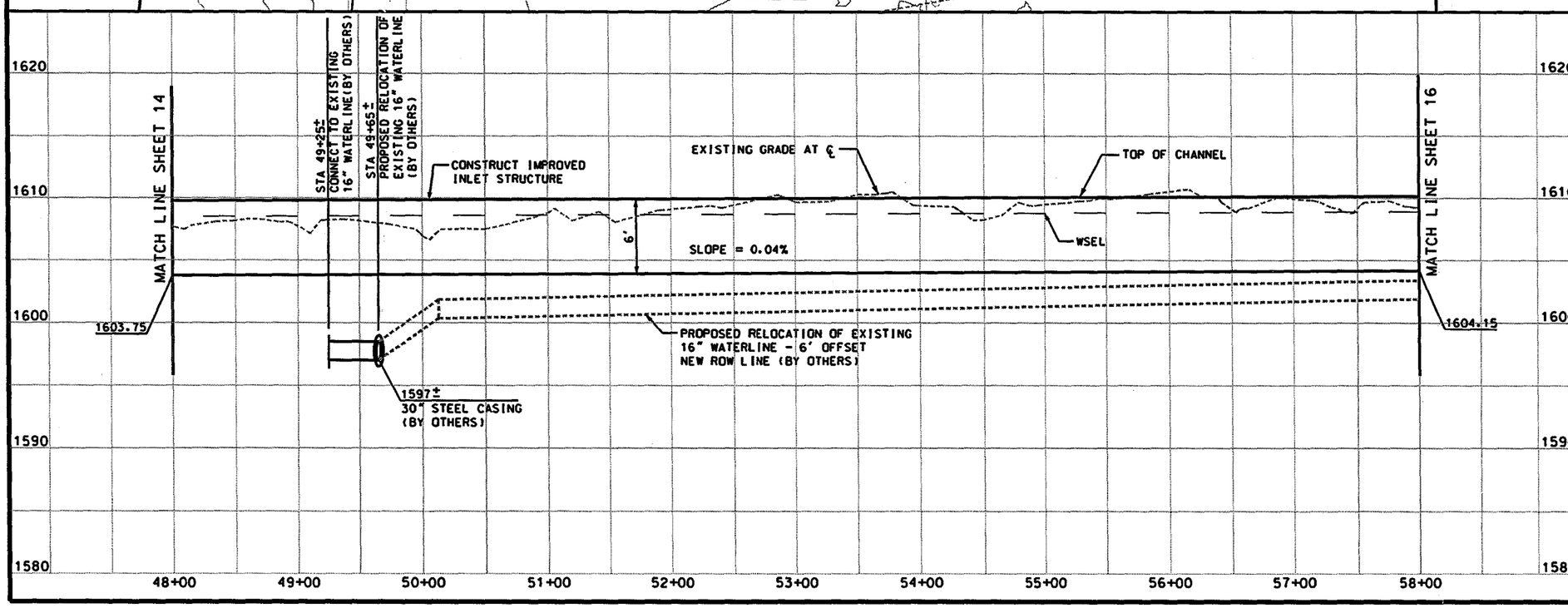
SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	14 OF 16





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SCOTTSDALE, ARIZONA 85251

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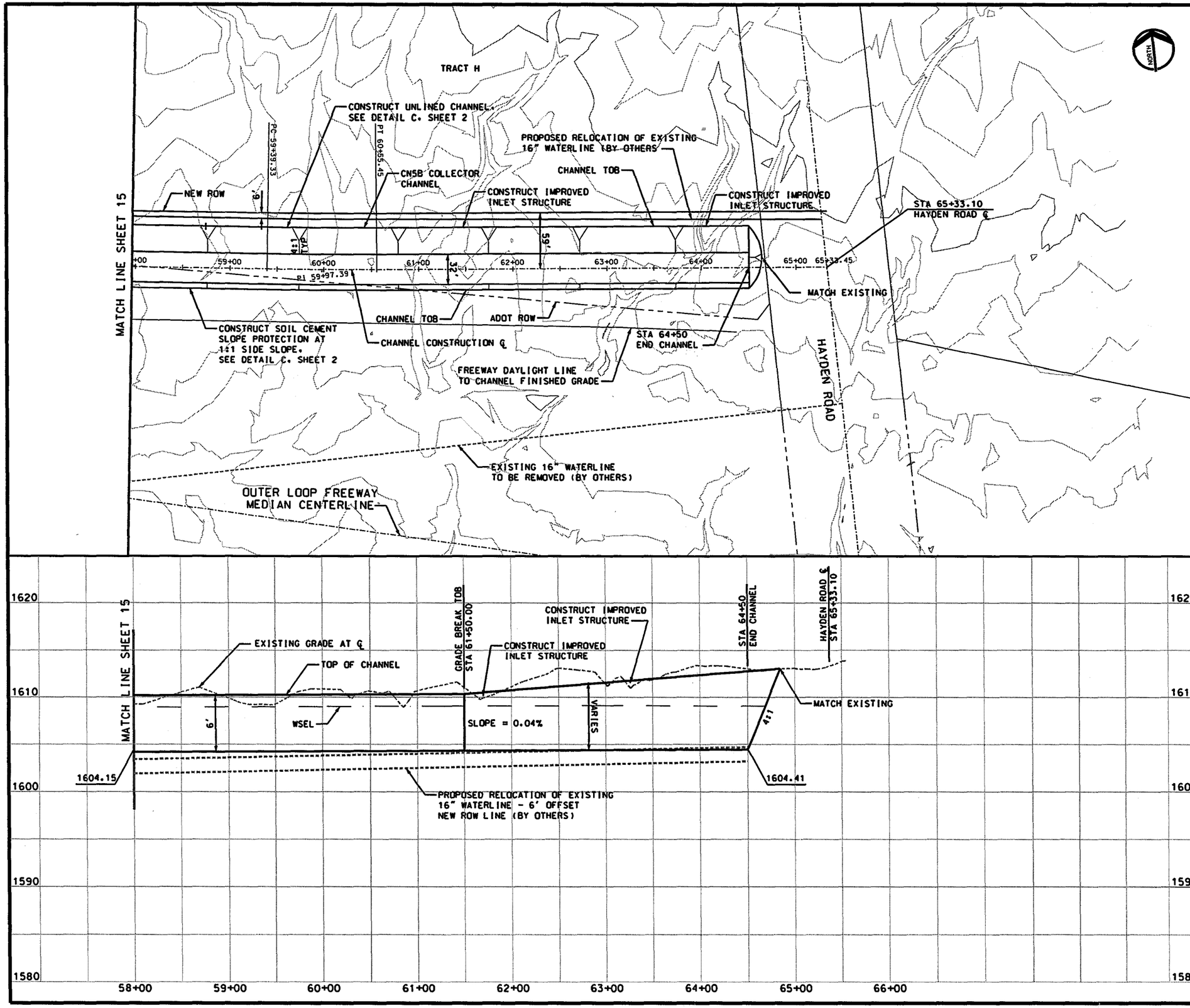
HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	15 OF 16



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HORIZ. 1"=100'	MPB	11/20/97		
VERT. 1"=10'	DRAWN BY TJC	AS-BUILT	PROJECT NO. 5001.0007	16 OF 16

Outer Loop Drainage System
ROW Control
Ramp C

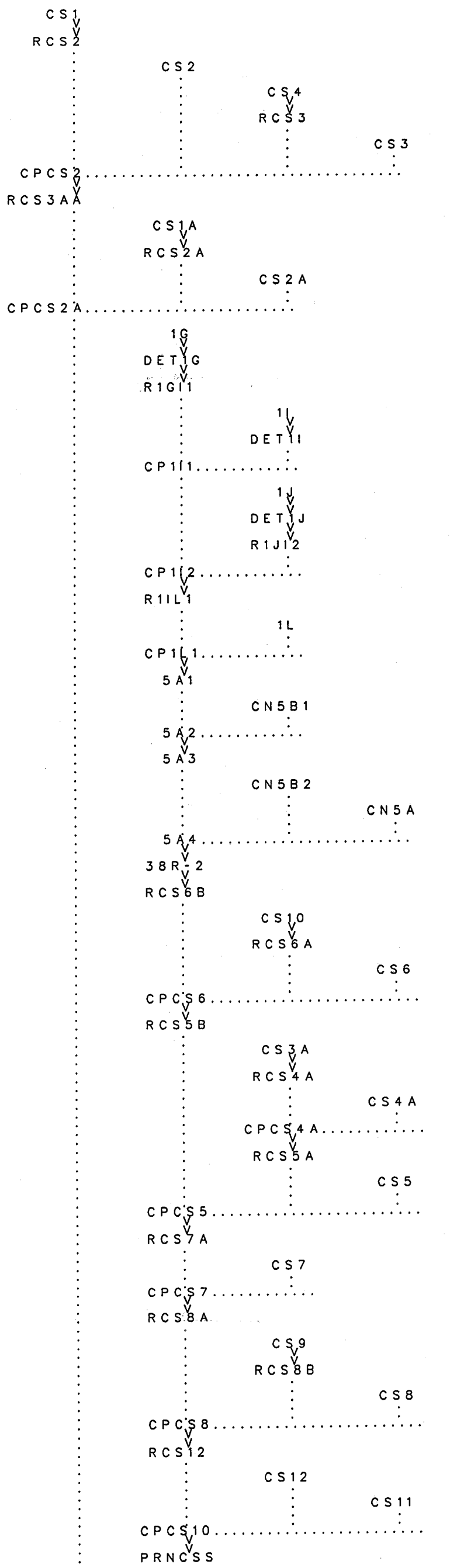
ADOT Ramp C Station	Ramp Control Line Elevation	Ramp Grade	Ramp Hinge Elevation	Collector Channel Station	Collector Channel Invert Elevation	Channel Slope	Water Surface Elevation	Freeway Embank ment '+Fill/ -Cut	Offset Distance Ramp to Channel CL	Offset Distance Channel CL to ROW	Offset Distance Ramp to ROW
1854+93.13	1605.55	-0.02694	1601.86								
1855+00.00	1605.36	-0.02694	1601.67								
1855+18.37	1604.87	-0.02694	1601.18								
1856+00.00	1604.48	-0.00473	1600.79	11+48.00	1601.01	-0.00028	1606.41	5.62	126.2	64.00	190.2
1856+43.13	1604.28	-0.00473	1600.59	11+90.00	1601.00	-0.00028	1606.40	5.81	127.3	64.00	191.3
1857+00.00	1604.77	0.00857	1601.08	12+46.00	1600.98	-0.00028	1606.38	5.31	124.3	64.00	188.3
1858+00.00	1605.62	0.00857	1601.93	13+49.00	1600.95	-0.00028	1606.35	4.42	119.0	64.00	183.0
1859+00.00	1606.48	0.00857	1602.79	14+49.00	1600.93	-0.00028	1606.33	3.53	113.7	64.00	177.7
1860+00.00	1607.34	0.00857	1603.65	15+49.00	1600.90	-0.00028	1606.30	2.65	108.4	64.00	172.4
1861+00.00	1608.20	0.00857	1604.51	16+49.00	1600.87	-0.00028	1606.27	1.76	103.1	64.00	167.1
1862+00.00	1609.05	0.00857	1605.36	17+49.00	1600.84	-0.00028	1606.24	0.88	97.8	64.00	161.8
1863+00.00	1609.91	0.00857	1606.22	18+49.00	1600.81	-0.00028	1606.21	-0.01	92.5	64.00	156.5
1864+00.00	1610.77	0.00857	1607.08	19+49.00	1600.79	-0.00028	1606.19	-0.89	88.0	64.00	152.0
1865+00.00	1611.62	0.00857	1607.93	20+49.00	1600.76	-0.00028	1606.16	-1.78	83.6	64.00	147.6
1866+00.00	1612.48	0.00857	1608.79	21+49.00	1600.73	-0.00028	1606.13	-2.66	79.2	64.00	143.2
1866+93.13	1613.28	0.00857	1609.59	22+42.00	1600.70	-0.00028	1606.10	-3.48	75.1	64.00	139.1
1867+00.00	1613.21	-0.01059	1609.52	22+49.00	1600.70	-0.00028	1606.10	-3.41	75.4	64.00	139.4
1868+00.00	1612.15	-0.01059	1608.46	23+48.00	1600.67	-0.00028	1606.07	-2.38	80.6	64.00	144.6
1869+00.00	1611.09	-0.01059	1607.40	24+44.00	1600.65	-0.00028	1606.05	-1.35	85.7	64.00	149.7
1870+00.00	1610.03	-0.01059	1606.34	25+39.00	1600.62	-0.00028	1606.02	-0.32	90.9	64.00	154.9
1870+61.34	1609.38	-0.01059	1605.69	25+97.00	1600.60	-0.00028	1606.00	0.32	94.4	64.00	158.4
1871+00.00	1608.97	-0.01059	1605.28	26+34.00	1600.59	-0.00028	1605.99	0.71	96.8	64.00	160.8

Table A1-2

ROW Control
ADOT Ramp A

ADOT Ramp A Station	Ramp Control Line Elevation	Ramp Grade	Ramp Hinge Elevation	Collector Channel Station	Collector Channel Invert Elevation	Channel Slope	Water Surface Elevation	Freeway Embankment '+Fill/ -Cut	Offset Distance Ramp to Channel CL	Offset Distance Channel CL to ROW	Offset Distance Ramp to ROW
1894+21.64	1612.84	0.00975	1609.15	50+30.00	1603.84	0.0004	1608.53	-0.62	79.9	53.00	132.9
1894+91.00	1613.52	0.00975	1609.83	51+00.00	1603.87	0.0004	1608.56	-1.27	76.7	53.00	129.7
1895+00.00	1613.60	0.00975	1609.91	51+09.00	1603.87	0.0004	1608.56	-1.35	76.2	53.00	129.2
1895+00.61	1613.61	0.00984	1609.92	51+09.61	1603.87	0.0004	1608.56	-1.36	76.2	53.00	129.2
1895+92.00	1614.32	0.00777	1610.63	52+00.00	1603.91	0.0004	1608.60	-2.03	72.9	53.00	125.9
1896+00.00	1614.38	0.00777	1610.69	52+08.00	1603.91	0.0004	1608.60	-2.09	72.6	53.00	125.6
1896+94.00	1614.74	0.00378	1611.05	53+00.00	1603.95	0.0004	1608.64	-2.41	71.0	53.00	124.0
1897+00.00	1614.76	0.00378	1611.07	53+06.00	1603.95	0.0004	1608.64	-2.43	70.9	53.00	123.9
1897+95.00	1614.74	-0.00022	1611.05	54+00.00	1603.99	0.0004	1608.68	-2.37	71.2	53.00	124.2
1898+00.00	1614.74	-0.00022	1611.05	54+05.00	1603.99	0.0004	1608.68	-2.37	71.2	53.00	124.2
1898+95.00	1614.34	-0.00422	1610.65	55+00.00	1604.03	0.0004	1608.72	-1.93	73.4	53.00	126.4
1899+00.00	1614.32	-0.00422	1610.63	55+05.00	1604.03	0.0004	1608.72	-1.90	73.5	53.00	126.5
1899+00.61	1614.31	-0.00656	1610.62	55+05.61	1604.03	0.0004	1608.72	-1.90	73.5	53.00	126.5
1899+95.00	1613.72	-0.00624	1610.03	56+00.00	1604.07	0.0004	1608.76	-1.27	76.6	53.00	129.6
1900+00.00	1613.69	-0.00624	1610.00	56+05.00	1604.07	0.0004	1608.76	-1.24	76.8	53.00	129.8
1900+95.00	1613.10	-0.00623	1609.41	57+00.00	1604.11	0.0004	1608.80	-0.61	79.9	53.00	132.9
1901+00.00	1613.07	-0.00623	1609.38	57+05.00	1604.11	0.0004	1608.80	-0.58	80.1	53.00	133.1
1901+95.00	1612.48	-0.00624	1608.79	58+00.00	1604.15	0.0004	1608.84	0.05	83.3	53.00	136.3
1902+00.00	1612.45	-0.00624	1608.76	58+05.00	1604.15	0.0004	1608.84	0.09	83.5	53.00	136.5
1902+95.00	1611.85	-0.00624	1608.16	59+00.00	1604.19	0.0004	1608.88	0.72	87.3	53.00	140.3
1903+00.00	1611.82	-0.00624	1608.13	59+05.00	1604.19	0.0004	1608.88	0.75	87.5	53.00	140.5
1903+96.00	1611.22	-0.00624	1607.53	60+00.00	1604.23	0.0004	1608.92	1.39	91.3	53.00	144.3
1904+00.00	1611.20	-0.00624	1607.51	60+04.00	1604.23	0.0004	1608.92	1.41	91.5	53.00	144.5
1904+95.00	1610.61	-0.00622	1606.92	61+00.00	1604.27	0.0004	1608.96	2.04	95.3	53.00	148.3
1905+00.00	1610.57	-0.00624	1606.88	61+05.00	1604.27	0.0004	1608.96	2.08	95.5	53.00	148.5
1905+25.00	1610.42	-0.00622	1606.73	61+30.00	1604.28	0.0004	1608.97	2.24	96.5	53.00	149.5
1905+96.00	1609.98	-0.00627	1606.29	62+00.00	1604.31	0.0004	1609.00	2.71	99.3	53.00	152.3
1906+00.00	1609.95	-0.00627	1606.26	62+04.00	1604.31	0.0004	1609.00	2.74	99.4	53.00	152.4
1906+50.61	1609.63	-0.00624	1605.94	62+54.61	1604.33	0.0004	1609.02	3.08	101.5	53.00	154.5
1906+96.00	1609.39	-0.00541	1605.70	63+00.00	1604.35	0.0004	1609.04	3.34	103.0	53.00	156.0
1907+00.00	1609.37	-0.00541	1605.68	63+04.00	1604.35	0.0004	1609.04	3.36	103.2	53.00	156.2
1907+96.00	1609.09	-0.00285	1605.40	64+00.00	1604.39	0.0004	1609.08	3.68	105.1	53.00	158.1
1908+00.00	1609.08	-0.00285	1605.39	64+04.00	1604.39	0.0004	1609.08	3.69	105.1	53.00	158.1
1908+50.61	1609.07	-0.00028	1605.38	64+54.61	1604.41	0.0004	1609.10	3.72	105.3	53.00	158.3
1908+97.00	1609.10	0.00059	1605.41	65+00.00	1604.43	0.0004	1609.12	3.71	105.3	53.00	158.3
1909+00.00	1609.10	0.00059	1605.41	65+03.00	1604.43	0.0004	1609.12	3.71	105.3	53.00	158.3
1909+39.73	1609.12	0.00058	1605.43	65+42.73	1604.45	0.0004	1609.14	3.71	105.2	53.00	158.2

