East Maricopa Floodway

Capacity Mitigation and Multi-Use Corridor Study

Conceptual Design Alternatives Report

Hydrology/Hydraulics Assessment

September 2000

FLOOD CONTROL DISTRICT of MARICOPA COUNTY

Collins/Piña
A Subsidiary of TetraTech, Inc.

Engineering
Planning • Surveying
Landscape Architecture
Construction Administration

East Maricopa Floodway
Capacity Mitigation and Multi-Use Corridor Study
Maricopa County
Contract FCD 1999C056

Prepared for
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EXECUTIVE SUMMARY

The Flood Control District of Maricopa County (District) initiated a study in August 1997 to assess the capacity of the East Maricopa Floodway (EMF) to determine if the existing floodway could convey its design discharges and to identify any problem areas for the existing and future flow conditions. The study's main objectives were to evaluate the impacts of changing conditions on the watershed, to prioritize any future channel improvements, to provide information for flood control management in the area and to regulate future discharges to the floodway.

The preliminary analysis demonstrated that the EMF does not have the capacity for the peak runoff resulting from the 100-year, 24-hour rainstorm event within its watersheds.

The current effort, a continuation of the original effort, involves three phases. Phase 1, already completed by the Landscape Architecture Department of Collins/Piña Consulting Engineers (CPE), evaluated the potential for multi-use opportunities along and adjacent to the East Maricopa Floodway. These uses include but are not limited to trails, active and passive recreation, and restoration facilities. This report represents Phase 2, which preliminarily evaluates and identifies infrastructure improvements, including channel improvements and the construction of stormwater detention basins, to mitigate the capacity shortfalls.

All analyses were completed using District HEC-1 and HEC-RAS models of the EMF and its watersheds. No new hydrologic analysis was completed for this report. Instead, the existing models were manipulated to represent the four alternatives. The four alternatives incorporate three offline stormwater detention basins: the Ray Basin, which would accept runoff from both the Powerline Floodway and San Tan Freeway Channel, the Rittenhouse Basin, which would accept flow from the EMF and the Rittenhouse Channel, and Chandler Heights Basin, which would accept runoff from the EMF and the Queen Creek Wash.

The design criteria of each alternative were to attenuate the runoff from the 100-year, 24-hour storm event so that the peak would be conveyed in the channel without overtopping and the flow south of Hunt Highway would be at or below 8100 cfs (the SCS design peak flow for that location). Further, the peak within the EMF, assuming future full development and build-out, has to be conveyed in the channel with the required SCS freeboard. Finally, the design is constrained to include no channel alterations through golf courses and minimize the pumping requirements from the stormwater detention basins back into the EMF or other channels.

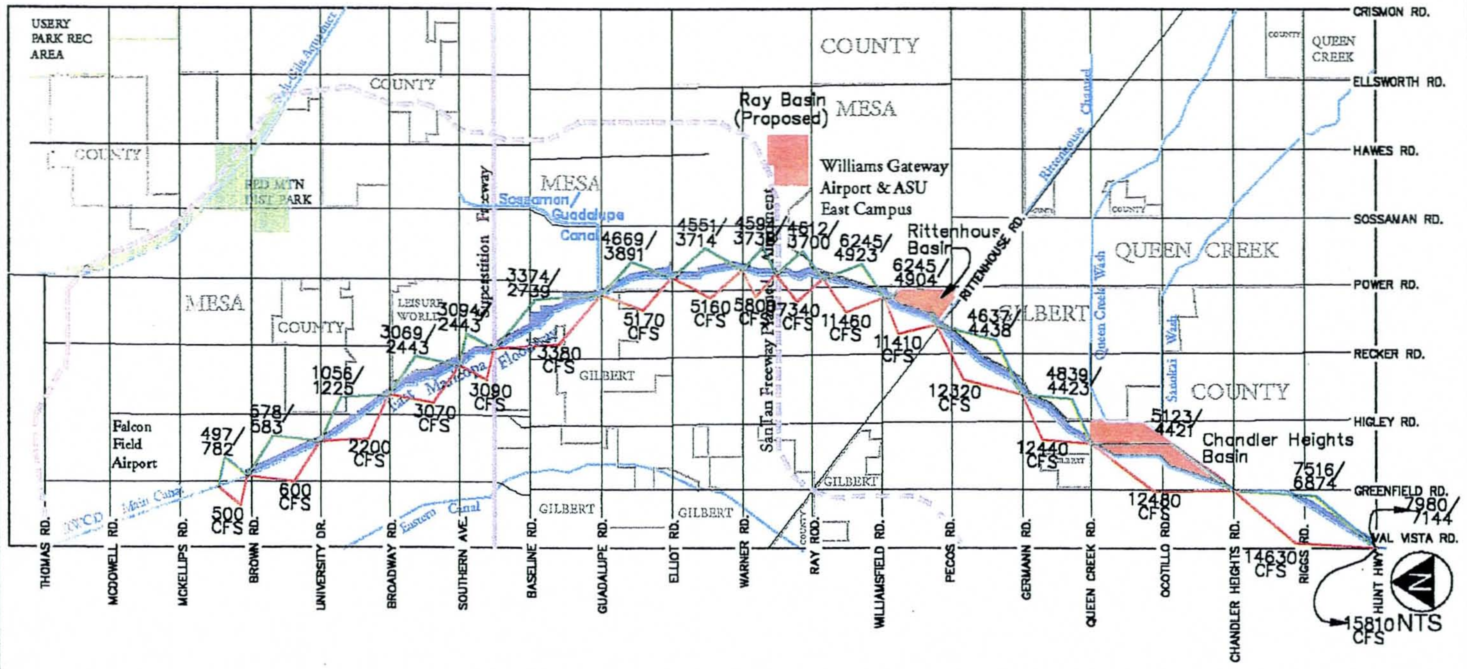
Alternative 1 is a strictly engineering, "no-frills" alternative with no multi-modal aspects. Alternative 2 incorporates recreational enhancements, both within the EMF channel and within the three detention basins. Alternative 3 incorporates environmental and habitat enhancement features, again, both within the EMF channel and within the offline detention basins. Alternative 4 eliminates the Ray Stormwater Detention Basin from the design. Because of this, the EMF channel in Alternative 4 had to be significantly altered, especially through the Williams Golf Course stretch, violating design criteria. No multi-modal features were incorporated into Alternative 4.

See Figure 1a-1d for each alternative design flows within the EMF versus the existing conditions with no improvements.

Except for the golf course exception of Alternative 4, the four alternatives meet the required design criteria. Runoff within the EMF is attenuated so that the 2002 peak flow does not overtop the channel and the 2002 peak south of Hunt Highway is below 8100 cfs. In addition, the Build-out peak can be conveyed within the EMF with the required SCS freeboard. To meet the requirements, the three offline stormwater detention basins within the alternatives require a large volume of storage. The large volumes require significant depths in the basins and, especially in Alternatives 2 and 3, significant pumping. Consequent to the large volumes, the culverts draining the basins are quite substantial.