OLD.12i



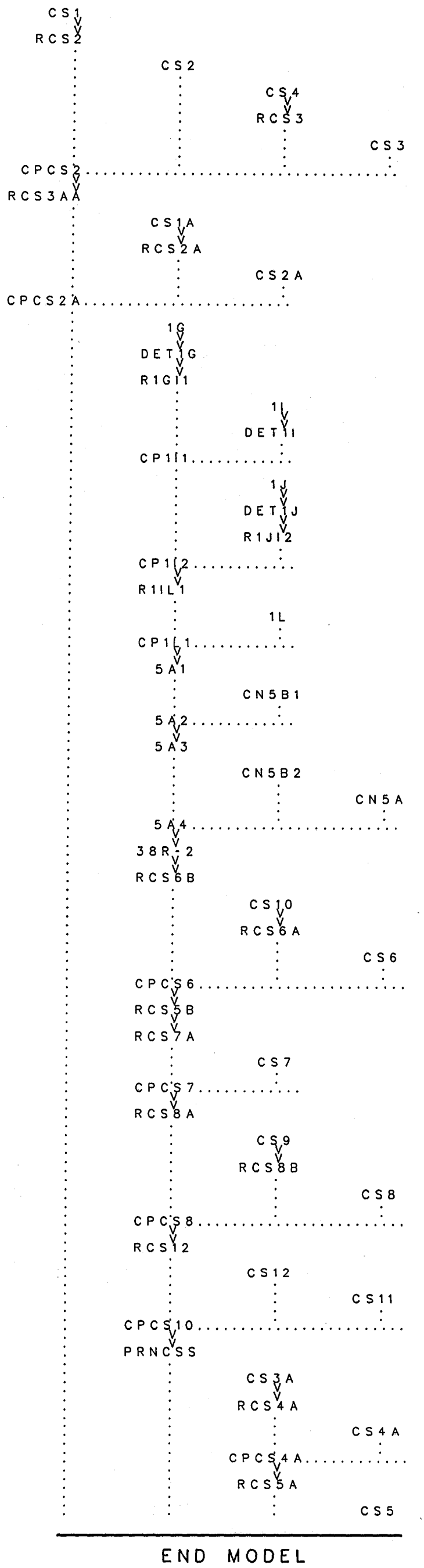
END MODEL

PENTACORE
ARIZONA
CIVIL ENGINEERING LAND SURVEYING PLANNING
CONSTRUCTION MANAGEMENT ADA CONSULTING
2255 N. 44th STREET, SUITE 255
PHOENIX, ARIZONA 85008 (602) 6819272

CITY OF SCOTTSDALE
OUTER LOOP DRAINAGE SYSTEM
HEC-1 MODEL OLD.12i

FIGURE A2-1

OLD.13i



END MODEL

PENTACORE
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CONSTRUCTION MANAGEMENT ADA CONSULTING
2255 N. 44th STREET, SUITE 255
PHOENIX, ARIZONA 85008 (602) 6819272