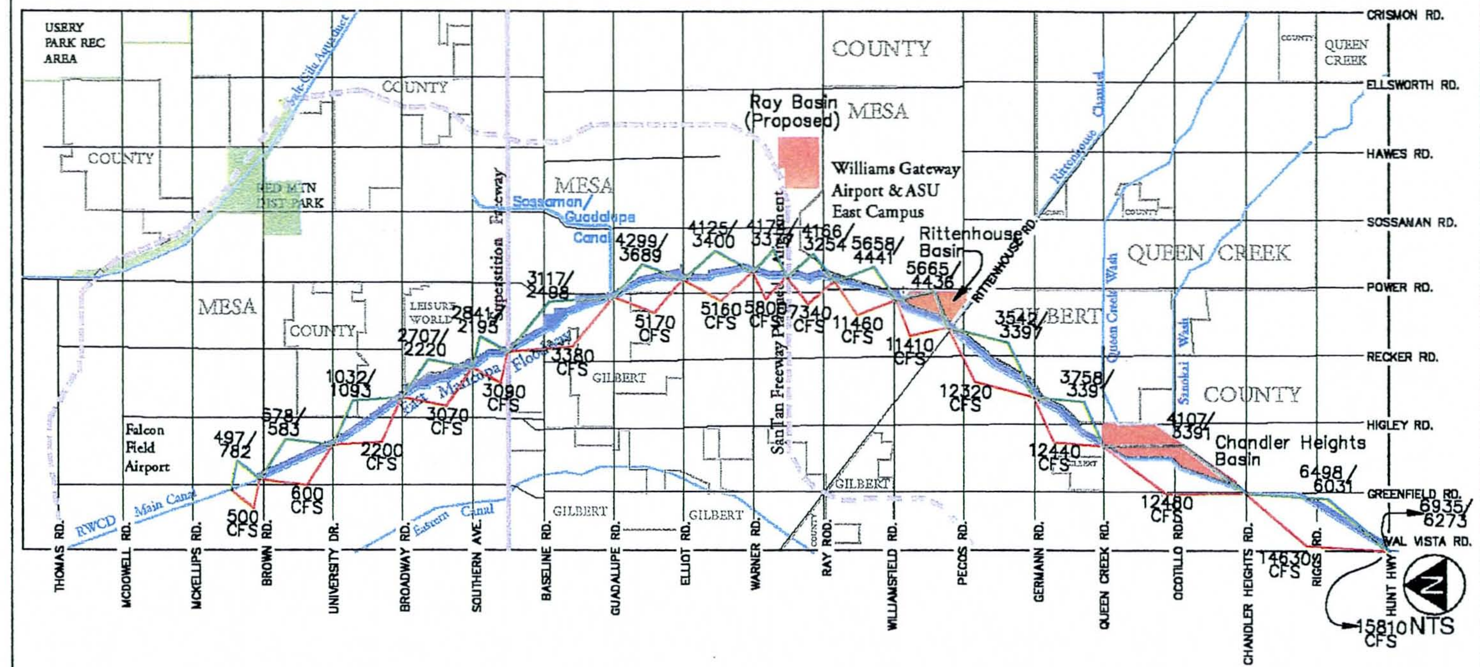
There are several conservative aspects to the design. First, because the build-out/freeboard criterion is more critical than the 2002 conveyance in most cases, the crest heights and lengths of the diversion weirs are usually based on the Built-out event. However, the resulting volume of diversion, which mandates the size of each stormwater detention basin, is based on the 2002 event (the volume of water that is diverted from the channels with the designed diversion weir crest height and length). Therefore, each basin is forced to store a larger volume of water to satisfy both criteria. In addition, the required detention volumes are conservative because the entire top of the flow hydrograph within the channel is lopped off at each diversion, diverting all flow above a certain rate, with no storage considered above the invert of the weir crest. The volume of detention storage could be minimized if water was allowed to be stored above the weir crest height to an elevation that, when reached, would allow the remainder of the flow in the channel to pass by. This would also allow water to flow from the stormwater detention basin back into the channel over the weir after the peak has passed, lessening the burden on the outlet culverts. These conservative design elements should be carefully considered in the final design of the basins. Their alteration could substantially reduce the cost of each stormwater detention basin.

With the conservative design approach, the final project costs are $89.7 million, $132.8 million, $147.1 million, and $99.9 million for Alternatives 1-4 respectively. Note that the recreational and multi-use designs of Alternatives 2-3 are more expensive than the "engineering" designs of Alternatives 1 and 4, due primarily to the larger required detention basins of the recreational alternatives. Also note that Alternative 4 costs do not include several bridge modifications that would increase the overall cost.



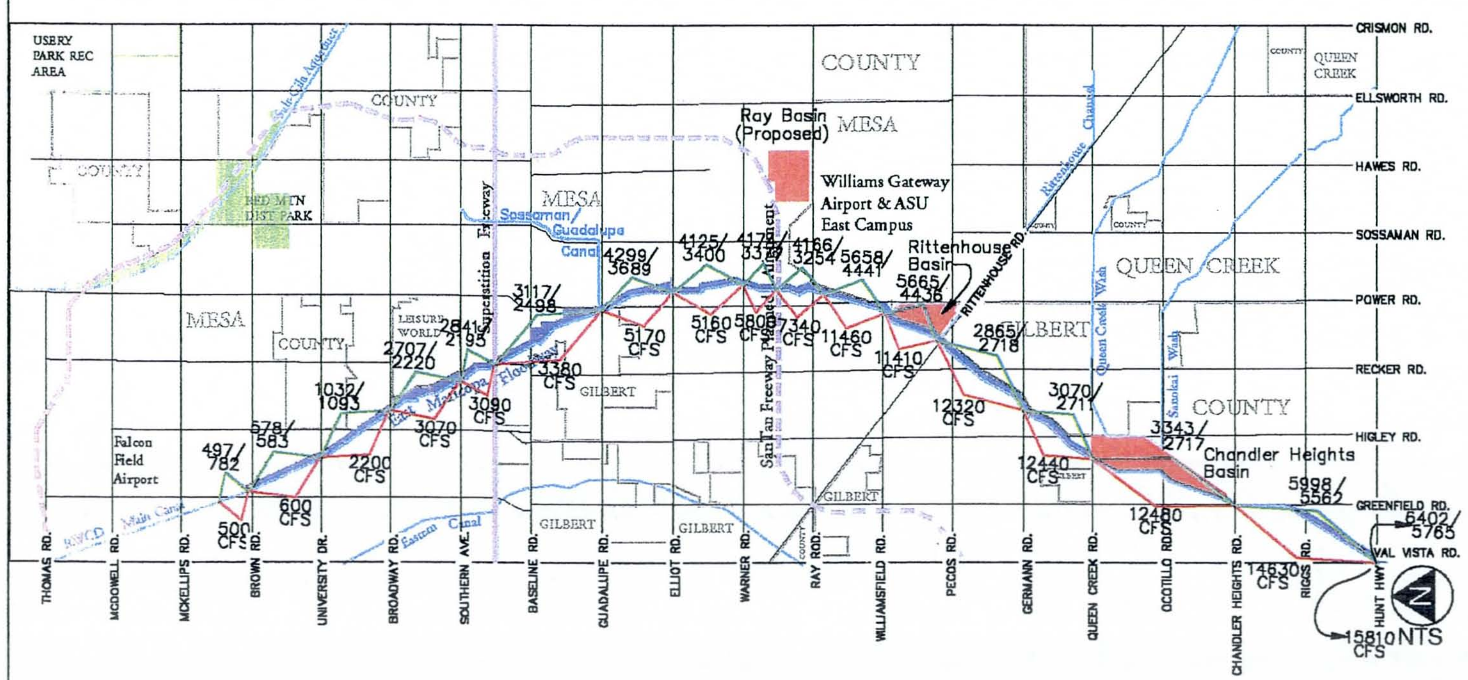
CPE ALTERNATIVE 1 PEAK FLOWS (2002/BUILDOUT CONDITIONS)
EXISTING CONDITIONS

FIGURE 1a
EMF ALTERNATIVE 1 PEAK FLOWS VS. EXISTING CONDITIONS W/ NO IMPROVEMENTS



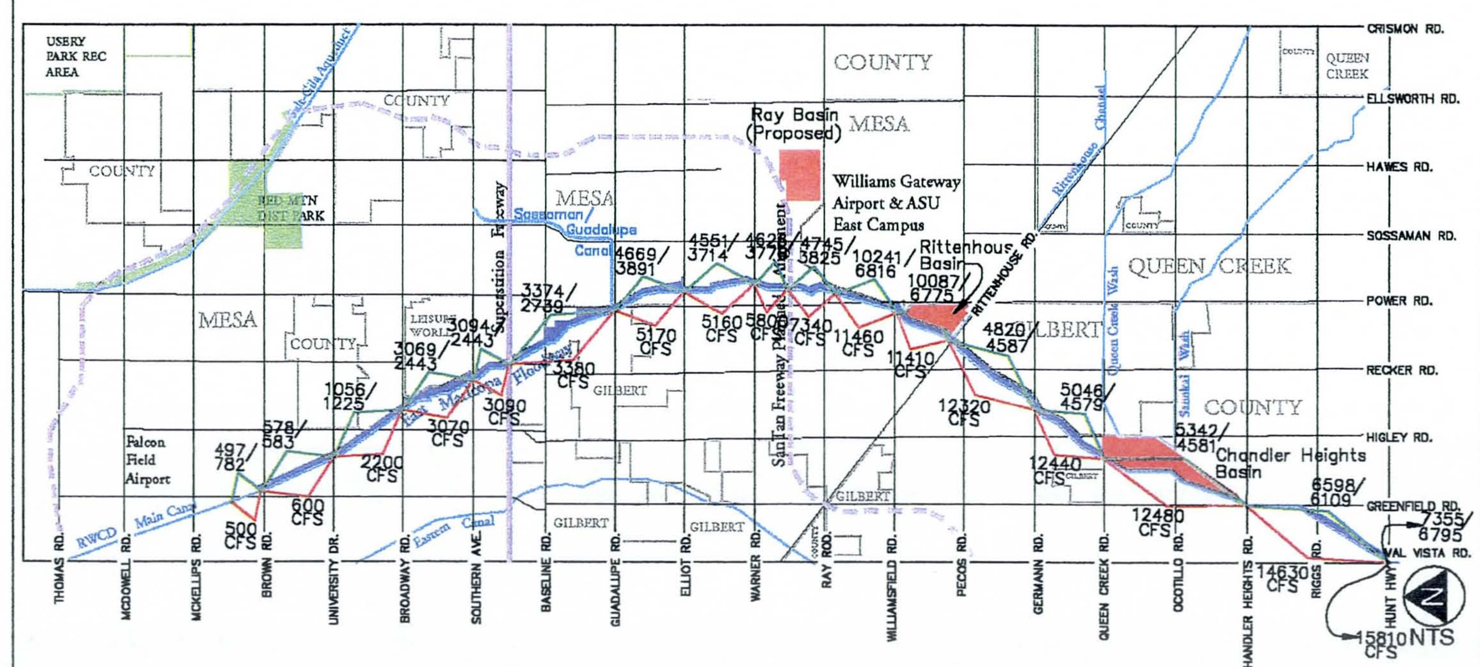
CPE ALTERNATIVE 2 PEAK FLOWS (2002 BUILDOUT CONDITIONS)
EXISTING CONDITIONS

FIGURE 1b
EMF ALTERNATIVE 2 PEAK FLOWS VS. EXISTING CONDITIONS W/ NO IMPROVEMENTS



CPE ALTERNATIVE 3 PEAK FLOWS (2002 / BUILDOUT CONDITIONS)
EXISTING CONDITIONS

FIGURE 1c
EMF ALTERNATIVE 3 PEAK FLOWS VS. EXISTING CONDITIONS W/ NO IMPROVEMENTS



CPE ALTERNATIVE 4 PEAK FLOWS (2002/BUILDOUT CONDITIONS)
EXISTING CONDITIONS

FIGURE 1d
EMF ALTERNATIVE 4 PEAK FLOWS VS. EXISTING CONDITIONS W/ NO IMPROVEMENTS

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1.0 INTRODUCTION

The Flood Control District of Maricopa County (District) initiated a study in August 1997 to assess the capacity of the East Maricopa Floodway (EMF) to determine if the existing floodway could convey its design discharges and to identify any problem areas for the existing and future flow conditions. The study's main objectives are to evaluate the impacts of changing conditions on the watershed, to prioritize any future channel improvements, to provide information for flood control management in the area and to regulate future discharges to the floodway. Huitt-Zollars, Inc., performed the initial conceptual development with their work on the Queen Creek and Sanokai Wash Hydraulic Master Plan. Suggested improvements included channel improvements within the floodway and the construction of stormwater detention basins adjacent to the EMF and/or within its watersheds to reduce peak discharges into the channel. Channel improvements included improved channel lining and the extension of existing levees.

The current effort, a continuation of the original effort, involves three phases. Phase 1, already completed by the Landscape Architecture Department of Collins/Piña Consulting Engineers (CPE), evaluated the potential for multi-use opportunities along and adjacent to the East Maricopa Floodway. These uses include but are not limited to trails, active and passive recreation, and groundwater recharge facilities. This report represents Phase 2, which preliminarily evaluates and identifies infrastructure improvements, including channel improvements and the construction of stormwater detention basins, to mitigate capacity shortfalls. Phase 3 will integrate the multi-use corridor study with the defined engineering alternatives to determine a preferred alternative concept that will mitigate the EMF capacity shortfalls but will be consistent with potential multi-uses.

1.1 PURPOSE OF STUDY & SUMMARY OF EMF UNDER EXISTING CONDITIONS

The EMF is the major regional flood control conveyance structure in Eastern Maricopa County. Because of development within its upstream watersheds, preliminary analysis demonstrates that the 100-year design peak discharge breaks out of the channel along approximately seven miles of the EMF channel: north and south of Riggs Road Bridge, at the inlet of the Powerline Floodway, north of Rittenhouse Bridge, from Rittenhouse Road south to the Higley Road Bridge, and south of Elliot Road to Ray Road. The upstream flood breakouts could reduce the flow downstream so that the breakouts downstream could be lessened or eliminated. But because of this lack of channel capacity, as the development in the upstream watersheds and along the EMF continues, the potential for serious flood damage increases.

Additionally, as the area around the EMF becomes more densely populated, the desire for recreation and alternative transportation corridors also increases. The EMF alignment has been viewed as an opportunity to create a multi-use linkage though Eastern Maricopa County to provide for parks, athletic facilities, biking/walking/equestrian trails and environmental improvements.

The purpose of this study is to provide four alternatives, each addressed at a design/concept level, for infrastructure improvements to the EMF. These alternatives include channel improvements and potential areas for offline stormwater detention basins. Each alternative has been formulated so that the EMF can convey the 100-year peak discharge with the required freeboard for the watershed under build-out (future) conditions. In addition, selected alternatives integrate multi-use concepts to provide for recreation and/or environmental improvements.

1.2 STUDY AREA AND LIMITS

The Soil Conservation Service, now referred to as the National Resources Conservation Service, constructed the EMF in 1989 for flood control in the East Valley of Maricopa County. The EMF is now owned and operated by the District and is approximately 27 miles long, extending from Princess Basin near Brown Road in north Mesa to the Gila River in Pinal County. It is a compacted earthen trapezoidal channel along the majority of its reaches, ranging in width from 150 to 300 feet, and ranging in depth from eight to twelve feet. There is a one-mile-long reach near the Williams Gateway Airport and a stretch through the Gila Indian Community where the EMF is concrete lined. See Figure 2 for the four typical cross sections.

The EMF intercepts runoff from portions of the City of Mesa, the City of Chandler, the Town of Gilbert, the Town of Queen Creek, unincorporated Maricopa County, Pinal County and the Gila River Indian Community. Its runoff originates within three major watersheds: Buckhorn-Mesa, Apache Junction-Gilbert, and Williams-Chandler. From these watersheds, six major drainage channels contribute to the EMF. These channels and their locations are as follows:

Broadway Channel: Drains portions of Mesa and unincorporated Maricopa County and flows west to discharge to the EMF just south of Broadway Road.

Superstition Freeway Channel: Drains portions of Mesa and unincorporated Maricopa County and flows west to discharge to the EMF just north of the Superstition Freeway. (US 60).

Guadalupe Channel: Drains portions of Mesa and unincorporated Maricopa County and flows west to discharge to the EMF just south of Guadalupe Road.

Powerline Floodway: Drains portions of the Williams Gateway Airport and unincorporated Maricopa County, including the outflow from the Powerline Dam and flows west to discharge to the EMF near Ray Road.

Rittenhouse Road Channel: Drains portions of Queen Creek, Mesa and Gilbert and flows northwesterly to discharge to the EMF just north of Rittenhouse Road.

Queen Creek and Sanokai Wash: These two washes drain portions of Queen Creek, Gilbert, unincorporated Maricopa County, and Pinal County. The confluence of the two washes is located near the intersection of Higley and Ocotillo Roads. The flow in Queen Creek Wash then discharges to the EMF just north of Chandler Heights Road.

See Figure 3, EMF Mitigation Site Map.

1.3 REVIEW OF PREVIOUS WORK

To date, numerous studies have been completed for the EMF capacity mitigation design. These reports form the basis of the current Capacity Mitigation and Multi-use Corridor Study. A brief outline of each study is detailed below:

East Maricopa Floodway Capacity Assessment Study (HNTB, 1999): This study evaluated the conveyance capacity of the entire EMF for both the existing and future conditions, 100-year, 24-hour discharges. It also evaluated the capacity for the SCS design discharge that was used in the initial design of the EMF and determined the conveyance capacity of the EMF under bank-full conditions. This study has been incorporated in the hydraulic evaluation of the EMF for the Capacity Mitigation and Multi-use Corridor Study.

Sanokai Wash Floodplain Delineation Study (Entellus, 1999): This study developed the existing conditions hydrology for the Sanokai Wash watershed and delineated the Sanokai Wash floodplain. The study area includes the Sanokai Wash watershed located in the southeast corner of Maricopa County and northern Pinal County. The hydrology was incorporated into the District's EMF hydrologic analysis.