CITY OF SCOTTSDALE
OUTER LOOP DRAINAGE SYSTEM
HEC-1 MODEL OLD.13i

FIGURE A2-2

```
ID    MODEL OLD.12i        PENTACORE ARIZONA   CJH  11/7/97
ID
ID    HYDROLOGY ANALYSIS FOR OUTER LOOP DRAINAGE SYSTEM
ID
ID    CORE NORTH AREA (BETWEEN HAYDEN AND SCOTTSDALE ROAD NORTH OF THE
ID         PROPOSED PIMA FREEWAY) MODELED FROM HYDROLOGY MODEL PREPARED BY
ID         ROBERT WARD FOR 2002 LAND-USE CONDITIONS - GHFR1.24I
ID
ID ***************************************************************************
ID ***** THE FOLLOWING ARE NOTES FROM ROBERT WARD'S HYDROLOGY MODEL *********
ID ***************************************************************************
ID
ID    HYDROLOGY ANALYSIS FOR RISK ASSESSMENT TO PIMA FREEWAY
ID    FROM SCOTTSDALE ROAD TO PIMA ROAD
ID
ID    PREPARED BY ROBERT L. WARD, P.E. -  OCTOBER 1995
ID
ID    MODEL GHFR1.24I
ID
ID ***************** 2002 LAND-USE CONDITION *******************
ID
ID    100-YEAR, 24-HOUR HYPOTHETICAL STORM
ID
ID    UIBW CURVE NUMBERS FOR ALL SUB-BASINS
ID
ID    RAINFALL DATA TAKEN FROM PREFRE OUTPUT SUMMARY PREPARED BY DEI
ID    PROFESSIONAL SERVICES AND PUBLISHED IN THE DRAINAGE PLAN FOR GRAYHAWK
ID    VILLAGES 2 & 3, REVISED 3/27/95.
ID
ID ******************************************************************************
ID
ID    PIMA ROAD CHANNEL & POWERLINE CORRIDOR CHANNEL ARE IN-PLACE
ID
ID    SUB-BASINS CN5A, CN5B1, CN5B2, CN5C, CN6A, CN7A, & CN7B ARE INCLUDED
ID    ALONG THE NORTH SIDE OF THE FREEWAY.  THESE BASINS ARE ASSUMMED TO BE
ID    DEVELOPED TO DENSITIES CONSISTENT WITH THE 1992 UPPER INDIAN BEND WASH
ID    REGIONAL PLAN HEC-1 MODEL MP3.24I.  CN7A & CN 7B ARE ASSUMED TO BE
ID    "GENERAL INDUSTRIAL" ZONING AS SHOWN ON THE 1987 SCOTTSDALE GENERAL
ID    LAND-USE PLAN.
ID
ID
ID    NOTE: ALL HEC-1 DATA FOR SUB-BASINS 1G THROUGH 1L ARE TAKEN FROM
ID        GILBERTSON ASSOCIATES MODEL UDPLAN.DAT, AS PUBLISHED IN THE
ID        AUGUST 1995 REPORT TITLED "UNIT DRAINAGE PLAN, GRAYHAWK VILLAGE 1"
ID        THE GILBERTSON MODEL USED SCS UNIT HYDROGRAPH METHODOLOGY.
ID
ID        THE CITY OF SCOTTSDALE HAS REQUIRED GILBERTSON ASSOCIATES TO RE-DO
ID        THIS MODEL WITH KINEMATIC WAVE METHOGOLOGY IN ORDER TO BE
ID        CONSISTENT WITH ALL PREVIOUS MODELING EFFORTS FOR THIS AREA.
ID        AS OF 10/18/95, GILBERTSON ASSOCIATES HAS NOT COMPLIED WITH THIS
ID        REQUEST.  ACCORDINGLY, THE UNIT HYDROGRAPH DATA WILL BE USED FOR
ID        THE REFERENCED SUB-BASINS UNTIL THIS REVISED MODELING DATA IS
ID        AVAILABLE.
ID
ID ****************************************************************************
ID ************************** END ROBERT WARD NOTES ***************************
ID ****************************************************************************
ID
ID CORE SOUTH MODELED BY PENTACORE BASED ON:
ID    CITY OF SCOTTSDALE ZONING AND LAND-USE MAPPING
ID    CITY OF SCOTTSDALE DESIGN STANDARDS - DRAINAGE JULY 1996
ID    CITY OF SCOTTSDALE DIGITAL TERRAIN MAPPING
ID    CITY OF SCOTTSDALE PARCEL MAPPING
ID    GENERAL CONCEPT DRAINAGE PLAN - SLA REPORT 1987 (FOR STATE TRUST LANDS)
ID
ID
*DIAGRAM
```

```
IT   5 7NOV97        300
IO   5    0
*
*
KK  CS1
KM Basin runoff calculation for CS1
BA .0327
PH        10   .74   1.45   2.47   2.75   2.94   3.30   3.74   4.17
LS        77    85
UK  250 0.0140  0.045    100
RK 1400 0.0130  0.035        TRAP     5     1
RK  350 0.0044  0.025        TRAP    10     2
KK RCS2
KM  ROUTE FLOW THROUGH SUBBASIN CS2
RS   1   FLOW    -1
RC .025   .025   .025    682 0.0023
RX   0    10    20    28    38    46    60    80
RY   4     4     4     0     0     4     4     4
KK  CS2
KM Basin runoff calculation for CS2
BA .0410
LS        77    85
UK  200 0.0130  0.045    100
RK 1650 0.0128  0.035        TRAP     5     1
KK  CS4
KM Basin runoff calculation for CS4
BA .0498
LS        77    85
UK  350 0.0140  0.045    100
RK  200 0.0112  0.035  0.029   TRAP    10     1
RK  600 0.0055  0.025        TRAP    10     2
KK RCS3
KM  ROUTE FLOW THROUGH SUBBASIN CS3
RS   1   FLOW    -1
RC .025   .025   .025    437 0.0041
RX   0    10    20    28    38    46    60    80
RY   4     4     4     0     0     4     4     4
KK  CS3
KM Basin runoff calculation for CS3
BA .0494
LS        77    85
UK  200 0.0110  0.045    100
RK 1700 0.0133  0.035        TRAP     5     1
KK CPCS2
HC   4
KKRCS3AA
KM  ROUTE FLOW THROUGH SUBBASIN CS3A
RS   1   FLOW    -1
RC .025   .025   .025    800 0.0133
RX   0    10    20    28    38    46    60    80
RY   4     4     4     0     0     4     4     4
KK  CS1A
KM Basin runoff calculation for CS1A
BA .0183
LS        77    85
UK  250 0.0140  0.045    100
RK  529 0.0130  0.035        TRAP     5     1
RK  550 0.0040  0.025        TRAP    10     2
KK RCS2A
KM  ROUTE FLOW THROUGH SUBBASIN CS2A
RS   1   FLOW    -1
RC .025   .025   .025    435 0.0065
RX   0    10    20    28    38    46    60    80
RY   4     4     4     0     0     4     4     4
KK  CS2A
KM Basin runoff calculation for CS2A
BA .0196
```

```
LS        77    85
UK  200 0.0130  0.045    100
RK  750 0.0128  0.035        TRAP     5     1
KKCPCS2A
HC    3
KK   1G
KM     EXCESS RUNOFF FROM SUBAREA 1G
BA  .059
LS        74    53
UD  .06
*
KK DET1G
KM     DETENTION FOR PARCEL 1G
RS    1   STOR     0
SV    0    4.8
SQ    0     26
SE    0     3
ST    3    15    2.8    1.5
*
KK R1GI1
KM     ROUTE DET1G TO THOMPSON PEAK PARKWAY
KM     TRAPEZOIDAL CHANNEL PER 76TH STREET PLANS
KM                SIDE SLOPE - 3:1 (H:V)
KM                MAX DEPTH  - 1.6 FT.
KM                BOT.WIDTH  - 8.0 FT.
RS    1   FLOW    -1
RC 0.025  0.025  0.025   1400  0.017
RX    0    0.5    1.0    5.8   13.8   18.6    19   19.5
RY  1.8    1.7    1.6     0     0    1.6    1.7    1.8
*
KK   1I
KM     EXCESS RUNOFF FROM SUBAREA 1I
BA  .062
LS        74    52
UD  .07
*
KK DET1I
KM     DETENTION FOR SUBAREA 1I
RS    1   STOR     0
SV    0    4.0
SQ    0     39
SE    0     3
ST    3    15    2.8    1.5
*
KK CP1I1
KM     COMBINE HYDROGRAPHS FROM R1G1I AND DET1I
HC    2
*
KK   1J
KM     EXCESS RUNOFF FROM SUBAREA 1J
BA .0175
LS        74    64
UD  .06
*
KK DET1J
KM     DETENTION FOR SUBAREA 1J
RS    1   STOR     0
SV    0    2.1
SQ    0     11
SE    0     3
ST    3    15    2.8    1.5
*
KK R1JI2
KM     ROUTE DET1J TO 76TH STREET (WEST)
KM     TRIANGULAR CHANNEL PER PLANS (1661-E-94)
KM                SIDE SLOPE - 4:1 (H:V)
KM                MAX DEPTH  - 3.0 FT.
```

```
RS   1   FLOW    -1
RC .025   .025   .025   1400   .005
RX   0    0.5    1.0   12.9    13    25   25.5    26
RY  3.2   3.1    3.0     0     0   3.0    3.1   3.2
*
KK CP1I2
KM    COMBINE HYDROGRAPHS FROM DET1J AND 1I
HC   2
*
KK R1IL1
KM    ROUTE 1I TO 1L
KM    TRAPEZOIDAL CHANNEL PER 76TH STREET PLANS
KM              SIDE SLOPE - 3:1 (H:V)
KM              MAX DEPTH  - 3.5 FT.
KM              BOT.WIDTH  - 8.0 FT.
RS   1   FLOW    -1
RC  .03    .03    .03    730   .015
RX   0    0.5    1.0   11.5   19.5    30   30.5    31
RY  3.7   3.6    3.5     0     0   3.5    3.6   3.7
*
KK   1L
KM    EXCESS RUNOFF FROM SUBAREA 1L
BA .0484
LS        74    64
UD .217
*
KK CP1L1
KM    COMBINE HYDROGRAPHS FROM 1L AND R1IL1
HC   2
*
*
*    END OF GILBERTSON ASSOCIATES SUB-BASINS
*    BEGIN KINEMATIC WAVE SIMULATIONS FOR REMAINING SUB-BASINS
*
*
KK  5A1    CP
KM    ROUTE CP 1L1 TO CP 5A1
RS   2   FLOW    -1
RC .020   .020   .020   2500  .0150
RX   0     1     2    22    42    62    63    64
RY  10    10    10     0     0    10    10    10
*
KK CN5B1    SUB
KM    RUNOFF FROM SUB CN5B1
BA .2583
LS        74   64.5
UK  100  .0150   .15    100
RK 3500  .0133   .020  .1292   TRAP    10     2
RK 2500  .0150   .020         TRAP    20     2
*
KK  5A2    CP
KM    COMBINE SUB CN5B1 & CP 5A1
HC   2
*
KK  5A3    CP
KM    ROUTE CP 5A2 TO DETENTION BASIN 38R-2
RS   2   FLOW    -1
RC .020   .020   .020   1500  .0150
RX   0     1     2    22    42    62    63    64
RY  10    10    10     0     0    10    10    10
*
KK CN5B2    SUB
BA .2066
LS        74   72.8
UK  100  .0150   .15    100
RK 1300  .0071   .020  .0413   TRAP    10     2
RK 3500  .0150   .020         TRAP    20     2
```

```
*
KK CN5A    SUB
KM     RUNOFF FROM SUB CN5A
BA .2985
LS         74   71.5
UK  100  .0150   .15    100
RK 1600  .0050   .020  .0637   TRAP    10     2
RK 4900  .0152   .020         TRAP    20     2
*
KK  5A4    CP
KM     COMBINE CP 5A3, SUB CN5A, & SUB CN5B2
HC    3
*
KK 38R-2    DAM
KO    3
KM     PERFORM RESERVOIR ROUTING OPERATION FOR DETENTION BASIN NO. 2 (38R-2)
KM     USE RESERVOIR DATA PROVIDED BY CITY OF SCOTTSDALE
KM     OUTLET - 1-72" CONCRETE PIPE
RS    1   STOR
SV    0   0.05   0.39   1.90   4.80   8.20  11.90  19.90  28.70  33.40
SV 38.30  43.30  48.60  54.00  59.70  65.60
SQ    0    16    52   127   216   274   291   342   390   412
SQ  435   451   468   486   500   512
SE 1590  1591  1592  1593  1594  1595  1596  1598  1600  1601
SE 1602  1603  1604  1605  1606  1607
ST 1607   600   2.9   1.5
KK RCS6B
KM ROUTE OUTFLOW FROM DETENTION BASIN THROUGH PRINCIPAL OUTLET TO
KM INTERSECTION OF UNION HILLS ROAD AND 76TH STREET (MILLER)
KM USE KINEMATIC WAVE ROUTING FOR 72" DIAMETER CONCRETE PIPE CULVERT
KO    1
RK 1600 0.0108   .015         CIRC     6
KK CS10
KM Basin runoff calculation for CS10
KO    1
BA .0343
LS         77    23
UK  150 0.0140  0.045    100
RK  750 0.0142  0.035         TRAP    10     1
RK  600 0.0020  0.025         TRAP    10     2
KK RCS6A
KM   ROUTE FLOW THROUGH SUBBASIN CS6
RS    1   FLOW    -1
RC .025   .025   .025   1600 0.0057
RX    0    10    20    28    38    46    60    80
RY    4     4     4     0     0     4     4     4
KK  CS6
KM Basin runoff calculation for CS6
KO    1
BA .0455
LS         77    85
UK  250 0.0140  0.045    100
RK 1450 0.0142  0.035         TRAP    10     1
KK CPCS6
KM COMBINE RUNOFF FROM CS6 WITH OUTFLOW FROM DETENTION BASIN 38R-2
HC    3
KK RCS5B
KM ROUTE RUNOFF FROM CONCENTRATION POINT CPCS6 THROUGH CS5
KM USE KINEMATIC WAVE ROUTING FOR 84" DIAMETER CONCRETE PIPE CULVERT
KO    1
RK 2000 0.0108   .013         CIRC     7
KK  CS3A
KM Basin runoff calculation for CS3A
BA .0139
LS         77    85
UK  200 0.0110  0.045    100
RK  800 0.0133  0.035         TRAP     5     1
```

```
RK  180 0.0029  0.025        TRAP    10    2
KK RCS4A
KM  ROUTE FLOW THROUGH SUBBASIN CS4A
RS   1  FLOW    -1
RC .025  .025  .025   800 0.0050
RX   0    10    20    28    38    46    60    80
RY   4     4     4     0     0     4     4     4
KK  CS4A
KM Basin runoff calculation for CS4A
BA .0339
LS        77    85
UK  350 0.0140  0.045   100
RK 1000 0.0112  0.035        TRAP    10    1
KKCPCS4A
HC   2
KK RCS5A
KM  ROUTE FLOW THROUGH SUBBASIN CS5
RS   1  FLOW    -1
RC .025  .025  .025   800 0.0050
RX   0    10    20    28    38    46    60    80
RY   4     4     4     0     0     4     4     4
KK  CS5
KM Basin runoff calculation for CS5
BA .0476
LS        77    85
UK  350 0.0140  0.045   100
RK 1600 0.0148  0.035        TRAP    10    1
KK CPCS5
KM COMBINE RUNOFF FROM CS5, RCS5A, AND RCS5B
HC   3
KK RCS7A
KM ROUTE RUNOFF FROM CONCENTRTION POINT CPCS5 THROUGH CS7
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS   1  FLOW    -1
RC .045  .035  .045   400 0.0009
RX   0    10    20    52    77   109   119   219
RY 110   109   108   100   100   108   109   110
KK  CS7
KM Basin runoff calculation for CS7
BA .0723
LS        77    85
UK  300 0.0140  0.045   100
RK 2700 0.0153  0.035        TRAP     5    1
KK CPCS7
KM COMBINE RUNOFF FROM CPCS5 WITH RUNOFF FROM CS7
HC   2
KK RCS8A
KM ROUTE RUNOFF FROM CONCENTRTION POINT CPCS7 THROUGH CS8
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS   1  FLOW    -1
RC .045  .035  .045   500 0.0009
RX   0    10    20    52    77   109   119   219
RY 110   109   108   100   100   108   109   110
KK  CS9
KM Basin runoff calculation for   CS9
BA .0352
LS        77    85
UK  300 0.0140  0.045   100
RK  500 0.0140  0.035 0.00838  TRAP     5    1
RK 3000 0.0130  0.035        TRAP     5    1
KK RCS8B
KM ROUTE CS9 THROUGH CS8
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=5' Z=1
RS   1  FLOW    -1
RC .045  .035  .045   500 0.0073
```

```
RX   0    10    20    22    27    29    39    49
RY  104   103   102   100   100   102   103   104
KK  CS8
KM Basin runoff calculation for CS8
BA .0480
LS        77    85
UK  250 0.0150  0.045   100
RK 3800 0.0137  0.035       TRAP    15     2
KK CPCS8
KM COMBINE RUNOFF FROM CPCS7, CS9 WITH RUNOFF FROM CS8
HC   3
KK RCS12
KM ROUTE RUNOFF FROM CONCENTRATION POINT CPCS8 TO PRINCESS BOULEVARD
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS   1   FLOW    -1
RC .045   .035   .045   400 0.0009
RX   0    10    20    52    77   109   119   219
RY  110   109   108   100   100   108   109   110
KK  CS12
KM Basin runoff calculation for CS12
BA .0126
LS        77    85
UK   50 0.0150  0.045   100
RK  850 0.0089  0.035       TRAP    15     2
KK  CS11
KM Basin runoff calculation for CS11
BA .0035
LS        77    10
UK   50 0.0150  0.045   100
RK  400 0.0089  0.035       TRAP    15     2
KKCPCS10
KM COMBINE RUNOFF FROM CPCS8 WITH RUNOFF FROM CS10
HC   3
KKPRNCSS
KM ROUTE RUNOFF THROUGH PRINCESS BOULEVARD
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS   1   FLOW    -1
RC .045   .035   .045    50 0.0009
RX   0    10    20    52    77   109   119   219
RY  110   109   108   100   100   108   109   110
ZZ
```

```
ID    MODEL OLD.13i        PENTACORE ARIZONA   CJH  11/7/97
ID
ID    HYDROLOGY ANALYSIS FOR OUTER LOOP DRAINAGE SYSTEM
ID
ID    CORE NORTH AREA (BETWEEN HAYDEN AND SCOTTSDALE ROAD NORTH OF THE
ID         PROPOSED PIMA FREEWAY) MODELED FROM HYDROLOGY MODEL PREPARED BY
ID         ROBERT WARD FOR 2002 LAND-USE CONDITIONS - GHFR1.24I
ID
ID ************************************************************************
ID ***** THE FOLLOWING ARE NOTES FROM ROBERT WARD'S HYDROLOGY MODEL *********
ID ************************************************************************
ID
ID    HYDROLOGY ANALYSIS FOR RISK ASSESSMENT TO PIMA FREEWAY
ID    FROM SCOTTSDALE ROAD TO PIMA ROAD
ID
ID    PREPARED BY ROBERT L. WARD, P.E. -  OCTOBER 1995
ID
ID    MODEL GHFR1.24I
ID
ID ***************** 2002 LAND-USE CONDITION *******************
ID
ID    100-YEAR, 24-HOUR HYPOTHETICAL STORM
ID
ID    UIBW CURVE NUMBERS FOR ALL SUB-BASINS
ID
ID    RAINFALL DATA TAKEN FROM PREFRE OUTPUT SUMMARY PREPARED BY DEI
ID    PROFESSIONAL SERVICES AND PUBLISHED IN THE DRAINAGE PLAN FOR GRAYHAWK
ID    VILLAGES 2 & 3, REVISED 3/27/95.
ID
ID ****************************************************************************
ID
ID    PIMA ROAD CHANNEL & POWERLINE CORRIDOR CHANNEL ARE IN-PLACE
ID
ID    SUB-BASINS CN5A, CN5B1, CN5B2, CN5C, CN6A, CN7A, & CN7B ARE INCLUDED
ID    ALONG THE NORTH SIDE OF THE FREEWAY.  THESE BASINS ARE ASSUMMED TO BE
ID    DEVELOPED TO DENSITIES CONSISTENT WITH THE 1992 UPPER INDIAN BEND WASH
ID    REGIONAL PLAN HEC-1 MODEL MP3.24I.  CN7A & CN 7B ARE ASSUMED TO BE
ID    "GENERAL INDUSTRIAL" ZONING AS SHOWN ON THE 1987 SCOTTSDALE GENERAL
ID    LAND-USE PLAN.
ID
ID
ID    NOTE: ALL HEC-1 DATA FOR SUB-BASINS 1G THROUGH 1L ARE TAKEN FROM
ID        GILBERTSON ASSOCIATES MODEL UDPLAN.DAT, AS PUBLISHED IN THE
ID        AUGUST 1995 REPORT TITLED "UNIT DRAINAGE PLAN, GRAYHAWK VILLAGE 1"
ID        THE GILBERTSON MODEL USED SCS UNIT HYDROGRAPH METHODOLOGY.
ID
ID        THE CITY OF SCOTTSDALE HAS REQUIRED GILBERTSON ASSOCIATES TO RE-DO
ID        THIS MODEL WITH KINEMATIC WAVE METHOGOLOGY IN ORDER TO BE
ID        CONSISTENT WITH ALL PREVIOUS MODELING EFFORTS FOR THIS AREA.
ID        AS OF 10/18/95, GILBERTSON ASSOCIATES HAS NOT COMPLIED WITH THIS
ID        REQUEST.  ACCORDINGLY, THE UNIT HYDROGRAPH DATA WILL BE USED FOR
ID        THE REFERENCED SUB-BASINS UNTIL THIS REVISED MODELING DATA IS
ID        AVAILABLE.
ID
ID ***************************************************************************
ID ************************* END ROBERT WARD NOTES **************************
ID ***************************************************************************
ID
ID CORE SOUTH MODELED BY PENTACORE BASED ON:
ID    CITY OF SCOTTSDALE ZONING AND LAND-USE MAPPING
ID    CITY OF SCOTTSDALE DESIGN STANDARDS - DRAINAGE JULY 1996
ID    CITY OF SCOTTSDALE DIGITAL TERRAIN MAPPING
ID    CITY OF SCOTTSDALE PARCEL MAPPING
ID    GENERAL CONCEPT DRAINAGE PLAN - SLA REPORT 1987 (FOR STATE TRUST LANDS)
ID
```

```
ID  REVISIONS TO CORE SOUTH ROUTING SCHEME INCLUDED
ID     DRAINAGE AREAS EAST OF 76TH STREET ALIGNMENT ROUTED DIRECTLY SOUTH
ID     TO THE CAP DETENTION BASIN
ID
ID
*DIAGRAM
IT    5 21OCT95          300
IO    5     0
*
*
KK  CS1
KM Basin runoff calculation for CS1
BA .0327
PH          10    .74   1.45   2.47   2.75   2.94   3.30   3.74   4.17
LS          77    85
UK  250  0.0140  0.045    100
RK  1400  0.0130  0.035          TRAP     5     1
RK  350  0.0044  0.025          TRAP    10     2
KK  RCS2
KM   ROUTE FLOW THROUGH SUBBASIN CS2
RS    1   FLOW    -1
RC .025   .025   .025    682  0.0023
RX    0    10    20    28    38    46    60    80
RY    4     4     4     0     0     4     4     4
KK  CS2
KM Basin runoff calculation for CS2
BA .0410
LS          77    85
UK  200  0.0130  0.045    100
RK  1650  0.0128  0.035          TRAP     5     1
KK  CS4
KM Basin runoff calculation for CS4
BA .0498
LS          77    85
UK  350  0.0140  0.045    100
RK  200  0.0112  0.035  0.029   TRAP    10     1
RK  600  0.0055  0.025          TRAP    10     2
KK  RCS3
KM   ROUTE FLOW THROUGH SUBBASIN CS3
RS    1   FLOW    -1
RC .025   .025   .025    437  0.0041
RX    0    10    20    28    38    46    60    80
RY    4     4     4     0     0     4     4     4
KK  CS3
KM Basin runoff calculation for CS3
BA .0494
LS          77    85
UK  200  0.0110  0.045    100
RK  1700  0.0133  0.035          TRAP     5     1
KK CPCS2
HC    4
KKRCS3AA
KM   ROUTE FLOW THROUGH SUBBASIN CS3A
RS    1   FLOW    -1
RC .025   .025   .025    800  0.0133
RX    0    10    20    28    38    46    60    80
RY    4     4     4     0     0     4     4     4
KK  CS1A
KM Basin runoff calculation for CS1A
BA .0183
LS          77    85
UK  250  0.0140  0.045    100
RK  529  0.0130  0.035          TRAP     5     1
RK  550  0.0040  0.025          TRAP    10     2
KK RCS2A
```

```
KM   ROUTE FLOW THROUGH SUBBASIN CS2A
RS    1   FLOW     -1
RC .025   .025   .025    435  0.0065
RX    0    10    20    28    38    46    60    80
RY    4     4     4     0     0     4     4     4
KK  CS2A
KM Basin runoff calculation for CS2A
BA .0196
LS          77    85
UK  200 0.0130  0.045    100
RK  750 0.0128  0.035          TRAP     5     1
KKCPCS2A
HC    3
KK    1G
KM     EXCESS RUNOFF FROM SUBAREA 1G
BA  .059
LS          74    53
UD   .06
*
KK DET1G
KM     DETENTION FOR PARCEL 1G
RS    1   STOR     0
SV    0   4.8
SQ    0    26
SE    0     3
ST    3    15    2.8    1.5
*
KK R1GI1
KM     ROUTE DET1G TO THOMPSON PEAK PARKWAY
KM     TRAPEZOIDAL CHANNEL PER 76TH STREET PLANS
KM                SIDE SLOPE - 3:1 (H:V)
KM                MAX DEPTH  - 1.6 FT.
KM                BOT.WIDTH  - 8.0 FT.
RS    1   FLOW     -1
RC 0.025  0.025  0.025   1400  0.017
RX    0    0.5    1.0    5.8   13.8   18.6    19   19.5
RY  1.8    1.7    1.6     0     0    1.6    1.7    1.8
*
KK    1I
KM     EXCESS RUNOFF FROM SUBAREA 1I
BA  .062
LS          74    52
UD   .07
*
KK DET1I
KM     DETENTION FOR SUBAREA 1I
RS    1   STOR     0
SV    0   4.0
SQ    0    39
SE    0     3
ST    3    15    2.8    1.5
*
KK CP1I1
KM     COMBINE HYDROGRAPHS FROM R1G1I AND DET1I
HC    2
*
KK    1J
KM     EXCESS RUNOFF FROM SUBAREA 1J
BA .0175
LS          74    64
UD   .06
*
KK DET1J
KM     DETENTION FOR SUBAREA 1J
RS    1   STOR     0
```

```
SV   0   2.1
SQ   0    11
SE   0    3
ST   3    15    2.8    1.5
*
KK R1JI2
KM     ROUTE DET1J TO 76TH STREET (WEST)
KM     TRIANGULAR CHANNEL PER PLANS (1661-E-94)
KM              SIDE SLOPE - 4:1 (H:V)
KM              MAX DEPTH  - 3.0 FT.
RS   1   FLOW    -1
RC .025   .025   .025   1400   .005
RX   0    0.5    1.0   12.9    13    25   25.5    26
RY  3.2   3.1    3.0     0     0   3.0    3.1   3.2
*
KK CP1I2
KM     COMBINE HYDROGRAPHS FROM DET1J AND 1I
HC   2
*
KK R1IL1
KM     ROUTE 1I TO 1L
KM     TRAPEZOIDAL CHANNEL PER 76TH STREET PLANS
KM              SIDE SLOPE - 3:1 (H:V)
KM              MAX DEPTH  - 3.5 FT.
KM              BOT.WIDTH  - 8.0 FT.
RS   1   FLOW    -1
RC  .03    .03    .03    730   .015
RX   0    0.5    1.0   11.5   19.5    30   30.5    31
RY  3.7   3.6    3.5     0     0   3.5    3.6   3.7
*
KK   1L
KM     EXCESS RUNOFF FROM SUBAREA 1L
BA .0484
LS        74     64
UD  .217
*
KK CP1L1
KM     COMBINE HYDROGRAPHS FROM 1L AND R1IL1
HC   2
*
*
*     END OF GILBERTSON ASSOCIATES SUB-BASINS
*     BEGIN KINEMATIC WAVE SIMULATIONS FOR REMAINING SUB-BASINS
*
*
KK  5A1     CP
KM     ROUTE CP 1L1 TO CP 5A1
RS   2   FLOW    -1
RC .020   .020   .020   2500  .0150
RX   0     1     2     22    42    62    63    64
RY  10    10    10      0     0    10    10    10
*
KK CN5B1    SUB
KM     RUNOFF FROM SUB CN5B1
BA .2583
LS        74   64.5
UK  100  .0150    .15    100
RK 3500  .0133   .020  .1292   TRAP    10     2
RK 2500  .0150   .020         TRAP    20     2
*
KK  5A2     CP
KM     COMBINE SUB CN5B1 & CP 5A1
HC   2
*
KK  5A3     CP
```