East Mesa Area Drainage Master Plan (District/Dibble and Assoc., 1998): This study was initiated in order to provide flood protection to the East Mesa Area. It determined the existing and future conditions hydrology for the East Mesa area for planning purposes and identified drainage problems and proposed drainage facilities to address current and future flooding problems. The study's boundaries are the Central Arizona Project (CAP) canal to the north, the Powerline Flood Retarding Structure, the Vineyard Flood Retarding Structure, and the Rittenhouse Flood Retarding Structure to the east, Queen Creek Road to the south and the EMF to the west. This study's hydrologic analysis was incorporated into the hydrologic model used in the Capacity Mitigation and Multi-use Corridor Study.

Queen Creek Area Drainage Master Study (Wood and Associates, 1991): This study was initiated to identify stormwater problems in the Queen Creek area and provide a master drainage plan to mitigate these problems. The Goldmine and San Tan Mountains to the south, the EMF to the west, the CAP canal to the east, and the Powerline Freeway to the north bind the study's limits. The study's hydrologic analysis was incorporated into the hydrologic model used in the Capacity Mitigation and Multi-use Corridor Study after the loss models were updated into Maricopa County Green and Ampt methods by Huit-Zollars.

Queen Creek/Sanokai Wash Hydraulic Master Plan (Huitt-Zollars, under study): This study was initiated to formulate drainage improvements for use by local municipalities as a guide for future development in the area. The study includes an update of the existing conditions hydrologic model of the Queen Creek watershed and the development of the future conditions hydrologic model for the Queen Creek/Sanokai Wash watersheds. The study area incorporates the entire Queen Creek and Sanokai Wash watersheds, which include portions of northern Pinal County. This study's hydrologic analysis was incorporated into the hydrologic model used in the Capacity Mitigation and Multi-use Corridor Study.

East Maricopa Floodway Capacity Mitigation Study Report, (Huitt-Zollars, 2000): This study integrates the above studies into both a hydrologic model and a hydraulic model for the EMF. It proposes several alternatives for structural improvements to the EMF and for watershed and EMF stormwater detention basins. This study and its hydrologic and hydraulic models are the framework for the 'Capacity Mitigation and Multi-use Corridor Study (the current study).

1.4 STUDY CRITERIA

The Capacity Mitigation and Multi-use Corridor Study incorporates the hydrologic/hydraulic requirements mandated by the District. They include the following:

1) The 100-year, 24-hour Maricopa County storm event with SCS Type II temporal rainfall distribution was used in all hydrologic models to determine the peak 100-year discharges. All models were generated by others. Two sets of District models were used as the basis for all anlaysis: the 2002 conditions and the future build-out conditions. The 2002 models simulate developed conditions projected for the year 2002 with some capitol improvement projects in place (specifically the CAP stormwater detention basins and the Elliot Stormwater Detention Basin) as well as local on-site retention where it is required. Future build-out models represent complete build-out conditions according to projected zoning restrictions in the entirety of the watersheds contributing to the EMF with all CIP infrastructure plus all required on-site stormwater retention in place.
2) At Hunt Highway, peak discharges within the EMF will be reduced, at maximum, to the SCS design discharge (8100 cfs) to ensure that Reaches 1 and 2, which are within Pinal County an the Gila River Indian Community, will not be adversely impacted by the improvements.
3) Minimum freeboard within the EMF must meet SCS criteria which are as follows:
   - $0.20 \times (Y + v^2 / 2g)$ for subcritical flow conditions
   - $0.25 \times (Y + v^2 / 2g)$ for supercritical flow conditions
   - Minimum 1-foot clearance from WSEL to low cord of bridge or top of culvert

   For this study, future (build-out) conditions require that minimum freeboard based on SCS criteria be met. Existing/2002 conditions require all flows be contained within the full channel cross-section.
4) No channel improvements will be made through golf courses.
5) All stormwater detention basins should drain within 36 hours after the passing of the 100-year design storm event.

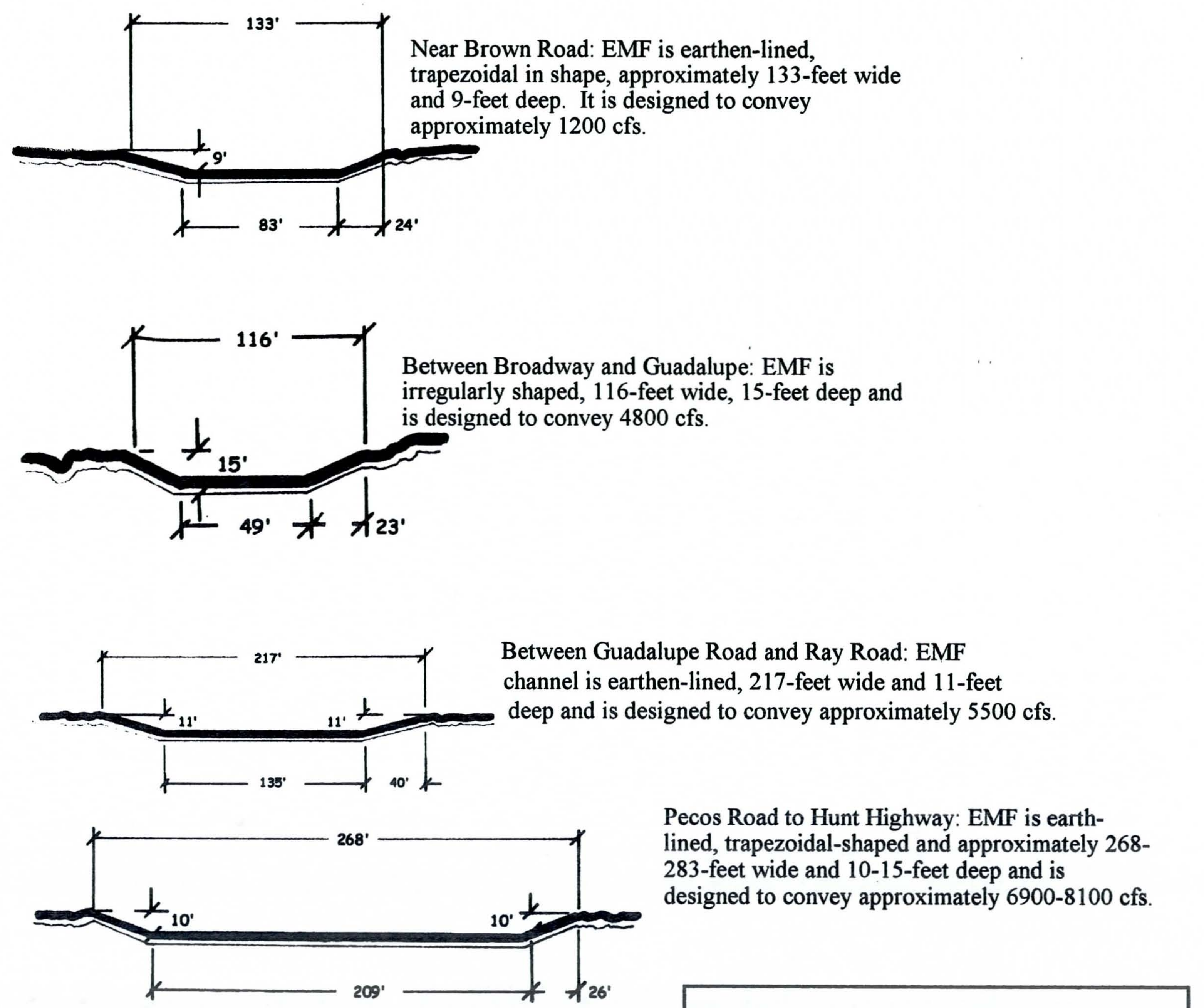
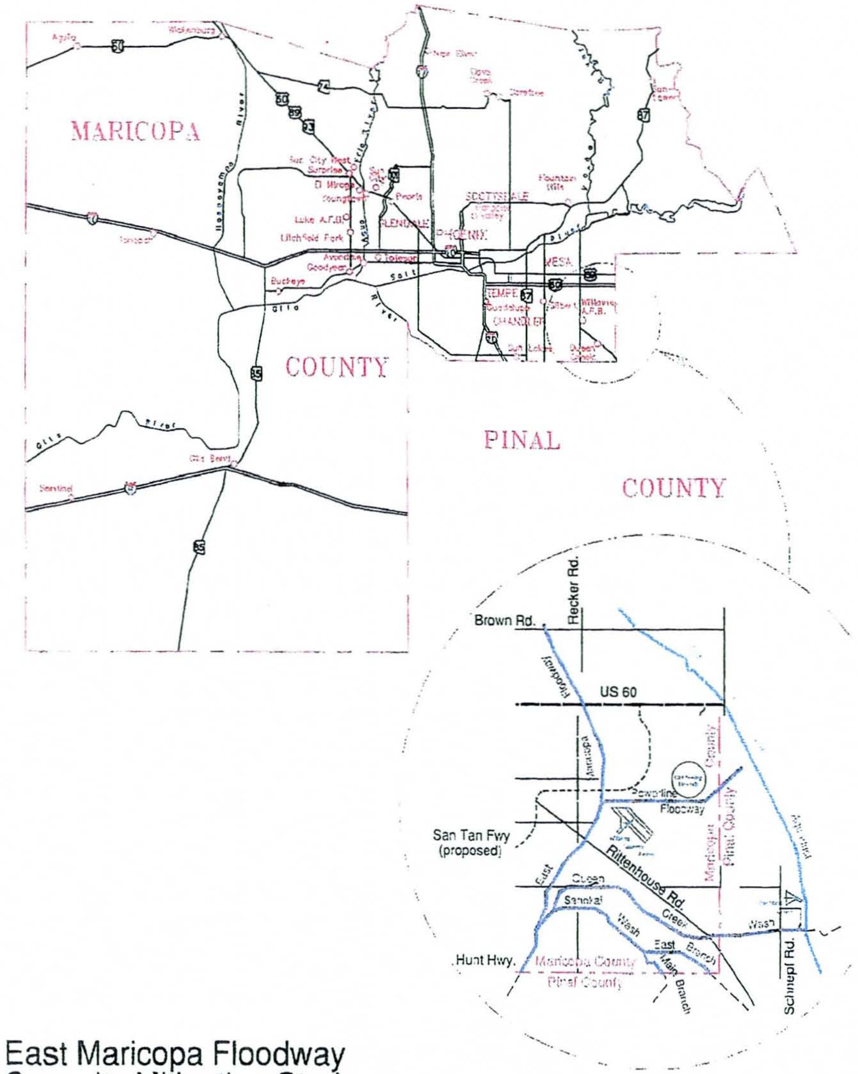


Figure 2: Four Typical Cross Sections of the EMF.



East Maricopa Floodway
Capacity Mitigation Study

Figure 3 Location and Vicinity Map

2.0 HYDROLOGY

2.1 STUDY METHODOLOGY

No new hydrologic analysis was completed for the mitigation study. The previously generated hydrologic models completed in the previous studies were used to evaluate the three alternatives. These models were last integrated and modified by Huitt-Zollars and the District early in 2000, incorporating the most recent hydrologic modeling. The District and Huitt-Zollars modified the models so that they are consistent with Maricopa County methodology and represent the most accurate routing. For the study, only routing and diversion modifications within these models were made to reflect the proposed alternatives. No change was made to the rainfall/runoff portions of the models or to the subbasin configuration, hydrologic parameters, or routing configuration, except to reflect the proposed alternatives.

The hydrologic models, compiled and modified by the District and Huitt-Zollars and used in this alternative evaluation, were generated with the U.S. Army Corp of Engineers' HEC-1 flood hydrograph package. The HEC-1 models were prepared according to the methodology presented in the "Drainage Design Manual for Maricopa County, Arizona, Volume I, Hydrology".

The 24-hour, 100-year precipitation data were based upon the "Precipitation Frequency Atlas for Arizona" (NOAA, 1973). The rainfall was applied uniformly for critically centering over the watersheds with depth-area reduction factors based on Maricopa County methodology. Runoff parameters were based on soil data from the "Soil Survey of Eastern Maricopa and Northern Pinal Counties Areas, Arizona" (USDA, 1974) and Maricopa County soils maps. Where soil data were not available, estimates were made from surrounding basins. Land uses for the 2002 model were assigned as projected in the year 2002, according to Maricopa County and local municipal planners. Land uses for the Future conditions/Build-out model for areas within Maricopa County were assigned as they are projected to be with ultimate development according to Maricopa County and local municipal planners. Subbasins within Pinal County were left as in existing conditions due to Pinal County's development standards. Pinal County only mandates retention mitigation so that the rates of post-development runoff equal the rates of pre-development runoff in the 100-year storm event.

As outlined in the Maricopa County Hydrology Manual, Green and Ampt equations were used to determine rainfall losses and the Maricopa County S-Graph method was used for hydrograph generation. The subbasins within the Sanokai Wash FIS Study had their hydrographs developed from the Clark Unit Hydrograph Method. Muskingum or normal-depth routing was used for the majority of the subwatershed hydrograph routing to the EMF. Normal-depth routing was used for hydrograph routing through the EMF channel. Simulations of modifications were made using the District's basin and routing models as a base. See Table 1 for a summary of all model names and descriptions for both the District's models and the four alternatives.

Table 1: HEC-1 Model Name Summary

Model/ Area	District		Alternative 1		Alternative 2		Alternative 3		Alternative 4	
NE	NE2002	NEBUILD	NE2002	NEBUILD	NE2002	NEBUILD	NE2002	NEBUILD	NE2002	NEBUILD
NW	NW2002	NWBUILD	NW2002	NWBUILD	NW2002	NWBUILD	NW2002	NWBUILD	NW2002	NWBUILD
QC/SW	EXQCSW	QCSWBLT	EXQCSW	QCSWBLT	EXQCSW	QCSWBLT	EXQCSW	QCSWBLT	EXQCSW	QCSWBLT
SE	2002UP/ 2002QCRS	MIDDOUT	SEO0ALT1	SEBTALT1	SEO0ALT2	SEBTALT2	SEO0ALT3	SEBTALT3	SEO0ALT4	SEBTALT4
Routing	2002UPRT/ 2002ACRS	BUILDRT	RT02ALT1	RTBTALT1	RT02ALT2	RTBTALT2	RT02ALT3	RTBTALT3	RT02ALT4	RTBTALT4

2.2 EXISTING CONDITIONS (2002) MODEL

2.2.1 Modeling Parameters and Assumptions

The modeling parameters were updated in the 2002 model by the District to reflect projected development in the year 2002. The changes from the existing conditions models include the simulation of local onsite retention within areas where regulations require it. They also include proposed CIP improvement projects and CAP stormwater detention basins, minus their outfall channels.

2.2.2 Model Descriptions

There are four watershed models in the HEC-1 package. The fifth routing model simulates the routing of the contributing runoff through the EMF. A discussion of the five District 2002 base models follows:

1) Northwest (NW2002): Simulates runoff north of U.S. 60 and east of the EMF. Hawes Road is the boundary between this model and one to the east (Northeast). This model does not include any capital improvement projects.
2) Northeast (NE2002): Simulates runoff east of Hawes Road and the Sossaman channel. This model includes the CAP stormwater detention basins. The outlet channels to the CAP stormwater detention basins have not been modeled.
3) Southeast (2002UP): This model simulates runoff in the area south of U.S. 60 and north of the Rittenhouse channel and Queen Creek Road. The model includes retention in all subbasins north of Warner Road. It also models the CIP infrastructure that is in place between Crismon Road and the Maricopa County line, the proposed Santan Freeway Channel, and the Ray Basin along the Powerline Freeway. In addition, the model simulates routing flow through the Ellsworth Channel from Pecos Road to the Powerline Floodway.
4) Queen Creek/Sanokai Wash (EXQCSW): Simulates runoff in the Queen Creek and Sanokai Wash watersheds. No CIP infrastructure is included in this model.
5) EFM Routing Model (2002UPRT & ACROSSRT): These models simulate the hydrologic routing of runoff within the EMF. They have offline stormwater detention basins in place for some of the alternatives. The difference between the two models is that 2002UPRT corresponds to the routing of the Ellsworth Channel in 2002UPRT and ACROSSRT corresponds to the routing of flow across Pecos Road to the Rittenhouse Channel in 2002ACRS.

2.2.3 Results

The results of the existing 2002 model, with all the proposed CIP infrastructure improvements and offline EMF stormwater detention basins in place, demonstrate that the flow is attenuated sufficiently to contain the 100-year peak flows within the EMF. However, freeboard requirements are not met in a few locations, particularly below Riggs Road in Reach 3 and in the Williams Golf Course in Reach 4.

2.3 FUTURE CONDITIONS/BUILD-OUT MODEL

2.3.1 Modeling Parameters and Assumptions

These models include the simulation of ultimate build-out conditions in all the upstream watersheds contributing to the EMF except for those located within Pinal County. The Build-out models also have all of the capitol improvements in place and include all required mitigating on-site stormwater retention.