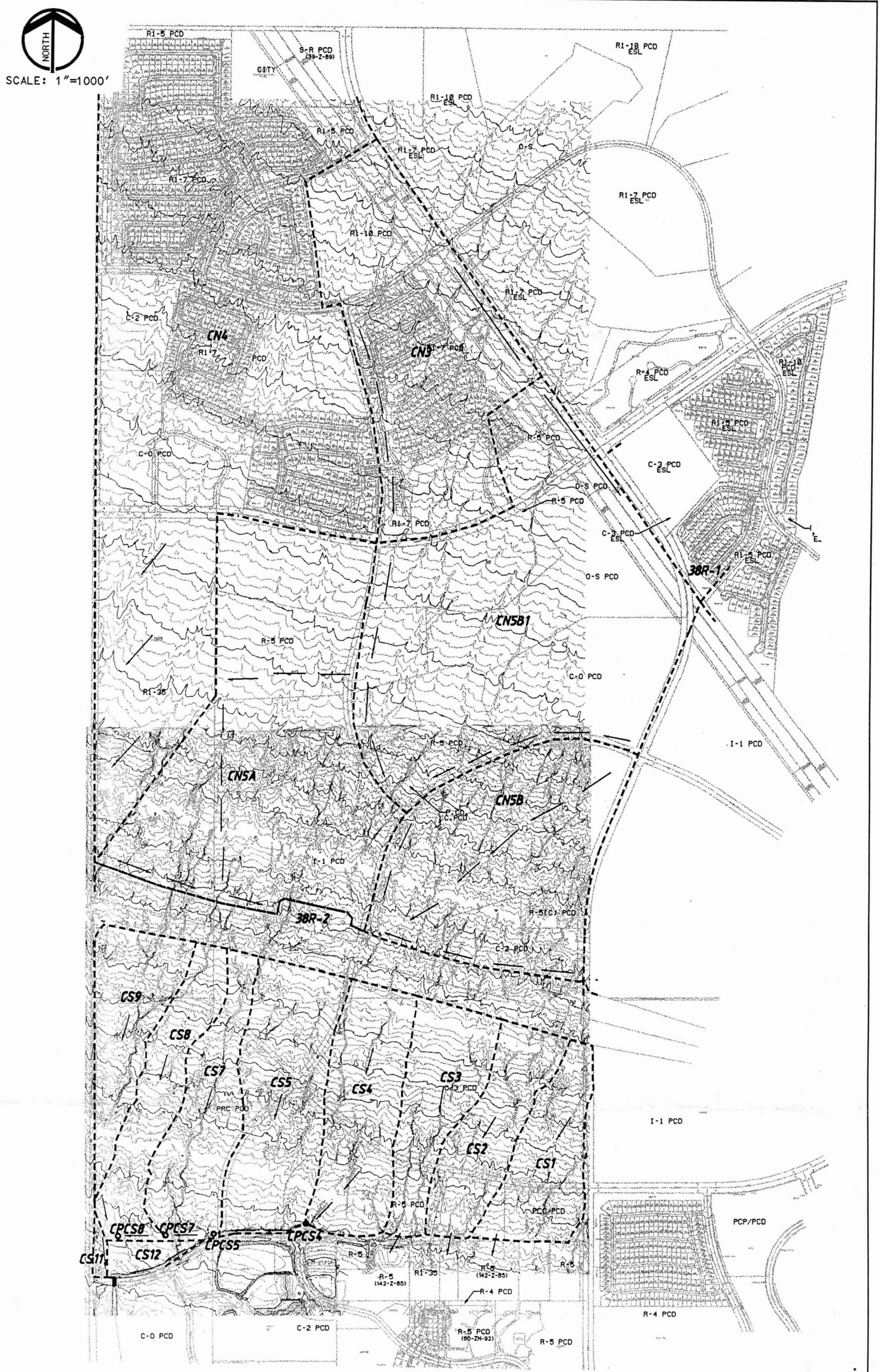
```
KM     ROUTE CP 5A2 TO DETENTION BASIN 38R-2
RS    2   FLOW     -1
RC .020   .020   .020   1500  .0150
RX    0     1     2     22     42     62     63     64
RY   10    10    10      0      0     10     10     10
*
KK CN5B2    SUB
BA .2066
LS          74   72.8
UK  100  .0150    .15    100
RK 1300  .0071   .020  .0413   TRAP     10      2
RK 3500  .0150   .020          TRAP     20      2
*
KK  CN5A    SUB
KM     RUNOFF FROM SUB CN5A
BA .2985
LS          74   71.5
UK  100  .0150    .15    100
RK 1600  .0050   .020  .0637   TRAP     10      2
RK 4900  .0152   .020          TRAP     20      2
*
KK   5A4     CP
KM     COMBINE CP 5A3, SUB CN5A, & SUB CN5B2
HC    3
*
KK 38R-2    DAM
KO    3
KM     PERFORM RESERVOIR ROUTING OPERATION FOR DETENTION BASIN NO. 2 (38R-2)
KM     USE RESERVOIR DATA PROVIDED BY CITY OF SCOTTSDALE
KM     OUTLET - 1-72" CONCRETE PIPE
RS    1   STOR
SV    0   0.05   0.39   1.90   4.80   8.20  11.90  19.90  28.70  33.40
SV 38.30  43.30  48.60  54.00  59.70  65.60
SQ    0     16     52    127    216    274    291    342    390    412
SQ  435    451    468    486    500    512
SE 1590   1591   1592   1593   1594   1595   1596   1598   1600   1601
SE 1602   1603   1604   1605   1606   1607
ST 1607    600    2.9    1.5
KK RCS6B
KM ROUTE OUTFLOW FROM DETENTION BASIN THROUGH PRINCIPAL OUTLET TO
KM INTERSECTION OF UNION HILLS ROAD AND 76TH STREET (MILLER)
KM USE KINEMATIC WAVE ROUTING FOR 72" DIAMETER CONCRETE PIPE CULVERT
KO    1
RK 1600 0.0108   .015          CIRC      6
KK  CS10
KM Basin runoff calculation for CS10
KO    1
BA .0343
LS          77     23
UK  150 0.0140  0.045    100
RK  750 0.0142  0.035          TRAP     10      1
RK  600 0.0020  0.025          TRAP     10      2
KK RCS6A
KM   ROUTE FLOW THROUGH SUBBASIN CS6
RS    1   FLOW     -1
RC .025   .025   .025   1600 0.0057
RX    0     10     20     28     38     46     60     80
RY    4      4      4      0      0      4      4      4
KK   CS6
KM Basin runoff calculation for CS6
KO    1
BA .0455
LS          77     85
UK  250 0.0140  0.045    100
RK 1450 0.0142  0.035          TRAP     10      1
```

```
KK CPCS6
KM COMBINE RUNOFF FROM CS6 WITH OUTFLOW FROM DETENTION BASIN 38R-2
HC   3
KK RCS5B
KM ROUTE RUNOFF FROM CONCENTRATION POINT CPCS6 THROUGH CS5
KM USE KINEMATIC WAVE ROUTING FOR 84" DIAMETER CONCRETE PIPE CULVERT
KO   1
RK 2000 0.0108  .013        CIRC    7
KK RCS7A
KM ROUTE RUNOFF FROM CONCENTRTION POINT CPCS5 THROUGH CS7
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS   1  FLOW   -1
RC .045  .035  .045   400 0.0009
RX   0   10   20   52   77  109  119  219
RY 110  109  108  100  100  108  109  110
KK  CS7
KM Basin runoff calculation for CS7
BA .0723
LS       77   85
UK  300 0.0140  0.045   100
RK 2700 0.0153  0.035        TRAP    5    1
KK CPCS7
KM COMBINE RUNOFF FROM CPCS5 WITH RUNOFF FROM CS7
HC   2
KK RCS8A
KM ROUTE RUNOFF FROM CONCENTRTION POINT CPCS7 THROUGH CS8
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS   1  FLOW   -1
RC .045  .035  .045   500 0.0009
RX   0   10   20   52   77  109  119  219
RY 110  109  108  100  100  108  109  110
KK  CS9
KM Basin runoff calculation for   CS9
BA .0352
LS       77   85
UK  300 0.0140  0.045   100
RK  500 0.0140  0.035 0.00838  TRAP    5    1
RK 3000 0.0130  0.035        TRAP    5    1
KK RCS8B
KM ROUTE CS9 THROUGH CS8
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=5' Z=1
RS   1  FLOW   -1
RC .045  .035  .045   500 0.0073
RX   0   10   20   22   27   29   39   49
RY 104  103  102  100  100  102  103  104
KK  CS8
KM Basin runoff calculation for CS8
BA .0480
LS       77   85
UK  250 0.0150  0.045   100
RK 3800 0.0137  0.035        TRAP   15    2
KK CPCS8
KM COMBINE RUNOFF FROM CPCS7, CS9 WITH RUNOFF FROM CS8
HC   3
KK RCS12
KM ROUTE RUNOFF FROM CONCENTRATION POINT CPCS8 TO PRINCESS BOULEVARD
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS   1  FLOW   -1
RC .045  .035  .045   400 0.0009
RX   0   10   20   52   77  109  119  219
RY 110  109  108  100  100  108  109  110
KK  CS12
```

```
KM Basin runoff calculation for CS12
BA .0126
LS        77    85
UK   50 0.0150  0.045    100
RK  850 0.0089  0.035        TRAP    15     2
KK  CS11
KM Basin runoff calculation for CS11
BA .0035
LS        77    10
UK   50 0.0150  0.045    100
RK  400 0.0089  0.035        TRAP    15     2
KKCPCS10
KM COMBINE RUNOFF FROM CPCS8 WITH RUNOFF FROM CS10
HC    3
KKPRNCSS
KM ROUTE RUNOFF THROUGH PRINCESS BOULEVARD
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS    1   FLOW    -1
RC .045   .035   .045    50 0.0009
RX    0    10    20    52    77   109   119   219
RY  110   109   108   100   100   108   109   110
KK  CS3A
KM Basin runoff calculation for CS3A
BA .0145
LS        77    85
UK  200 0.0110  0.045    100
RK  800 0.0133  0.035        TRAP     5     1
RK  180 0.0029  0.025        TRAP    10     2
KK RCS4A
KM   ROUTE FLOW THROUGH SUBBASIN CS4A
RS    1   FLOW    -1
RC .025   .025   .025   800 0.0050
RX    0    10    20    28    38    46    60    80
RY    4     4     4     0     0     4     4     4
KK  CS4A
KM Basin runoff calculation for CS4A
BA .0399
LS        77    85
UK  350 0.0140  0.045    100
RK 1000 0.0112  0.035        TRAP    10     1
KKCPCS4A
HC    2
KK RCS5A
KM   ROUTE FLOW THROUGH SUBBASIN CS5
RS    1   FLOW    -1
RC .025   .025   .025   800 0.0050
RX    0    10    20    28    38    46    60    80
RY    4     4     4     0     0     4     4     4
KK  CS5
KM Basin runoff calculation for CS5
BA .0406
LS        77    85
UK  350 0.0140  0.045    100
RK 1600 0.0148  0.035        TRAP    10     1
ZZ
```

```
ID    MODEL OLD.14i        PENTACORE ARIZONA   MPB  11/7/97
ID
ID    HYDROLOGY ANALYSIS FOR RISK ASSESSMENT TO PRINCESS BLVD. CULVERTS
ID
ID    FROM SCOTTSDALE ROAD TO HAYDEN ROAD FOR CORE SOUTH AREA SOUTH OF
ID    THE PROPOSED ADOT OUTER LOOP FREEWAY
ID
ID
ID ***************** EXISTING LAND-USE CONDITION *******************
ID
ID    100-YEAR, 6-HOUR HYPOTHETICAL STORM
ID
ID    SOIL TYPE "B" FOR ENTIRE DRAINAGE AREA
ID
ID    CURVE NUMBERS BASED ON THE CITY OF SCOTTSDALE DESIGN MANUAL AND
ID    CONSISTENT WITH VALUES USED FOR CORE NORTH UNDEVELOPED DESERT CN = 74
ID
ID    PARTIAL AREA EAST OF THE 76TH STREET ALIGNMENT DRAINS INTO AN UNLINED
ID    EARTHEN CHANNEL AND IS CONVEYED WESTERLY TO THE EXISTING CULVERTS AT
ID    PRINCESS BOULEVARD NEAR SCOTTSDALE ROAD
ID
ID ******************************************************************************
ID
*DIAGRAM
IT    5 21OCT95          300
IO    1     0
*
*
KK  CS4
KM Basin runoff calculation for CS4
BA .0801
PH              .74   1.45   2.47   2.75   2.94   3.30
LS         74
UK  350 0.0140  0.045    100
RK 1600 0.0112  0.035  0.029   TRAP     10      1
RK 2800 0.0136  0.035         TRAP     10      1
KK RCS5A
KM ROUTE RUNOFF FROM CONCENTRTION POINT CPCS4 THROUGH CS5
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=15' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 4' (EXISTING)
RS    1   FLOW    -1
RC .045   .035   .045   1000 0.0080
RX    0    100    125    141    156    172    197    297
RY105.25 104.25    104    100    100    104 104.25 105.25
KK  CS5
KM Basin runoff calculation for CS5
BA .0958
LS         74
UK  300 0.0140  0.035    100
RK 1750 0.0142  0.035     42   TRAP     25      2
RK 2300 0.0148  0.035         TRAP     25      2
KK CPCS5
KM COMBINE RUNOFF FROM CS5 AND CPCS4
HC    2
KK RCS7A
KM ROUTE RUNOFF FROM CONCENTRTION POINT CPCS5 THROUGH CS7
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8' (PROPOSED OPEN CHANNEL IMPROVEMENTS)
RS    2   FLOW    -1
RC .045   .025   .045    800 0.0009
RX    0     10     20     52     77    109    119    219
RY  110    109    108    100    100    108    109    110
KK  CS7
KM Basin runoff calculation for CS7
BA .0687
```

```
LS        74
UK  300 0.0140  0.045   100
RK 2700 0.0153  0.035        TRAP    5    1
KK CPCS7
KM COMBINE RUNOFF FROM CPCS5 WITH RUNOFF FROM CS7
HC   2
KK  CS9
KM Basin runoff calculation for   CS9
BA .0585
LS        74
UK  300 0.0140  0.045   100
RK  500 0.0140  0.035 0.00838   TRAP    5    1
RK 3000 0.0130  0.035        TRAP    5    1
KK RCS8B
KM ROUTE CS9 THROUGH CS8
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=5' Z=1
KM NATURAL CHANNEL UNIMPROVED
RS   1  FLOW   -1
RC .045  .035  .045   400 0.0073
RX   0   10   20   22   27   29   39   49
RY  104  103  102  100  100  102  103  104
KK  CS8
KM Basin runoff calculation for CS8
BA .0595
LS        74
UK  250 0.0150  0.045   100
RK 3800 0.0137  0.035        TRAP   15    2
KK CPCS8
KM COMBINE RUNOFF FROM CPCS7, CS9 WITH RUNOFF FROM CS8
HC   3
KK RCS12
KM ROUTE RUNOFF FROM CONCENTRTION POINT CPCS8 TO PRINCESS BOULEVARD
KM USE 8-POINT TRAPEZOIDAL CHANNEL ROUTING B=25' Z=4
KM MAIN CHANNEL DEPTH APPROXIMATELY 8'
KM SOIL CEMENT LINED PROPOSED OPEN CHANNEL IMPROVEMENTS
RS   1  FLOW   -1
RC .045  .019  .045   400 0.0080
RX   0   10   20   52   77  109  119  219
RY  110  109  108  100  100  108  109  110
KK  CS12
KM Basin runoff calculation for CS12
BA .0104
LS        74
UK   50 0.0150  0.045   100
RK  850 0.0089  0.035        TRAP   15    2
KK  CS11
KM Basin runoff calculation for CS11
BA .0039
LS        74
UK   50 0.0150  0.045   100
RK  400 0.0089  0.035        TRAP   15    2
KKPRNCSS
KM COMBINE RUNOFF FROM CS11, CS12 AND CPCS8
KM REPRESENTS THE FLOW AT PRINCESS BOULEVARD BOX CULVERTS
HC   3
ZZ
```



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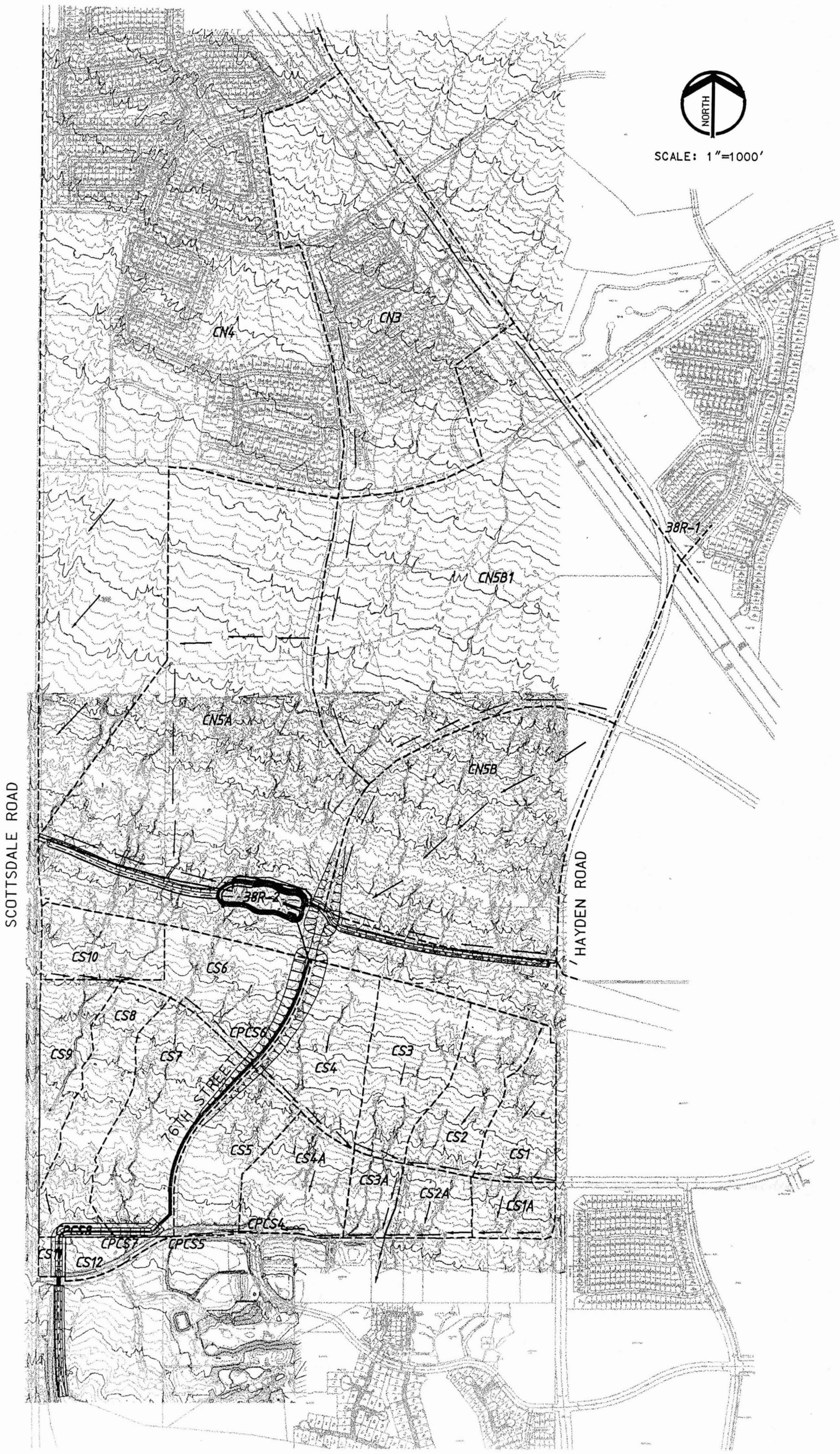
CITY OF SCOTTSDALE
OUTER LOOP DRAINAGE SYSTEM
EXISTING CORE SOUTH DRAINAGE PLAN