2.3.2 Model Descriptions

The build-out models are similar in structure to the 2002 models, with the four watershed models and the one EMF routing model. The differences between the base 2002 HEC-1 models and the base future conditions/build-out models, are as follows:

1) Northwest (NWBUILD): Corresponds to 2002 model NW2002 with the same boundaries. The differences between this model and its 2002 counterpart include some routing changes, the simulation of future development in all contributing subbasins with required on-site stormwater retention, and modeling of the outlet channels of the Central Arizona Project (CAP).
2) Northeast (NEBUILD): Corresponds to 2002 model NE2002 with the same boundaries. The differences between this model and its 2002 counterpart include a few routing differences and the simulation of future development in all contributing subbasins with required on-site stormwater retention.
3) Southeast (MIDDOUT): Corresponds to the 2002 model 2002UP with the same boundaries and with flow routed through the Ellsworth Channel from Pecos Road to the Powerline Floodway. The differences between this model and its 2002 counterpart include the simulation of a new stormwater detention east of Meridian Road and north of Powerline Floodway, the simulation of a new stormwater detention basin at Pecos Road, several routing differences, and the simulation of future development in all contributing subbasins with required on-site stormwater retention.
4) Queen Creek/Sanokai Wash (QCSWBLT): Corresponds to the 2002 model EXQCSW. The differences between this model and its 2002 counterpart include simulation of future development in all contributing Maricopa County subbasins with required on-site stormwater retention, and altered routing of some subbasins (directly east of the EMF) into the EMF instead of into the Queen Creek Wash. Subbasins within Pinal County were not changed for development.
5) EFM Routing Model (BUILTRT): Corresponds to the 2002 model 2002UPRT. The only changes between this model and its 2002 counterpart were made to integrate the changes made in the watershed models.

2.3.3 Results

The results of the Future conditions/Buildout model with all the proposed infrastructure CIP improvements and offline EMF stormwater detention basins in place (including the Guadalupe and Knox Basins), demonstrate that the flow is attenuated sufficiently to contain the 100-year peak flows within the EMF with adequate freeboard in all locations. These results are basis for comparison for the results of the four alternatives.

3.0 HYDRAULIC ANALYSIS

The hydraulic analysis of the EMF was done using the U.S Army Corp of Engineers' HEC-RAS Floodplain model that was compiled during the EMF Capacity Assessment study by HNTB. The model is structured in six reaches, modeling the EMF as it is in existing conditions. The reaches of the EMF are as follows:

- Reach 1: The termination of the EMF at the Gila River to the SR187 Bridge.
- Reach 2: The SR187 to the boundary of Maricopa and Pinal Counties.
- Reach 3: The boundary of Maricopa and Pinal Counties at Hunt Highway to the Queen Creek Road crossing.
- Reach 4: Queen Creek Road to the confluence with the Powerline Floodway near Ray road.
- Reach 5: The confluence of the Powerline Floodway to Broadway Boulevard.
- Reach 6: Broadway Boulevard to the outflow of the Princess Basin north of Brown Road.

All proposed hydraulic changes to the EMF are simulated using this model for evaluation.

According to the HNTB 1999 report, the HEC-RAS model of the EMF, when run with the existing conditions 100-year peak flows, demonstrates overflow along approximately seven miles within the reaches three through five. The same geometry within the HEC-RAS model run with the District's 2002 peak flows demonstrates containment in the EMF but with inadequate freeboard in a few locations. The HEC-RAS model run with the District's Build-out conditions model shows containment of the 100-year peak with adequate freeboard in all locations.

4.0 ALTERNATIVE FORMULATION ANALYSIS

Evaluation of the four alternatives were made using the District's 2002 and Build-out conditions HEC-1 models as a base. Offline stormwater detention basin modifications were made to the routing model, the 2002UPRT for existing conditions and the BUILTRT for future conditions. The only proposed changes within the watershed models are changes made to the Ray Stormwater Detention Basin within the southeast HEC-1 model. Changes to the channel were simulated in the EMF HEC-RAS model.

4.1 COMPONENTS

4.1.1 Channel Improvements

Proposed channel improvements within the alternatives include modifications to the cross section of the EMF, modifications to channel linings for more or less effective conveyance, and grading changes for slope alterations. The grading changes are accomplished by obtaining fall by removing grade control structures. The most significant changes were made to the EMF cross sections and slope in Alternative 4, with an addition of a low flow channel in Alternatives 2 and 3.

4.1.2 Stormwater Detention Basins

Stormwater detention basins are meant to attenuate flow to reduce the design discharge of the EMF. The mitigation alternatives propose locating stormwater detention basins in the upstream watersheds of the EMF (Ray Stormwater Detention Basin), as well as EMF stormwater detention basins (Rittenhouse and Chandler Heights Stormwater Detention Basins), which would provide offline flow attenuation from the EMF itself.

The detention basins are designed using diversion cards based on side weirs within the channels with a specific length and weir crest. All storage is accomplished within the detention basins below this weir crest elevation. Consequently, all flow within the channels over the weir crest elevations is diverted into the basins. The weir crest elevations and lengths were designed to meet the dual criteria, both to contain the peak 100-year flow within the EMF in the Existing Conditions, and to contain the peak 100-year flow in the EMF with required freeboard in Build-out conditions. Once both criteria were met, the design was used in both conditions' models.

4.1.2.1 Upstream Watershed Stormwater Detention Basins

Upstream watershed stormwater detention basins are intended to be regional basins located within the upstream watersheds of the EMF. The mitigation alternatives include their location in areas where right-of-way has already been or is likely to be acquired within the near future. They include the Ray Basin. The CAP stormwater detention basins and the Elliot Basin are in place in the base model and all the alternatives. These stormwater detention basins are not part of the analysis.

4.1.2.2 EMF Stormwater Detention Basins

EMF stormwater detention basins are intended to detain flow directly from the EMF. These offline basins would divert a portion of the flow within the EMF with a diversion structure within the channel.

4.2 SELECTED ALTERNATIVES

4.2.1 Alternative 1

Alternative 1, also known as the "strictly-flood-control alternative" involves only the construction of stormwater detention basins, both upstream watershed and EMF, and no changes to the hydraulics of the EMF. The stormwater detention basins are modeled realistically within the HEC-1 model, using diversion rating curves based on proposed weir specifications, channel geometry and flow depth, and stage/storage relationships based on the dimensions of the proposed stormwater detention basins. Outlet structures were designed assuming no flow out of the detention basins until after the storm flow. The goal of the outlet structure design for each basin is to drain the basin in 36-hours to a depth of under 6-inches above the outlet invert. The remaining water is assumed to infiltrate into the basin. Alternative 1 includes the following changes to the 2002 and Build-out hydrologic and hydraulic models:

1) The EMF offline Guadalupe and Knox stormwater detention basins, originally included in the base hydrologic models, were deleted.

This change was made because the District does not plan to acquire the land necessary for each basin.

2) The Ray Basin, an upstream watershed stormwater detention basin, has been expanded from its proposed volume within the base 2002 and Build-out models. In the base 2002 and Build-out models, the required storage volume of the Ray Basin is 874 and 316 acre-feet respectively, using an area of 114 acres. Instead of diverting flow from just the Powerline Floodway, Alternative 1 model simulates the diversion of flow from both the Powerline Floodway and the San Tan Freeway channel into the Ray Basin. See Table 2 for the summary of the Alternative 1, Ray Stormwater Detention Basin Specifications.

The Ray Basin diversion within the San Tan Freeway Channel consistes of a side weir with a length of 200 feet and a weir crest height of 4.50 feet above the channel invert. The diversion within

Table 2: Alternative 1 Specifications for Ray SDB

Alternative 1 Ray Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	9.25	
Top of basin from basin invert (ft) =	15.7	
Total area of detention basin (Acres) =	171	
Total Capacity (Ac-ft) =	1537	
Head to pump (ft) =	2	
Volume to Pump (Ac-ft) =	318	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	1532	780
Total div from San Tan Chnl	444	267
Total div from Powerline	1088	513
Surplus Capacity (Ac-ft) =	5	757
Side Weirs		
From San Tan Channel		
Weir Crest Height (ft) =	4.5	
Weir Length (ft) =	200	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	4	
From Powerline Floodway		
Weir Crest Height (ft) =	3.25	
Weir Length (ft) =	750	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	4	
Drainage of Detention Basin		
Head to Pump (ft) =	2	
Volume to Pump (Ac-ft) =	318	
Pump Style =	Screw	
Pump Quantity =	2	
Drainage Culvert =	9-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	1214	

the Powerline floodway consists of a side weir with a length of 750 feet and weir crest height of 3.25 feet above the channel invert. To obtain this geometry and maximize the available depth in the Ray Stormwater Detention Basin and minimize depth below the outlet structure (ponded water in the basin that must be pumped back into the Powerline Floodway), the depth of flow in each of the feeding channels (the San Tan Freeway Channel and the Powerline Floodway) must be raised four feet above its normal depth, Though many schemes are available to raise the channel water surface by four feet, one technique involves building a concrete diversion dam in the channel, then back-filling upstream of the dam to prevent water from collecting behind the diversion dam (Figure 4).