FIGURE A3-1



PENTACORE
ARIZONA
CIVIL ENGINEERING LAND SURVEYING PLANNING
CONSTRUCTION MANAGEMENT ADA CONSULTING
2255 N. 44th STREET, SUITE 255
PHOENIX, ARIZONA 85008 (602) 6819272

CITY OF SCOTTSDALE
OUTER LOOP DRAINAGE SYSTEM
DRAINAGE PLAN

FIGURE A3-2

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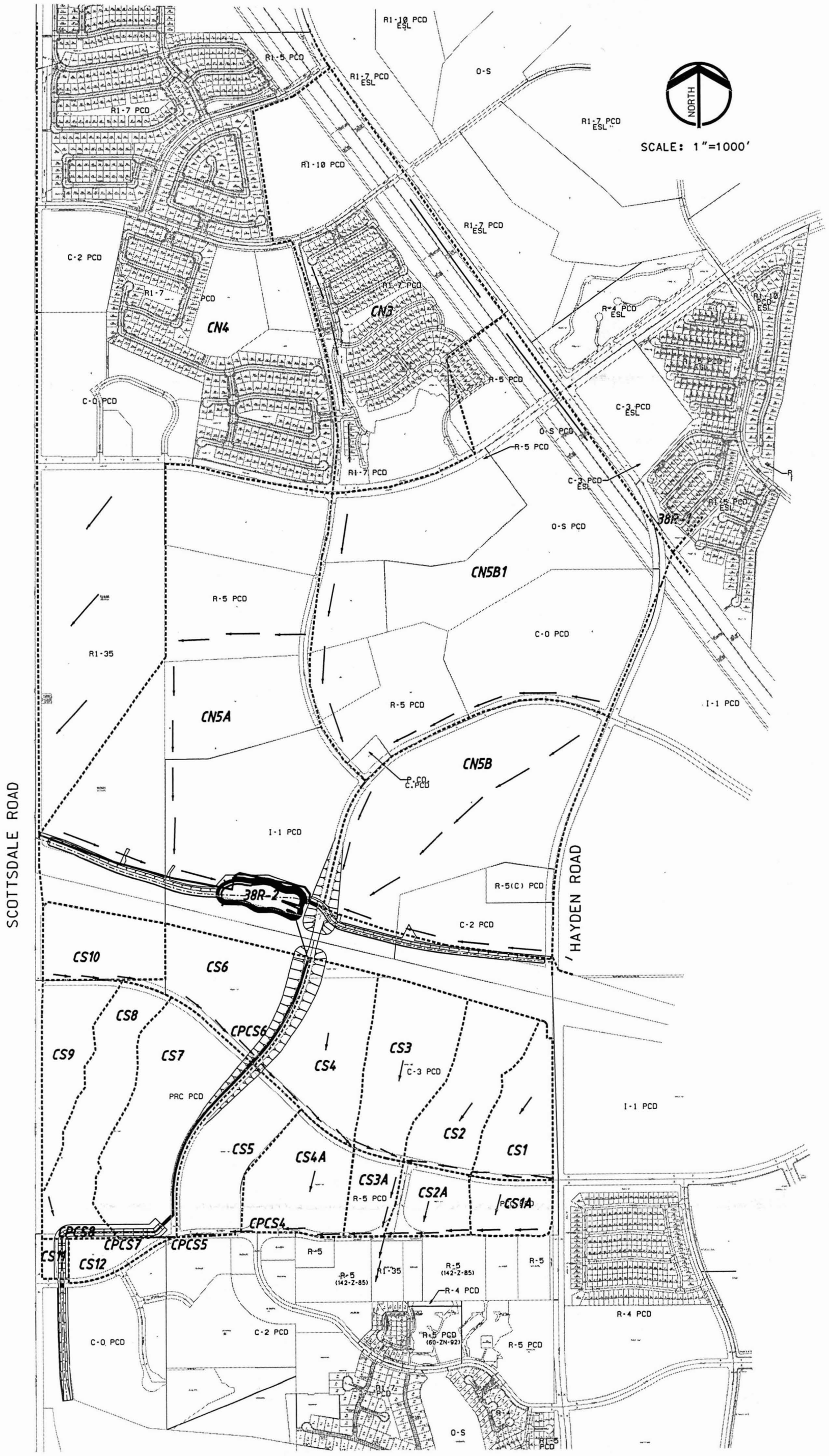

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CITY OF SCOTTSDALE
OUTER LOOP DRAINAGE SYSTEM
PLANNED LAND USE

FIGURE A3-3

Outer Loop Drainage System
Hydrology Subbasin Characteristics

Drainage Basin	Percent Impervious	CN (1) Number	Zoning Abrev.	Area (sf)	Area (Acres)	Area (sq.mi.)
CS1	85	77	C-3 PCD	910845	20.9	0.0327
CS1A	85	77	PCC PCD	511511	11.7	0.0183
CS2	85	77	C-3 PCD	1144091	26.3	0.0410
CS2A	85	77	R-5 PCD	545622	12.5	0.0196
CS3	85	77	C-3 PCD	1377542	31.6	0.0494
CS3A	85	77	R-5 PCD	403991	9.3	0.0145
CS4	85	77	C-3 PCD	1390097	31.9	0.0498
CS4A	85	77	R-5 PCD	1111505	25.5	0.0399
CS5	85	77	PRC PCD	1132162	26.0	0.0406
CS6	85	77	PRC PCD	1268034	29.1	0.0455
CS7	85	77	PRC PCD	2015951	46.3	0.0723
CS8	85	77	PRC PCD	1338107	30.7	0.0480
CS9	85	77	PRC PCD	982363	22.6	0.0352
CS10	23	77	R1-35	955775	21.9	0.0343
CS11	10	77	O-S	96519	2.2	0.0035
CS12	85	77	PRC PCD	352172	8.1	0.0126

(1) Western desert urban areas, Soil Group "B"
PCD, Planned Community Development (commercial)
PRC, Planned Regional Center (commercial)
PCC, Planned Community Center (commercial)
R1-35, Residential (35,000 sf/lot)
R-5, Apartments and Condominiums
C-3,
O-S, Open Space (golf courses)

% Impervious	Zoning	Description
85	PRC, PCD, and PCC	Commercial areas
85	R-3, R-5	Multi family
23	R1-35	Large Lots
10	OS	Improved open space

Outer Loop Drainage System
Precipitation Data

*	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U
6															
7	***PIMA ROAD HYDROLOGY (FROM NOAA AND COS DESIGN STANDARDS)***														
8	***Rainfall Depth (inches)***														
9															
10	STORM FREQUENCY	*2-year*		*5-year*		*10-year*		*25-year*		*50-year*		*100-year*		*500-year*	
11	Storm Duration														
12	5-min	$P_{2,5'} = 0.32$		$P_{5,5'} = 0.42$		$P_{10,5'} = 0.49$		$P_{25,5'} = 0.58$		$P_{50,5'} = 0.65$		$P_{100,5'} = 0.73$		$P_{500,5'} = 0.90$	
13	10-min	$P_{2,10'} = 0.48$		$P_{5,10'} = 0.63$		$P_{10,10'} = 0.74$		$P_{25,10'} = 0.89$		$P_{50,10'} = 1.00$		$P_{100,10'} = 1.12$		$P_{500,10'} = 1.38$	
14	15-min	$P_{2,15'} = 0.58$		$P_{5,15'} = 0.79$		$P_{10,15'} = 0.93$		$P_{25,15'} = 1.13$		$P_{50,15'} = 1.28$		$P_{100,15'} = 1.43$		$P_{500,15'} = 1.78$	
15	30-min	$P_{2,30'} = 0.77$		$P_{5,30'} = 1.06$		$P_{10,30'} = 1.25$		$P_{25,30'} = 1.52$		$P_{50,30'} = 1.73$		$P_{100,30'} = 1.94$		$P_{500,30'} = 2.42$	
16	1-hour	$P_{2,1'} = 0.94$		$P_{5,1'} = 1.31$		$P_{10,1'} = 1.56$		$P_{25,1'} = 1.90$		$P_{50,1'} = 2.16$		$P_{100,1'} = 2.42$		$P_{500,1'} = 3.03$	
17	2-hour	$P_{2,2'} = 1.08$		$P_{5,2'} = 1.49$		$P_{10,2'} = 1.76$		$P_{25,2'} = 2.14$		$P_{50,2'} = 2.43$		$P_{100,2'} = 2.72$		$P_{500,2'} = 3.40$	
18	3-hour	$P_{2,3'} = 1.17$		$P_{5,3'} = 1.60$		$P_{10,3'} = 1.89$		$P_{25,3'} = 2.30$		$P_{50,3'} = 2.61$		$P_{100,3'} = 2.92$		$P_{500,3'} = 3.64$	
19	6-hour	$P_{2,6'} = 1.35$		$P_{5,6'} = 1.83$		$P_{10,6'} = 2.15$		$P_{25,6'} = 2.60$		$P_{50,6'} = 2.95$		$P_{100,6'} = 3.30$		$P_{500,6'} = 4.10$	
20	12-hour	$P_{2,12'} = 1.58$		$P_{5,12'} = 2.11$		$P_{10,12'} = 2.48$		$P_{25,12'} = 2.99$		$P_{50,12'} = 3.39$		$P_{100,12'} = 3.78$		$P_{500,12'} = 4.69$	
21	24-hour	$P_{2,24'} = 1.80$		$P_{5,24'} = 2.40$		$P_{10,24'} = 2.81$		$P_{25,24'} = 3.38$		$P_{50,24'} = 3.82$		$P_{100,24'} = 4.26$		$P_{500,24'} = 5.28$	
22															

Outer Loop Drainage System
Hydrology Model Output Comparisons

	A	B	C	D	E	F
4	OLD.12i	General Plan Routing Configuration				
5	OLD.13i	Modified Core South Routing Configuration				
6						
7						
8						
9		OLD.12i		OLD.13i		
10		Peak Flow (cfs)	Time of Peak	Peak Flow (cfs)	Time of Peak	Δ Peak Flow (cfs)
11	CS1	131	12.08	131	12.08	0
12	RCS2	119	12.17	119	12.17	0
13	CS2	176	12.08	176	12.08	0
14	CS4	205	12.08	205	12.08	0
15	RCS3	178	12.08	178	12.08	0
16	CS3	208	12.08	208	12.08	0
17	CPCS2	656	12.08	656	12.08	0
18	RCS3AA	625	12.17	625	12.17	0
19	CS1A	76	12.08	76	12.08	0
20	RCS2A	66	12.17	66	12.17	0
21	CS2A	90	12.08	90	12.08	0
22	CPCS2A	743	12.17	743	12.17	0
23	1G	184	12.08	184	12.08	0
24	DET1G	26	12.58	26	12.58	0
25	R1GI1	26	12.58	26	12.58	0
26	1I	177	12.08	177	12.08	0
27	DET1I	43	12.42	43	12.42	0
28	CP1I1	68	12.50	68	12.50	0
29	1J	59	12.08	59	12.08	0
30	DET1J	8	12.58	8	12.58	0
31	R1JI2	8	12.67	8	12.67	0
32	CP1I2	76	12.50	76	12.50	0
33	R1IL1	76	12.50	76	12.50	0
34	1L	105	12.25	105	12.25	0
35	CP1L1	166	12.33	166	12.33	0
36	5A1	162	12.33	162	12.33	0
37	CN5B1	834	12.17	834	12.17	0
38	5A2	927	12.17	927	12.17	0
39	5A3	909	12.17	909	12.17	0
40	CN5B2	695	12.17	695	12.17	0
41	CN5A	1017	12.17	1017	12.17	0
42	5A4	2622	12.17	2622	12.17	0
43	38R-2	491	12.67	491	12.67	0
44	RCS6B	491	12.67	491	12.67	0
45	CS10	97	12.08	97	12.08	0
46	RCS6A	78	12.17	78	12.17	0
47	CS6	192	12.08	192	12.08	0
48	CPCS6	581	12.17	581	12.17	0
49	RCS5B	574	12.17	574	12.17	0
50	CS3A	61	12.08	63	12.08	-2
51	RCS4A	51	12.17	53	12.17	-2
52	CS4A	136	12.08	162	12.08	-26

Outer Loop Drainage System
Hydrology Model Output Comparisons

	A	B	C	D	E	F
4	OLD.12i	General Plan Routing Configuration				
5	OLD.13i	Modified Core South Routing Configuration				
6						
7						
8						
9		OLD.12i		OLD.13i		
10		Peak Flow (cfs)	Time of Peak	Peak Flow (cfs)	Time of Peak	Δ Peak Flow (cfs)
53	CPCS4A	179	12.08	207	12.08	-28
54	RCS5A	172	12.17	197	12.17	-25
55	CS5	182	12.08	154	12.08	28
56	CPCS5	898	12.17			898
57	RCS7A	900	12.17	577	12.25	323
58	CS7	265	12.08	265	12.08	0
59	CPCS7	1141	12.17	798	12.17	343
60	RCS8A	1087	12.17	765	12.17	322
61	CS9	130	12.17	130	12.17	0
62	RCS8B	120	12.17	120	12.17	0
63	CS8	165	12.17	165	12.17	0
64	CPCS8	1371	12.17	1050	12.17	321
65	RCS12	1335	12.25	1024	12.25	311
66	CS12	61	12.08	61	12.08	0
67	CS11	11	12.08	11	12.08	0
68	CPCS10	1356	12.25	1045	12.25	311
69	PRNCSS	1361	12.25	1048	12.25	313