In Alternative 1, the area of the Ray Basin is 171 acres with a total depth of 15.70 feet (9.25 feet from invert of basin to minimum weir crest and 6.45 feet of freeboard). This includes 2 feet of depth below the outlet culverts that must be pumped for evacuation. In the Alternative 1, 2002 and Build-out models, the required diverted storage volume has increased to 1532 (444 from San Tan and 1088 from Powerline) and 780 (267 from San Tan and 513 from Powerline) acre-feet respectively. In the Build-out conditions, the basin will have 757 acre-feet of surplus capacity. Surplus capacity is defined as the total extra detention basin volume not used during a given flood event. Note that while the basins are nearly completely filled in a 2002 event, each basin has significant unused capacity during the build-out event.

To drain the basin with the 2002 volume in the requisite 36 hours, the outlet structure for the Ray Stormwater Detention Basin into the Powerline Floodway has been proposed to be a 9 cell 6' x 3' reinforced concrete box culvert (RCBC). Flap-gates will be placed on the drainage channel end of the culverts to ensure that no backflow will go from the channel into the detention basins through the outlet structures. To drain the 2-feet of ponded water within Ray Stormwater Detention Basin below the invert of the outlet structure, a 2-pump station is proposed. See Figure 5 for the Alternative 1, Ray Stormwater Detention Basins stormwater detention basin plan, weir specifications and detention basin cross sections.

3) Alternative 1 keeps the EMF offline Rittenhouse Stormwater Detention Basin in place with revised specifications. The Rittenhouse Stormwater Detention Basin accepts diverted flow from both the EMF and the Rittenhouse channel. In the base HEC-1 2002 model, the Rittenhouse Stormwater Detention Basin diverts a total volume of 1026 acre-feet, including 168 acre-feet from the Rittenhouse Channel and 858 acre-feet from the EMF. In the base Build-out model, the Rittenhouse Stormwater Detention Basin diverts a total volume of 460 acre-feet, including 26 acre-feet from the Rittenhouse Channel and 434 acre-feet from the EMF. These storage volumes have been revised in the Alternative 1 model. See Table 3 for a complete summary of the Rittenhouse Stormwater Detention Basin's specifications.

In the Alternative 1, 2002 model, the required diverted storage volume is 587 acre-feet, which includes 267 acre-feet diverted from the Rittenhouse Channel and 320 acre-feet diverted from the EMF. In the Build-out model, the required diverted storage volume in the Rittenhouse Stormwater Detention Basin is 110 acre-feet, which includes 56 acre-feet from the Rittenhouse Channel and 54 acre-feet from the EMF.

Within Alternative 1, the area of the Rittenhouse Basin has been set to 111 acres with a total depth of 11.3 feet, which includes 6.5 feet of available depth from the invert of the basin to the minimum weir crest and 4.8 feet of freeboard.

In Alternative 1, the diversion structure within the Rittenhouse Channel is a side weir with a length of 150 feet and a crest height of 2.00 feet above the channel invert. The diversion structure within the EMF is a side weir with a length of 1500 feet and a crest height of 6.50 feet above the channel invert. To obtain this geometry, maximize the available depth in the Rittenhouse Stormwater Detention Basin, and minimize the depth below the outlet structure (ponded water that must be pumped back into the EMF), the depth of flow in the Rittenhouse Channel must be raised 1.17 feet above normal depth, either through a diversion structure or another method.

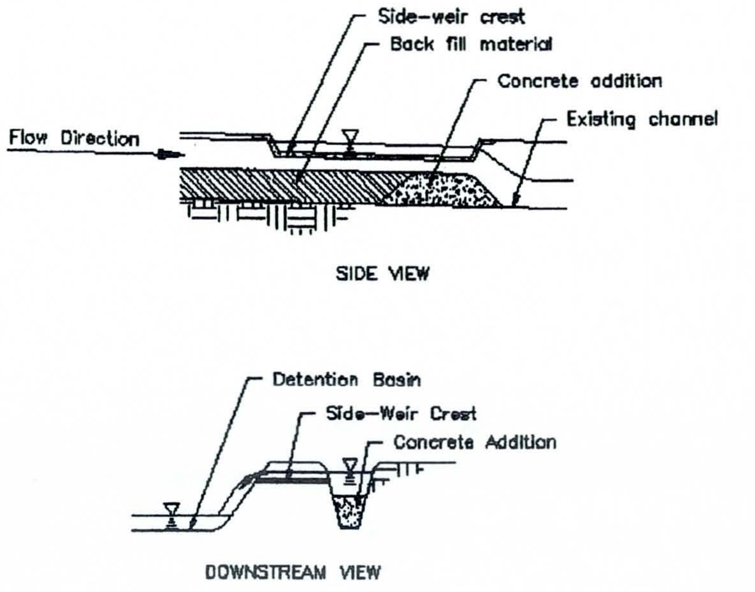
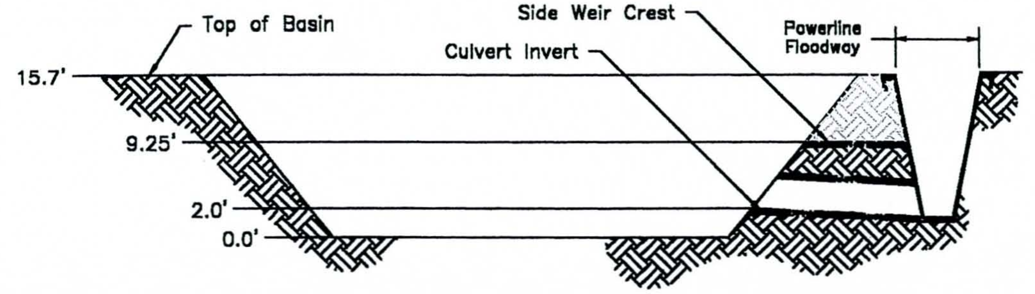


Figure 4: Proposed diversion structure within channel to raise water surface.



Ray Detention Basin Relative to Powerline Floodway
(NTS)