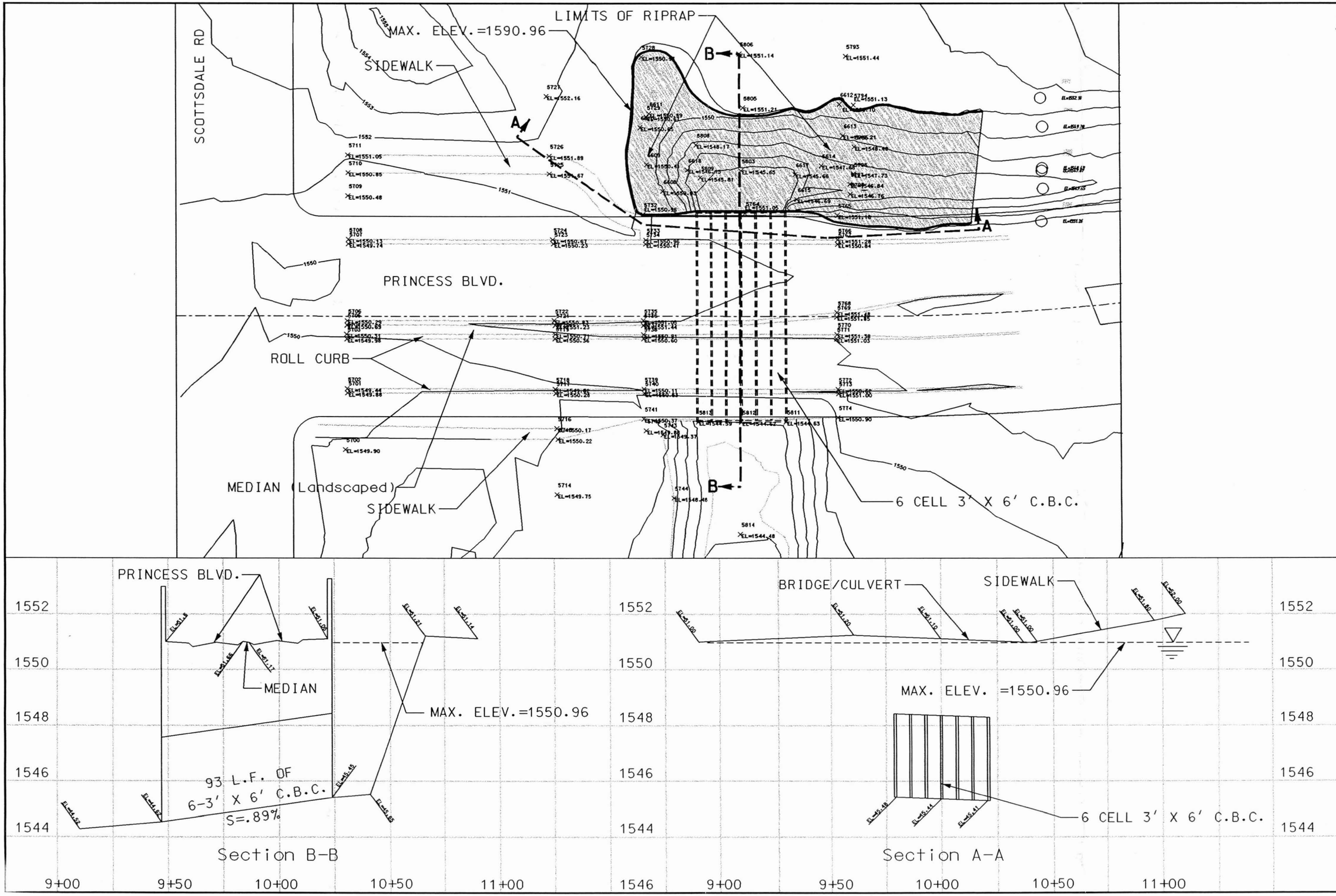

FIGURE
A4-1

CITY OF SCOTTSDALE
OUTER LOOP DRAINAGE SYSTEM
PRINCESS BLVD. CULVERT CROSSING

PENTACORE
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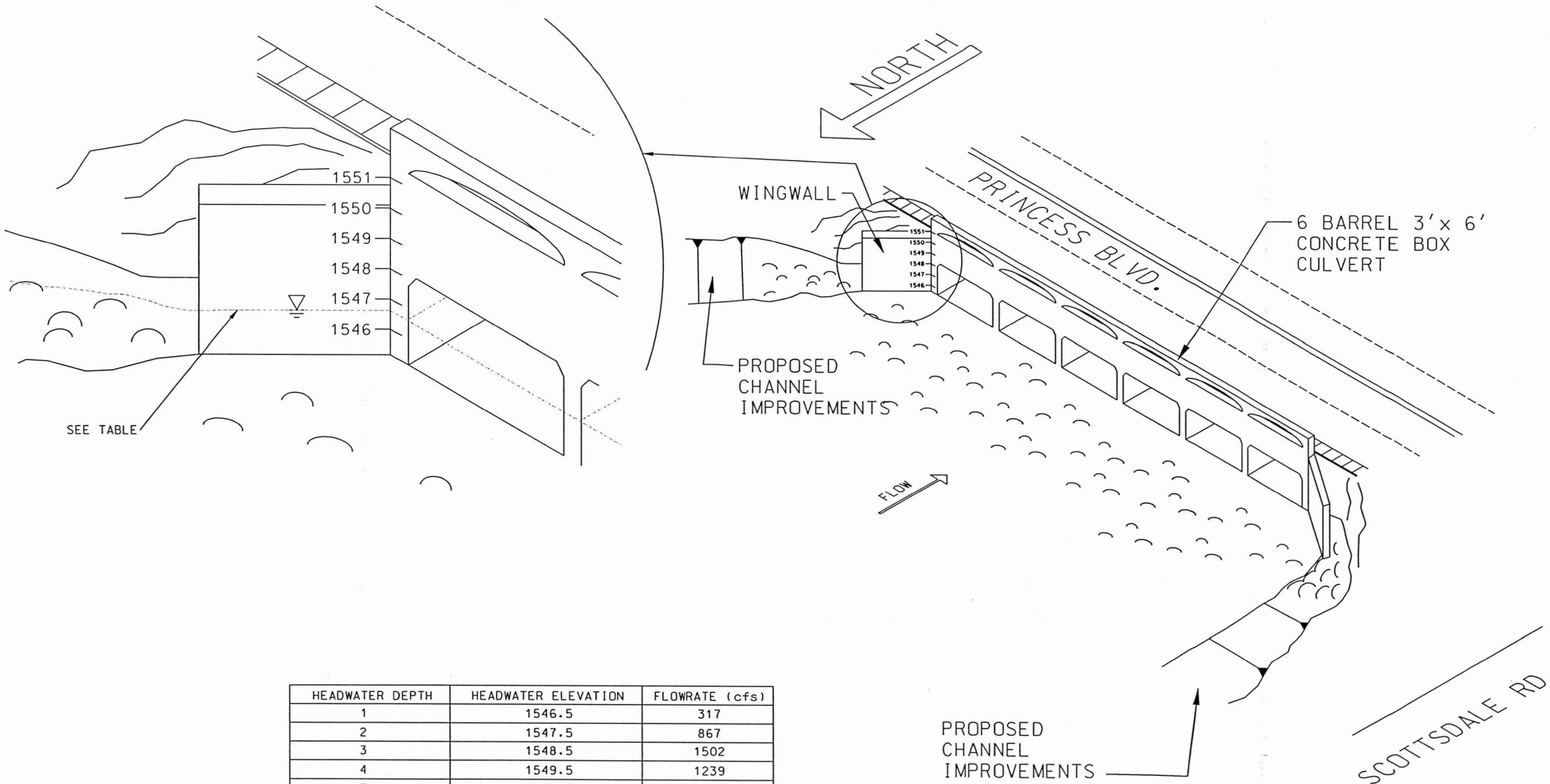

HEADWATER DEPTH	HEADWATER ELEVATION	FLOWRATE (cfs)
1	1546.5	317
2	1547.5	867
3	1548.5	1502
4	1549.5	1239
5	1550.5	1420
5.6	1551.0	1510

FIGURE A4-2

OUTER LOOP DRAINAGE SYSTEM
PRINCESS BLVD. CULVERT CROSSING

PENTACORE
ARIZONA
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Land Surveying · Planning
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Outer Loop Drainage System
Storm Drain Outlet Hyraulics

	A	B	C	D	E	F	G	H	I	J	K	L
1	Junction	Pipe	Q	HGL Up Dn (ft)	EGL Up Dn (ft)	Ground Up Dn (ft)	Slope Energy Constructed (ft)	Section Discharge Capacity (cfs)	Section Shape Size	Length (ft)	V avg (fps)	Dn Cover (ft)
2												
3	I-12	P-14	191	1,579.8	1,581.8	1,588.0	0.0080	191	Circular	100	11.58	5.0
4	J-1			1,578.9	1,581.0	1,585.0	0.0080	202	60 inch			
5		P-9	495	1,605.7	1,612.4	1,620.0	0.0289	495	Circular	457	20.83	10.0
6	I-8			1,592.5	1,599.2	1,602.0	0.0077	255	66 inch			
7		P-8	495	1,590.8	1,594.1	1,602.0	0.0096	495	Circular	400	13.90	9.0
8	I-7			1,587.5	1,590.2	1,597.0	0.0100	554	84 inch			
9		P-7	495	1,586.8	1,590.1	1,597.0	0.0096	495	Circular	400	13.90	7.5
10	I-6			1,583.5	1,586.2	1,591.5	0.0100	554	84 inch			
11		P-6	495	1,582.8	1,586.1	1,591.5	0.0095	495	Circular	400	15.37	5.0
12	J-1			1,578.2	1,582.3	1,585.0	0.0100	554	84 inch			
13		P-5	686	1,574.7	1,579.0	1,585.0	0.0107	686	Circular	500	16.61	8.7
14	I-4			1,569.4	1,573.7	1,579.0	0.0107	688	90 inch			
15		P-4	686	1,569.4	1,573.7	1,579.0	0.0107	686	Circular	447	16.61	8.5
16	I-3			1,564.6	1,568.9	1,574.0	0.0107	688	90 inch			
17		P-3	686	1,564.6	1,568.9	1,574.0	0.0107	686	Circular	450	16.61	6.8
18	I-2			1,559.8	1,564.1	1,567.5	0.0107	689	90 inch			
19		P-2	686	1,559.8	1,564.1	1,567.5	0.0107	686	Circular	450	16.61	6.2
20	I-1			1,555.0	1,559.3	1,562.0	0.0107	688	90 inch			
21		P-1	686	1,555.0	1,559.3	1,562.0	0.0099	686	Circular	148	17.14	5.7
22	Outlet			1,552.9	1,557.8	1,560.0	0.0106	685	90 inch			

Outer Loop Drainage System
HEC-RAS Output
CN5A Collector Channel

Channel Sta.	River Sta.	Q Total (cfs)	Vel Chnl (ft/s)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Max Chl Dpth (ft)	Min Ch El (ft)	W.S. Elev (ft)	Channel Length Downstream (ft)
11+50	1150	1086	3.4	1606.60	0.00028	5.40	1601.01	1606.41	200
13+50	1350	1086	3.4	1606.50	0.00028	5.40	1600.95	1606.35	200
15+50	1550	1086	3.4	1606.50	0.00028	5.40	1600.90	1606.30	200
17+50	1750	1086	3.4	1606.40	0.00028	5.40	1600.84	1606.24	200
19+50	1950	1086	3.4	1606.40	0.00028	5.40	1600.78	1606.18	200
21+50	2150	1086	3.4	1606.30	0.00028	5.40	1600.73	1606.13	200
23+50	2350	1086	3.4	1606.30	0.00028	5.40	1600.67	1606.07	200
25+50	2550	1086	3.4	1606.20	0.00028	5.40	1600.62	1606.02	200
27+50	2750	1086	3.4	1606.20	0.00028	5.40	1600.56	1605.96	200
29+50	2950	1086	3.4	1606.10	0.00028	5.40	1600.51	1605.91	20
29+70	2970	1086	3.5	1606.00	0.00028	5.40	1600.50	1605.90	

Outer Loop Drainage System
HEC-RAS Output
CN5B Collector Channel

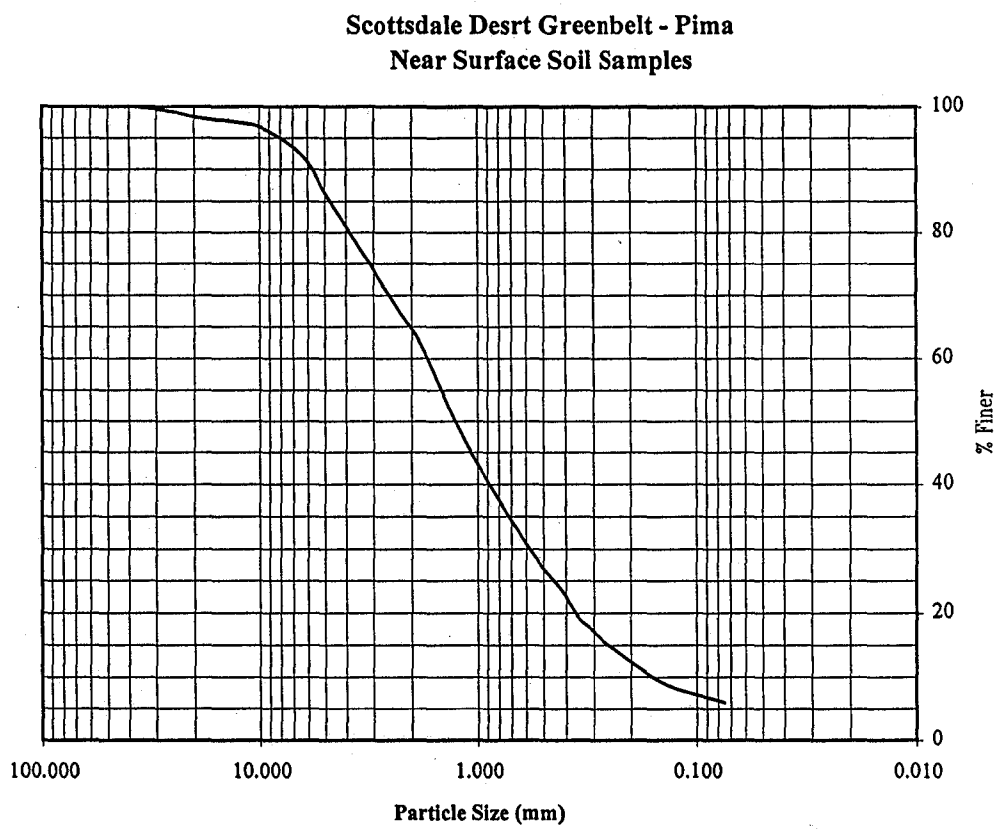
Channel Sta.	River Sta.	Q Total (cfs)	Vel Chnl (ft/s)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Max Chl Dpth (ft)	Min Ch El (ft)	W.S. Elev (ft)	Channel Length Downstream (ft)
64+50	6450	695	3.8	1609.30	0.00042	4.63	1604.47	1609.10	150
63+00	6300	695	3.8	1609.24	0.00041	4.65	1604.39	1609.04	170
61+30	6130	695	3.8	1609.17	0.00041	4.66	1604.31	1608.97	130
60+00	6000	695	3.8	1609.11	0.0004	4.69	1604.23	1608.92	200
58+00	5800	695	3.8	1609.03	0.0004	4.69	1604.15	1608.84	200
56+00	5600	695	3.8	1608.95	0.0004	4.69	1604.07	1608.76	200
54+00	5400	695	3.8	1608.87	0.0004	4.69	1603.99	1608.68	200
52+00	5200	695	3.8	1608.79	0.0004	4.69	1603.91	1608.60	200
50+00	5000	695	3.8	1608.71	0.0004	4.69	1603.83	1608.52	200
48+00	4800	695	3.8	1608.63	0.0004	4.69	1603.75	1608.44	200
46+00	4600	695	3.8	1608.55	0.0004	4.69	1603.67	1608.36	200
44+00	4400	695	3.8	1608.47	0.0004	4.69	1603.59	1608.28	200
42+00	4200	695	3.8	1608.39	0.0004	4.69	1603.51	1608.20	200
40+00	4000	695	3.8	1608.31	0.0004	4.69	1603.43	1608.12	165
38+35	3835	695	3.8	1608.25	0.0004	4.69	1603.36	1608.05	

Outer Loop Drainage System
Princess Boulevard Culverts -HEC-RAS Output

Analysis Approach	Q	W.S. Elev.	Q	W.S. Elev.
Special Bridge	1260	1550.62	930	1549.48
Normal Bridge	1240	1550.90	930	1550.92
Special Culvert - Inlet Control	989	1550.90	930	1550.53
Special Culvert - Outlet Control	1124	1550.90	930	1547.04

Outer Loop Drainage System
Statistical Soil Gradation - SW

			Near Surface	Standard Deviation
Sieve	Size (in)	Size (mm)	% Passing	% Passing
200	0.0030	0.075	6	5
100	0.0059	0.150	9	5
50	0.0130	0.330	18	6
40	0.0167	0.425	24	7
30	0.0223	0.567	29	7
16	0.0433	1.100	46	9
10	0.0711	1.806	62	10
8	0.0900	2.286	67	10
4	0.1870	4.750	85	8
1/4"	0.2500	6.350	92	5
3/8"	0.3750	9.525	96	3
1/2"	0.5000	12.700	98	3
3/4"	0.7500	19.050	98	2
1"	1.0000	25.400	99	2
1 1/2"	1.5000	38.100	100	0
2"	2.0000	50.800	100	0
3"	3.0000	76.200	100	0



Outer Loop Drainage System
Sediment Yield Results
Comparison for Wash Load Trap Efficiencies

Full Wash Load Contribution
Calculated Sediment Storage Volumes (acre-ft)

Drainage Basin	Calculation Method	Storm Frequency						Estimated Annual Volume
		100-yr	50-yr	25-yr	10-yr	5-yr	2-yr	
CN5A	Statistical Storms	1.116	0.798	0.534	0.278	0.147	0.037	0.126
CN5B	Statistical Storms	1.876	1.341	0.896	0.466	0.246	0.062	0.211
Miller Basin	Statistical Storms	2.992	2.139	1.430	0.744	0.394	0.099	0.338

Total Storage Volume 4.68
Mean Annual Yield (acre-ft/sq mi)* = 0.36

33% Wash Load Contribution
Calculated Sediment Storage Volumes (acre-ft)

Drainage Basin	Calculation Method	Storm Frequency						Estimated Annual Volume
		100-yr	50-yr	25-yr	10-yr	5-yr	2-yr	
CN5A	Statistical Storm Distribution	0.567	0.404	0.268	0.138	0.072	0.017	0.063
CN5B	Statistical Storm Distribution	0.955	0.680	0.448	0.230	0.120	0.029	0.105
Miller Basin	Statistical Storm Distribution	1.522	1.083	0.716	0.368	0.192	0.047	0.167

Total Storage Volume 2.36
Mean Annual Yield (acre-ft/sq mi)* = 0.18

Neglect Wash Load Contribution
Calculated Sediment Storage Volumes (acre-ft)

Drainage Basin	Calculation Method	Storm Frequency						Estimated Annual Volume
		100-yr	50-yr	25-yr	10-yr	5-yr	2-yr	
CN5A	Statistical Storm Distribution	0.296	0.209	0.136	0.069	0.035	0.008	0.031
CN5B	Statistical Storm Distribution	0.502	0.354	0.228	0.115	0.058	0.013	0.052
Miller Basin	Statistical Storm Distribution	0.798	0.563	0.364	0.183	0.093	0.021	0.084

Total Storage Volume 1.22
Mean Annual Yield (acre-ft/sq mi)* = 0.09

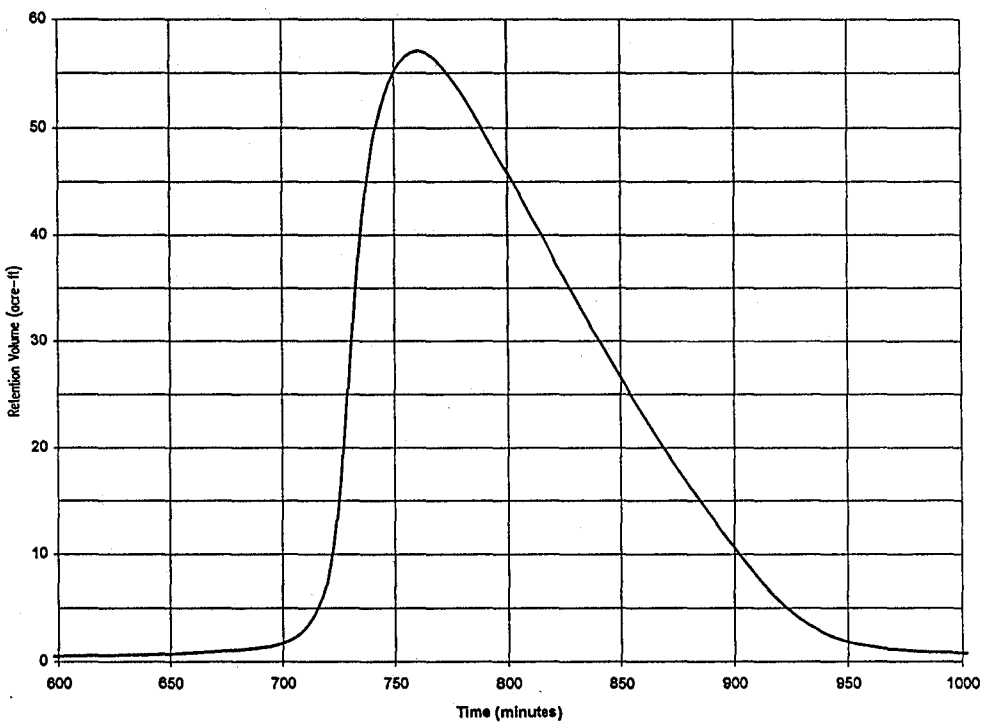
* Drainage area = 0.95 square miles

Outer Loop Drainage System
Hydraulic Retention Times

ω (fps) = 0.0147
#200 Sieve - .075 mm

Total Storage Volume (acre-ft)	Stage (minutes)	Retention Time (minutes)	Average Retention Time (minutes)	Average Fall Depth (ft)	settling time (minutes)	retained volume (acre-ft)
0	0					
0	1500	1500		0		0
5	185					
5	924	739	1120	4.1	4.65	5
10	195					
10	903	708	724	5.49	6.22	10
15	205					
15	885	680	694	6.39	7.24	15
20	727					
20	869	142	411	8.05	9.13	20
25	728					
25	854	126	134	8.58	9.73	25
30	732					
30	841	109	118	10.28	11.66	30
35	733					
35	827	94	102	11.33	12.85	35
40	734					
40	815	81	88	12.34	13.99	40
45	738					
45	802	64	73	13.32	15.10	45
50	742					
50	789	47	56	14.26	16.17	50
55	750					
55	773	23	35	15.18	17.21	55
57	760					
57	760	0	12	15.53	17.61	55

Total Inflow Volume (acre-ft) = 166.9
rap Efficiency - % Retained = 33%



D=3'
B=3'
n=6'
y=5.4'

V=15-22 fps (upstream approach)

V≈15-22 fps

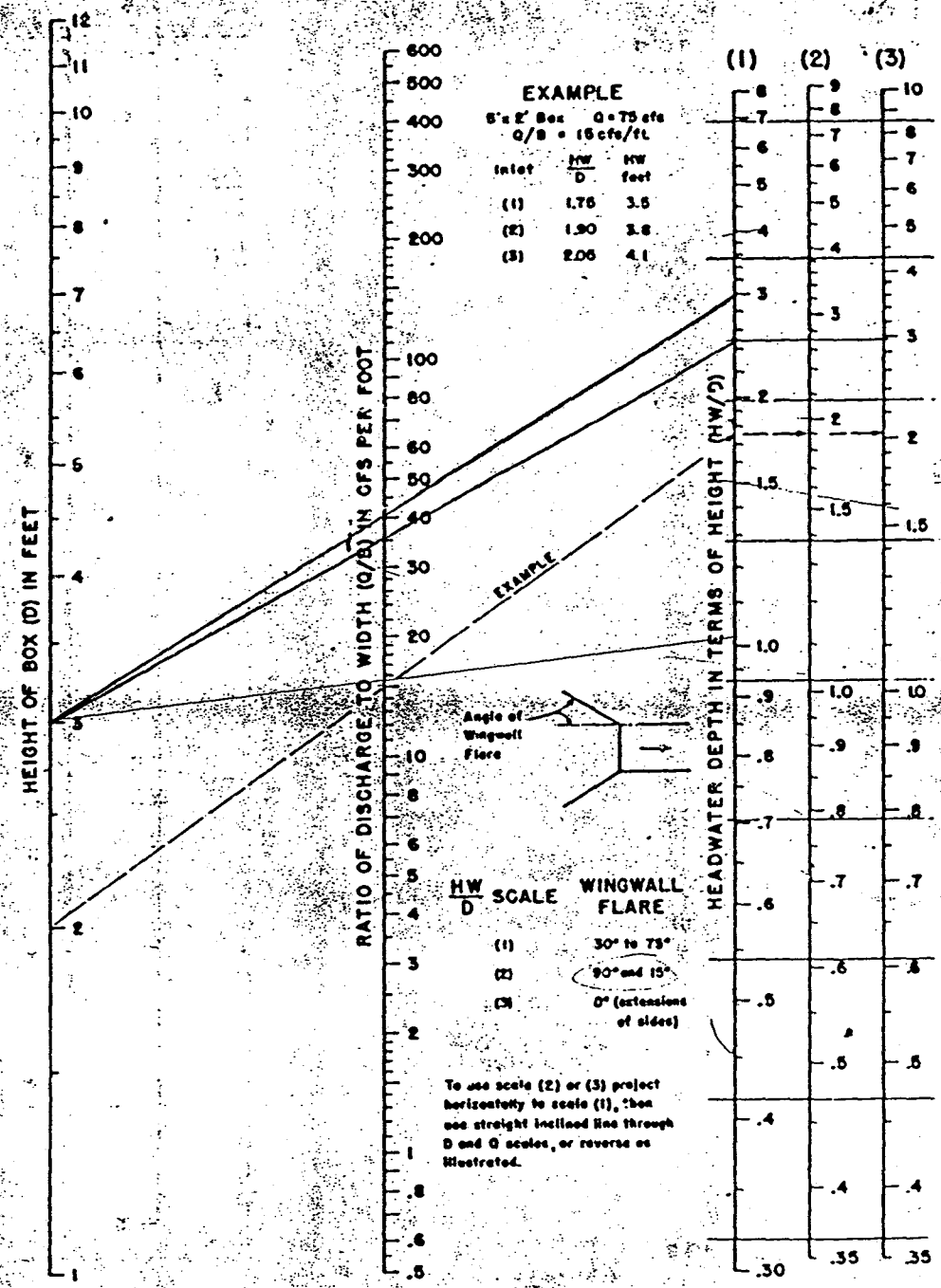
$= 5.4 + \frac{15^2}{64.4}$

Hw = 8.9

CHART #1

MMB 10/17/97

CHART I



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

$35 \leq Q_1 \leq 40$

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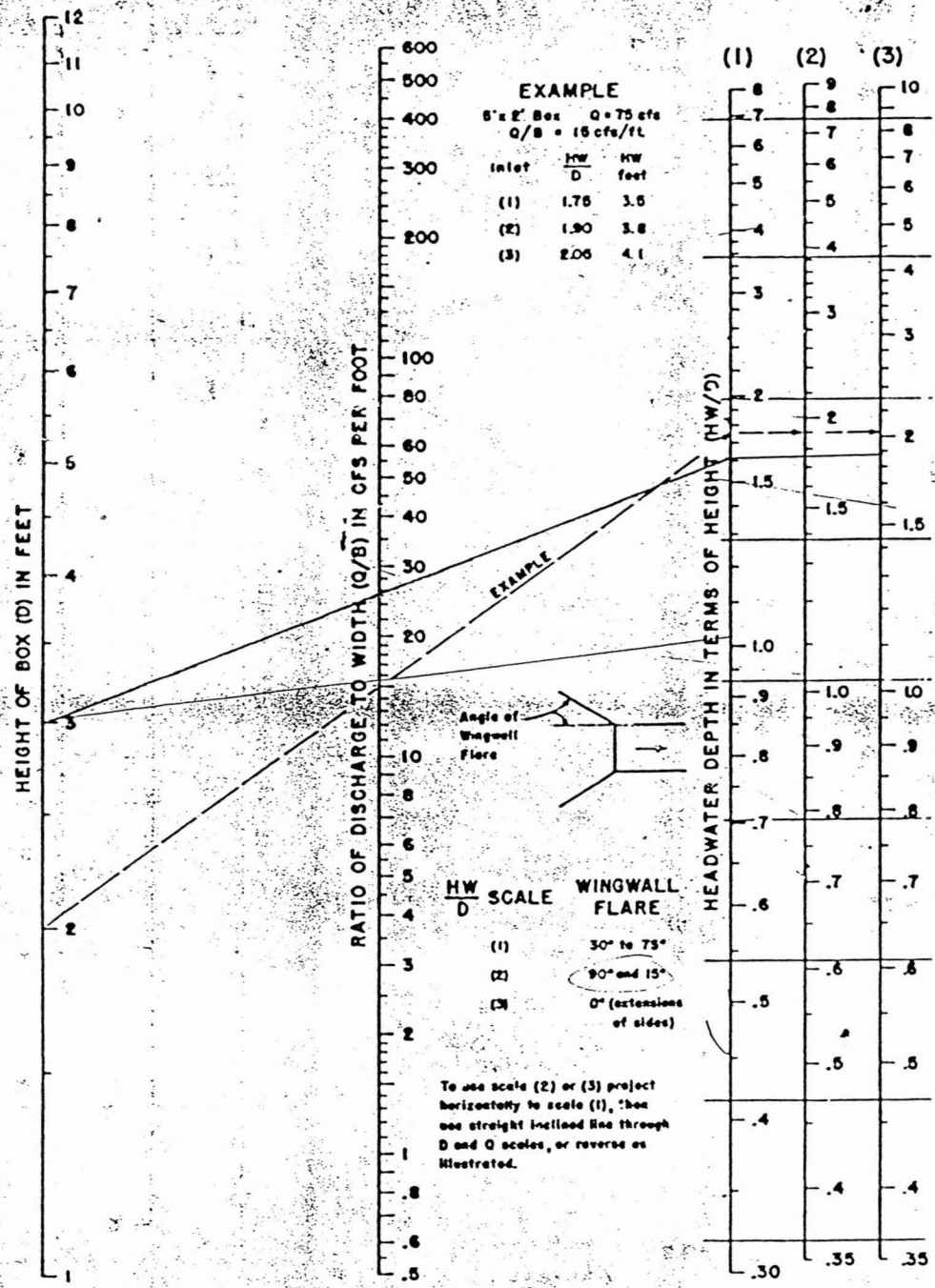
$1260 \leq Q_{total} \leq 1440$

Q = 930 cfs B = 6
Q1 = 155 cfs Q/B = 25.8

CHART #2

MPB 10/17/97

CHART I



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

$1.6 \leq \frac{HW}{D} \leq 1.9$

$4.8 \leq HW \leq 5.7$

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Outer Loop Drainage System
Cost Estimate

11/26/97

Qtally01

Sheet Number		Ch. Exc.	2' S. C.	8' S. C.	2'x3' Cutoff	3'x6' Cutoff	Reveg.	Salvage	Basin Perim Landscaping	Dissipator	66" Pipe	84" Pipe	90" Pipe	Junction Structure	Std. M.H.	90" Pipe Endwall	66" Pipe Outlt. Wks.	6' Fence	6' Gates	Riprap
GENERAL																			3	
1																				
2																				
3		132,207	444				307,108	352,100	68,800									1,100		1,875
4		7,407	4,207		1,600	70	32,000	96,000												
5		15,392	3,309		1,269	219	36,000	116,940												
6		28,373	676		250		133,040	133,040		170			80			1				
7							30,000	30,000					1,000		3					
8							30,000	30,000				85	915	1	1					
9							30,000	30,000				1,000			2					
10							15,150	15,750			457	116		1			1			
11		9,870	244	1,637	150		76,950	61,000										700		
12		16,370	570	2,518	350		114,000	120,000										1,000		
13		2,960	185	428		70	13,110	16,200										120		
14		8,229	346	2,268			81,880	89,300										910		
15		9,778	163	2,370			89,000	95,000										1,000		
16		7,797	163	1,541			59,185	63,175										650		
TOTALS		238,383	10,309	10,762	3,619	359	1,047,423	1,268,505	68,800	170	457	1,201	1,995	2	6	1	1	5,480	3	1,875
UNITS		CY	CY	CY	LF	LF	SF	SF	SF	CY	LF	LF	LF	EA	EA	EA	EA	LF	EA	CY
Unit Cost		$ 3.00	$ 45.00	$ 35.00	$ 15.00	$ 30.00	$ 0.05	$ 0.50	$ 1.00	$ 230.00	$ 175.00	$ 140.00	$ 170.00	$ 15,000.00	$ 2,500.00	$ 5,000.00	$ 10,000.00	$ 10.00	$ 1,500.00	$ 45.00
		Ch. Exc.	2' S. C.	8'x8' S. C.	2'x3' Cutoff	3'x6' Cutoff	Reveg.	Salvage		Dissipator	66" Pipe	84" Pipe	90" Pipe	Junction Structure		90" Pipe Endwall	66" Pipe Outlt. Wks.	6' Fence		Riprap
Subtotal Costs		$ 715,149	$ 463,905	$ 376,670	$ 54,285	$ 10,770	$ 52,371	$ 634,253	$ 68,800	$ 39,100	$ 79,975	$ 168,140	$ 339,150	$ 30,000	$ 15,000	$ 5,000	$ 10,000	$ 54,800	$ 4,500	$ 84,375
	Subtotal without R.O.W.	$ 3,206,243																		

464, 675

Mobilization	$20,000
Signage	$10,000
Survey	$25,000
Util. Relocation	$5,000
Subtotal	$3,266,242.65
15% Contingincy	$489,936.40
Grand Total	$3,756,179.05

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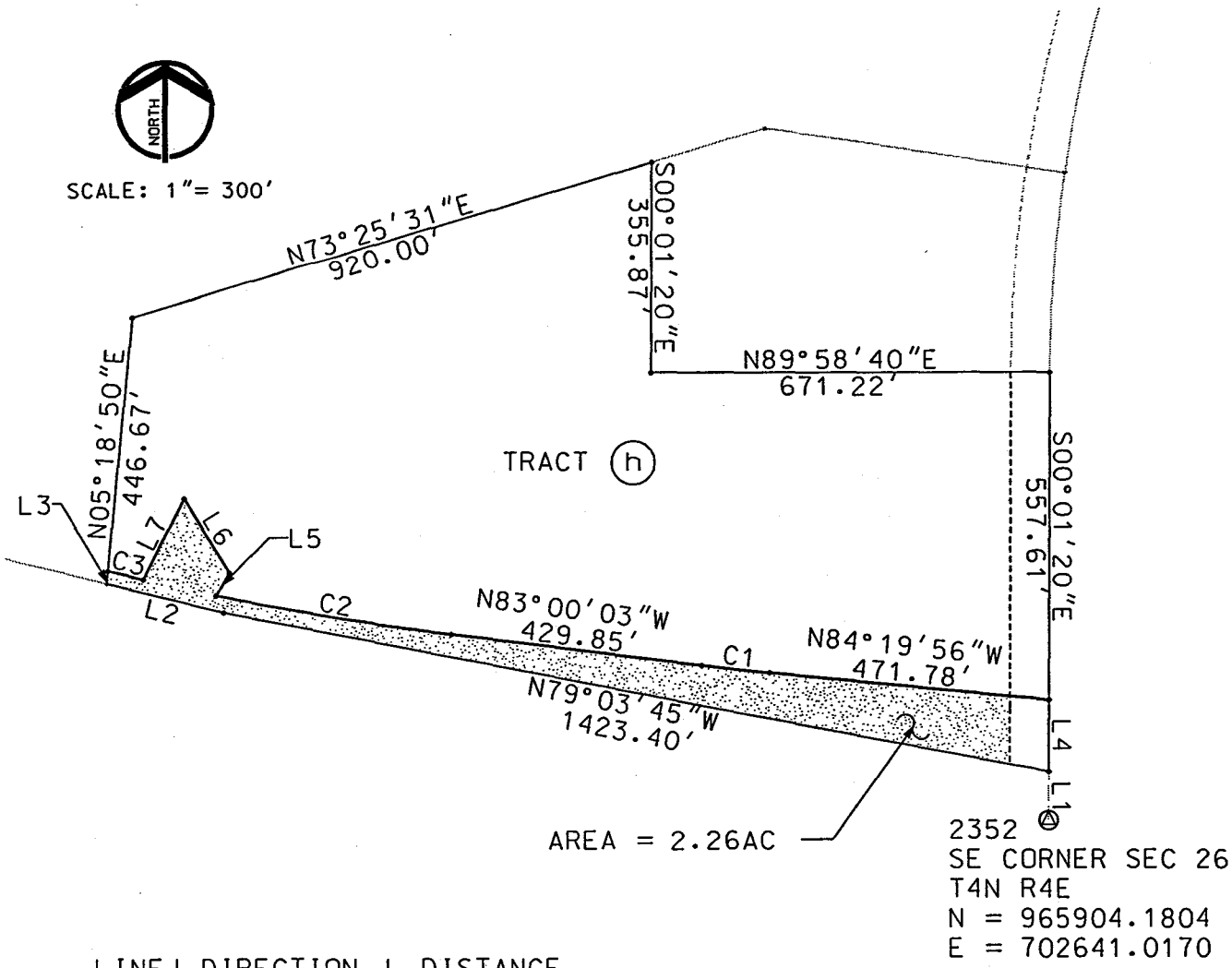
Outer Loop Drainage System
ROW Areas by Parcel

PARCEL	DESCRIPTION	ROW AREA (acres)
H	CORE NORTH	2.26
G	CORE NORTH	0.72
F	CORE NORTH	7.33
215-05-01B	CORE NORTH	2.40
COS	76TH STREET	2.56
TRACT 3A	CORE SOUTH	3.62
TRACT 3B	CORE SOUTH	1.20
TRACT 1	CORE SOUTH	0.84
CAP	TPC	2.03
	TOTAL =	22.96

Outer Loop Drainage System
Comparative Cost Analysis

				PENTACORE REPORT				GREINER REPORT	
Description	**TOTALS**	**UNITS**	**Unit Cost**	**Item Cost**	**TOTALS**	**UNITS**	**Unit Cost**	**Item Cost**	**DIFFERENCE**
Channel & Basin Excavation	238,383	CY	$3.00	$715,149	279,705	CY	$3.00	$839,115	$123,966
Plated Soil Cement	10,309	CY	$45.00	$463,905					($463,905)
Bulk Soil Cement	10,762	CY	$35.00	$376,670	12,202	CY	$25.00	$305,050	($71,620)
2'x3' S.C. Cutoff	3,619	LF	$15.00	$54,285					($54,285)
3'x6' S.C. Cutoff	359	LF	$30.00	$10,770					($10,770)
Revegetation (hydroseed)	1,047,423	SF	$0.05	$52,371	271,872	SF	$0.05	$13,594	($38,778)
Salvage	1,268,505	SF	$0.50	$634,253	1,427,219	SF	$0.50	$713,610	$79,357
Basin Perimeter Landscaping	68,800	SF	$1.00	$68,800					($68,800)
Energy Dissipator (Struct. Conc.)	170	CY	$230.00	$39,100					($39,100)
66" RGRCP	457	LF	$175.00	$79,975					($79,975)
84" CIP Concrete Pipe	1,201	LF	$140.00	$168,140					($168,140)
90" CIP Concrete Pipe	1,995	LF	$170.00	$339,150					($339,150)
Standard Manhole	6	EA	$2,500.00	$15,000					($15,000)
Junction Structure	2	EA	$15,000.00	$30,000					($30,000)
6' Fence	5,480	LF	$10.00	$54,800					($54,800)
6' Gates	3	EA	$1,500.00	$4,500					($4,500)
66" Outlet Works	1	EA	$10,000.00	$10,000					($10,000)
Structural Concrete					449	CY	$180.00	$80,820	$80,820
Box Culverts - Reinforcing Steel					67,630	LBS	$0.40	$27,052	$27,052
90" Pipe Endwall	1	EA	$5,000.00	$5,000					($5,000)
Riprap	1,875	CY	$45.00	$84,375	2,891	SY	$45.00	$130,095	$45,720
Mobilization	1	LS	$20,000.00	$20,000	1	LS	$60,000.00	$60,000	$40,000
Signage	1	LS	$10,000.00	$10,000	1	LS	$30,000.00	$30,000	$20,000
Construction Survey & "As-Builts"	1	LS	$25,000.00	$25,000	1	LS	$60,000.00	$60,000	$35,000
Utility Relocation	1	EA	$5,000.00	$5,000	1	EA	$18,630.00	$18,630	$13,630
Revegetation (fully revegetated)					422,500	SF	$1.00	$422,500	$422,500
6' Concrete Retaining Wall Buried					400	LF	$100.00	$40,000	$40,000
Subtotal				**$3,266,243**				**$2,740,465**	($525,778)
15% Contingency	0.15	LS	$3,266,243	$489,936	0.15	LS	$2,740,465	$411,070	($78,867)
Total Cost				**$3,756,179**				**$3,151,535**	**($604,644)**



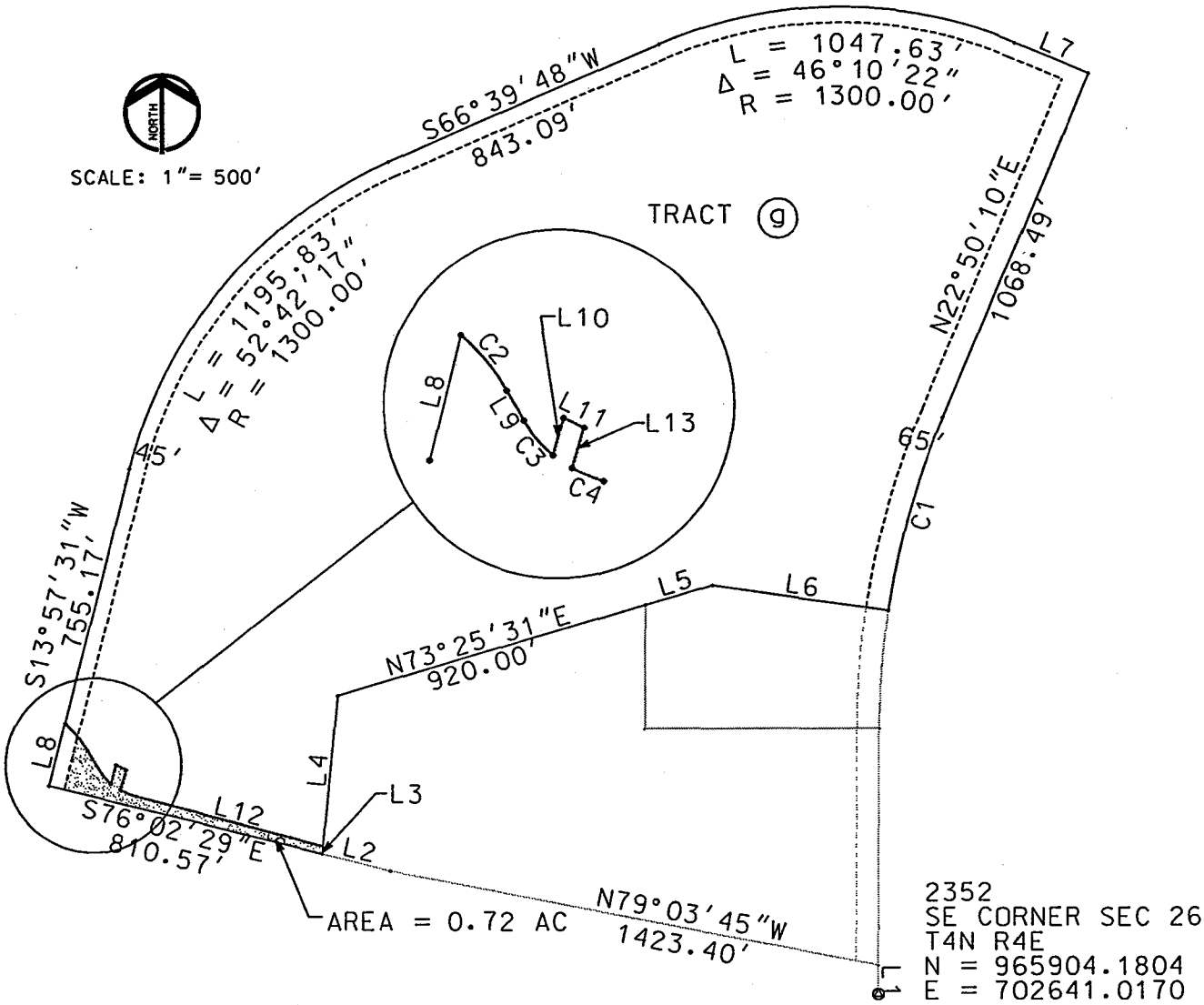
LINE	DIRECTION	DISTANCE ft
L4	S00°01'20"E	121.83
L1	N00°01'20"W	82.89
L2	N76°02'29"W	204.17
L3	N05°18'39"E	21.77
L5	S28°58'22"W	44.26
L6	S31°12'53"E	148.04
L7	S26°36'16"W	154.03

CURVE	LENGTH ft	DELTA	RADIUS ft
C1	114.74	01°19'53"	4937.50
C2	405.09	04°42'03"	4937.50
C3	61.25	00°42'39"	4937.50

Glenn D. Carter, R.L.S. #16509 Date

PENTACORE
ARIZONA
2255 N. 44th STREET, SUITE 255
PHOENIX, ARIZONA 85008 (602) 6819272

CORE NORTH TRACT h



LINE	DIRECTION	DISTANCE ft
L3	N05°18'50"E	8.62
L8	N13°57'33"E	185.25
L12	S75°00'00"E	462.73
L9	S30°00'06"E	49.87
L10	N15°09'39"E	55.38
L11	S63°47'35"E	32.56
L13	S16°03'57"W	60.24
L2	N76°02'29"W	204.17
L4	N05°18'49"E	433.52
L5	N73°25'31"E	200.00
L6	S81°34'25"E	510.00
L7	S67°09'50"E	227.33
L1	N00°01'20"W	82.89

CURVE	LENGTH ft	DELTA	RADIUS ft
C1	578.44	14°24'35"	2300.00
C2	103.37	18°57'06"	312.50
C3	65.84	20°07'11"	187.50
C4	48.44	14°48'06"	187.50

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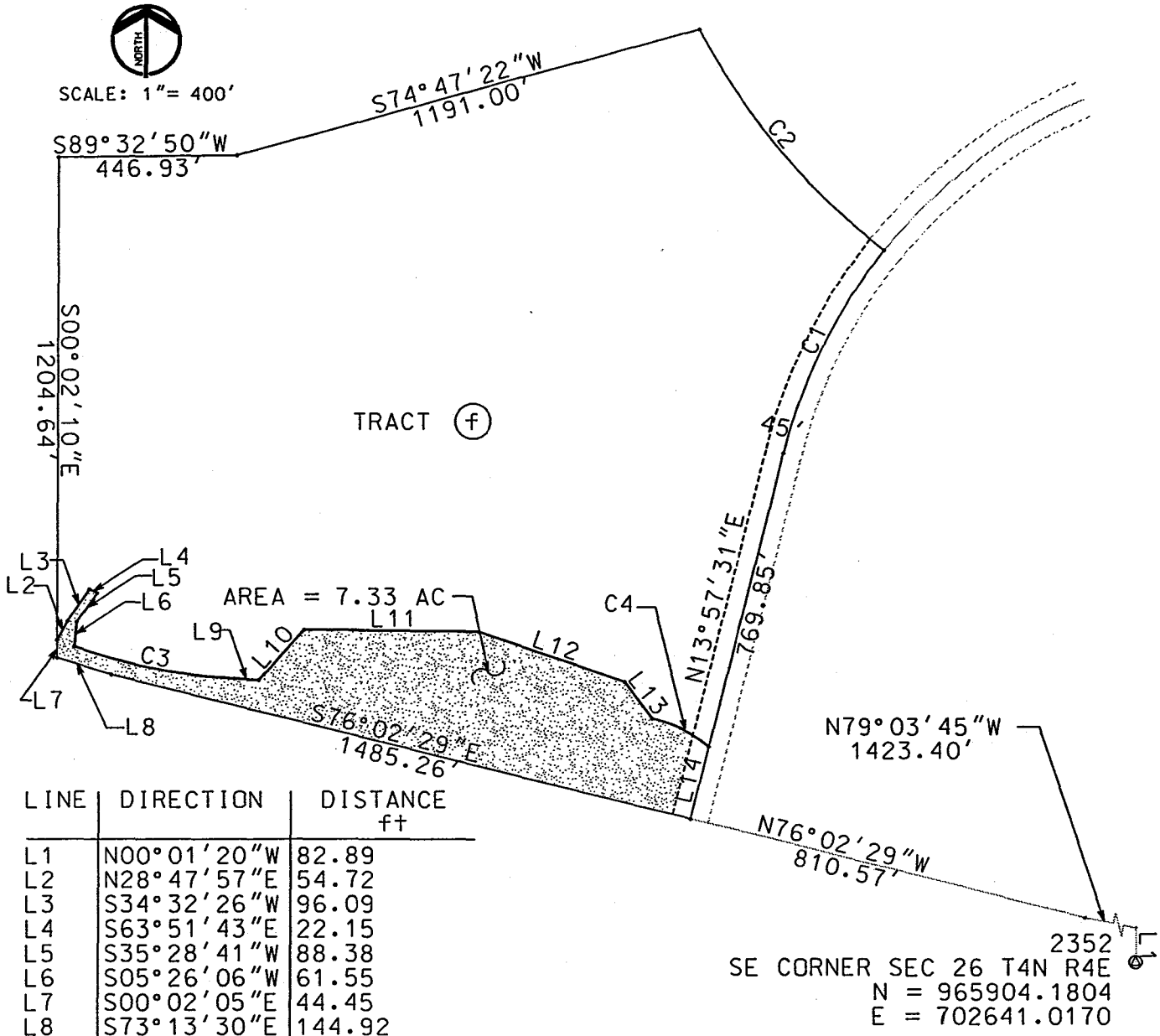
CORE NORTH TRACT g



LINE	DIRECTION	DISTANCE ft
L1	N00°01'20"W	82.89
L2	N28°47'57"E	54.72
L3	S34°32'26"W	96.09
L4	S63°51'43"E	22.15
L5	S35°28'41"W	88.38
L6	S05°26'06"W	61.55
L7	S00°02'05"E	44.45
L8	S73°13'30"E	144.92
L9	S87°00'26"E	61.96
L10	N41°36'17"E	167.76
L11	S89°21'02"E	436.53
L12	S70°55'33"E	383.79
L13	N36°50'32"W	112.45
L14	N13°57'33"E	185.25

CURVE	LENGTH ft	DELTA	RADIUS ft
C1	567.43	25°00'31"	1300.00
C2	665.72	23°41'29"	1610.00
C3	410.43	16°38'55"	1412.50
C4	142.07	26°02'56"	312.50

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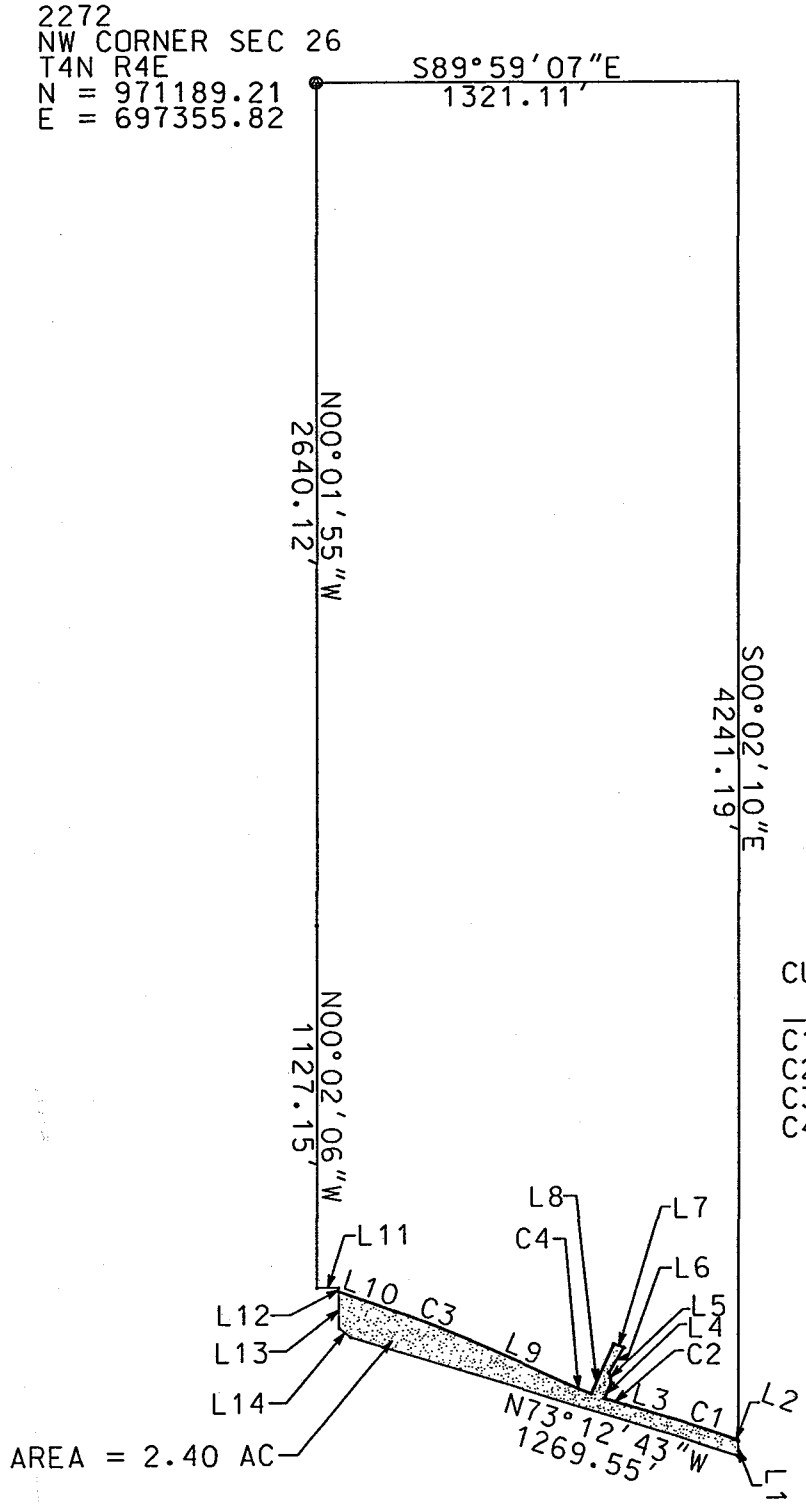
CORE NORTH TRACT f

2272
NW CORNER SEC 26
T4N R4E
N = 971189.21
E = 697355.82



NORTH

SCALE: 1"= 600'

LINE	DIRECTION	DISTANCE ft
L1	S00°02'11"E	44.15
L2	S69°54'10"E	16.46
L3	S73°58'02"E	140.58
L4	S24°04'10"W	53.06
L5	S13°52'44"E	31.19
L6	N30°55'47"E	90.39
L7	N70°09'59"W	33.11
L8	N24°31'11"E	168.32
L9	S66°12'32"E	327.76
L10	S70°56'34"E	186.03
L11	N89°56'57"E	69.99
L12	S00°03'04"E	12.33
L13	S00°03'01"E	112.67
L14	N51°19'39"W	44.81

CURVE	LENGTH ft	DELTA	RADIUS ft
C1	215.15	04°03'54"	3032.50
C2	69.34	02°42'25"	1467.50
C3	250.54	04°44'01"	3032.50
C4	86.52	03°22'41"	1467.50

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CORE NORTH TRACT