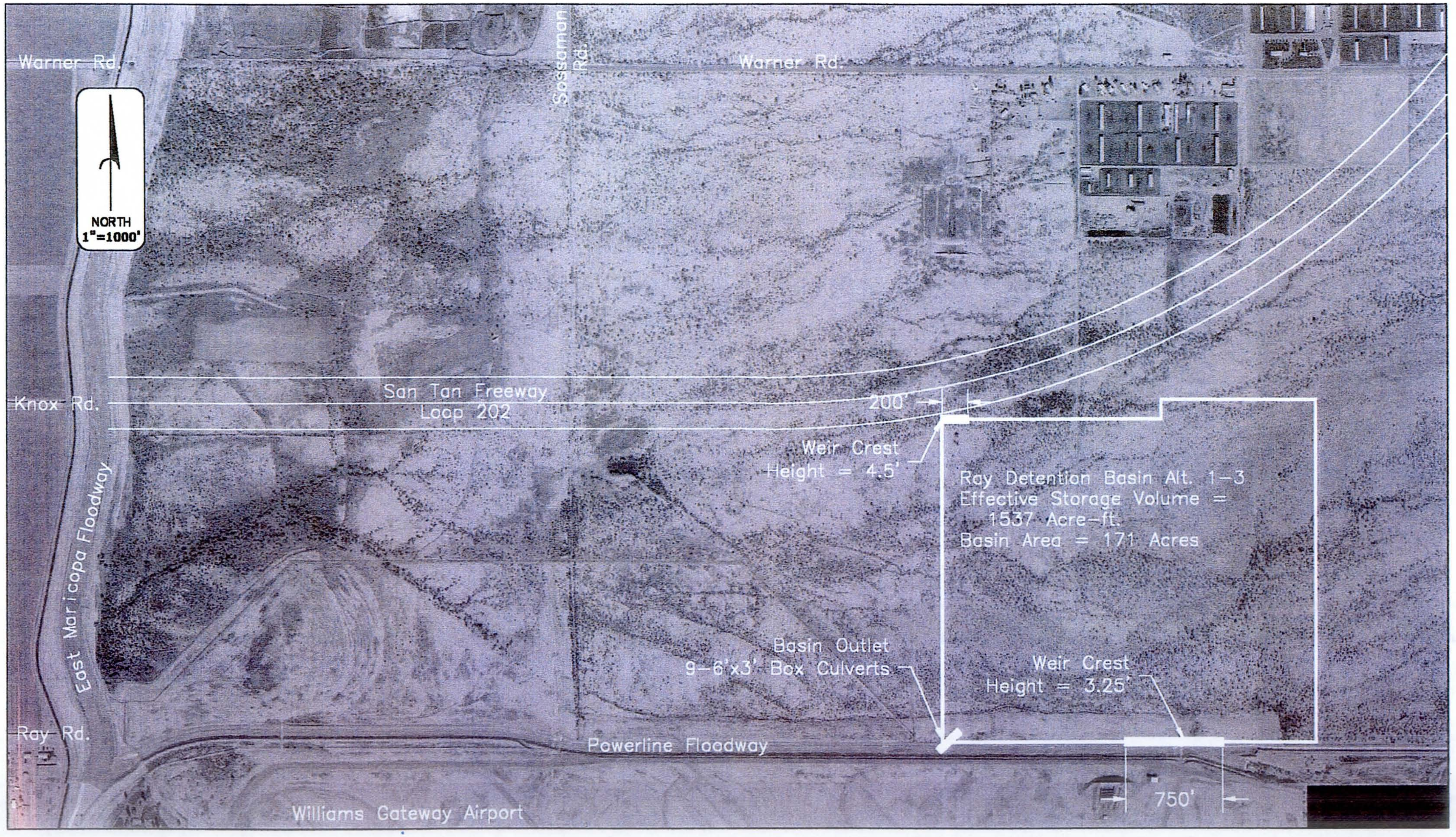


Figure 5
Alternative 1-3
Ray Stormwater
Detention
Basin
Specifications

Table 3: Alternative 1 Specifications for Rittenhouse SDB

Alternative 1 Rittenhouse Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	6.5	
Top of basin from basin invert (ft) =	11.3	
Total area of detention basin (Acres) =	111	
Total Capacity (Ac-ft) =	690	
Head to pump (ft) =	0	
Volume to Pump (Ac-ft) =	0	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	587	110
Total div from EMF	320	54
Total div from Ritt	267	56
Surplus Capacity (Ac-ft) =	103	580
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	6.5	
Weir Length (ft) =	1500	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
From Rittenhouse Channel		
Weir Crest Height (ft) =	2	
Weir Length (ft) =	150	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	1.17	
Drainage of Detention Basin		
Head to Pump (ft) =	0	
Volume to Pump (Ac-ft) =	0	
Pump Style =	N/A	
Pump Quantity =	N/A	
Drainage Culvert =	5-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	587	

To drain the basin with the 2002 storage volume in the requisite 36 hours, the outlet structure for the Rittenhouse Stormwater Detention Basin has been proposed to be a 5 cell 6' x 3' RCBC. No pumps are necessary for Rittenhouse Stormwater Detention Basin in Alternative 1.

See Figure 6 for the Alternative 1, Rittenhouse Stormwater Detention Basin plan, weir specifications and detention basin cross sections.

4) Alternative 1 keeps the EMF offline Chandler Heights Stormwater Detention Basin in place. The Chandler Heights Stormwater Detention Basin diverts flow from both the EMF and the Queen Creek channel. In the base 2002 model, 1470 acre-feet is diverted from the Queen Creek Channel and 868 acre-feet is diverted from the EMF, for a total of 2338 acre-feet of storage volume in the Chandler Heights Stormwater Detention Basin. In the base Build-out model, 1475 acre-feet is diverted from the Queen Creek Channel and 601 acre-feet is diverted from the EMF, for a total of 2076 acre-feet of storage volume in the Chandler Heights Stormwater Detention Basin. In the Alternative 1 model, these storage volumes have been changed. See Table 4 for a complete summary of the specifications of the Alternative 1 proposed Chandler Heights Stormwater Detention Basin.

The diversion structure to the Chandler Heights Stormwater Detention Basin within the Queen Creek Channel is a side weir with a length of 500 feet and a crest height of 6.20 feet above the channel invert. The diversion structure within the EMF is a side weir with a length of 1000 feet and a crest height of 5.00 feet above the channel invert. No additional depth is necessary within either channel for this geometry.

The area of the Chandler Heights Detention Basin within Alternative 1 is 300 acres with a total depth of 13.9 feet. This includes 5.00 feet of available depth from the minimum weir crest, 8.9 feet of freeboard, and no depth below the invert of the outlet structure. In the Alternative 1, 2002 model, a total of 1249 acre-feet is diverted into the Chandler Heights Stormwater Detention Basin, including 540 acre-feet from Queen Creek and 709 acre-feet from the EMF. In the Alternative 1 Build-out model, the necessary storage volume within the Chandler Heights Stormwater Detention Basin is 804 acre-feet, including 515 acre-feet from the Queen Creek and 289 acre-feet from the EMF.

To drain the basin with the 2002 storage volume in the requisite 36 hours, the outlet structure for the Chandler Heights Stormwater Detention Basin has been proposed to be a 13 cell 6' x 3' RCBC. No pumps are necessary for the Chandler Heights Stormwater Detention Basin in Alternative 1.

See Figure 7 for the Alternative 1, Chandler Heights Stormwater Detention Basin plan, weir specifications and detention basin cross sections.

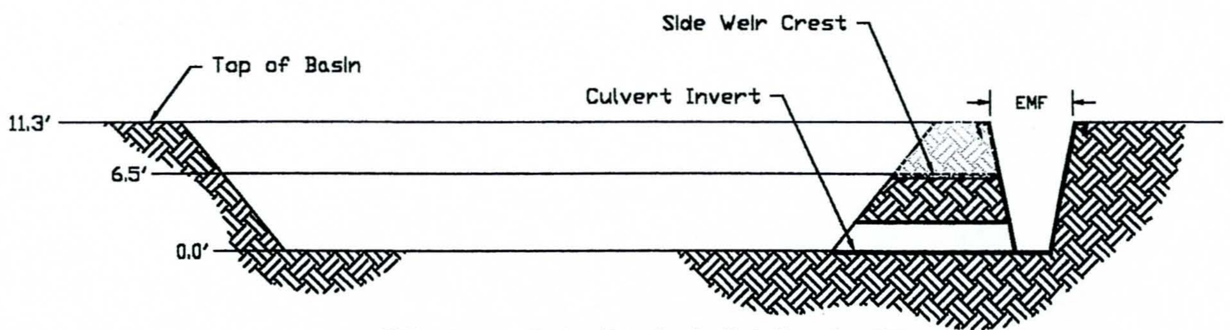
5) In Alternative 1, the original channel geometry of the EMF was used in all HEC-RAS modeling. No modifications were made to the channel.

Table 4: Alternative 1 Chandler Heights Stormwater Detention Basin

Alternative 1 Chandler Heights Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	5	
Top of basin from basin invert (ft) =	13.9	
Total area of detention basin (Acres) =	300	
Total Capacity (Ac-ft) =	1415	
Head to pump (ft) =	0	
Volume to Pump (Ac-ft) =	0	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	1249	804
Total div from EMF	709	289
Total div from Queen Creek	540	515
Surplus Capacity (Ac-ft) =	166	611
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	5	
Weir Length (ft) =	1000	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	none	
From Queen Creek		
Weir Crest Height (ft) =	6.2	
Weir Length (ft) =	500	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
Drainage of Detention Basin		
Head to Pump (ft) =	0	
Volume to Pump (Ac-ft) =	0	
Pump Style =	N/A	
Pump Quantity =	N/A	
Drainage Culvert =	13-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	1249	

4.2.1.1 Mitigation Effectiveness

Alternative 1 is effective in attenuating the runoff within the EMF so that the peak 100-year discharge, in the Existing/2002 conditions, is contained within the channel. The 100-year peak flow in the Future/Build-out conditions is contained within the channel with adequate freeboard at all locations. The elimination of the Guadalupe and Knox basins does result in an increase in peak flow, and therefore water surface elevation, in the EMF from the location of the proposed Guadalupe Stormwater Detention Basin northeast of the intersection of Power Road and Guadalupe Road, and the inlet of the Powerline Floodway. However, the increase in depth and freeboard requirement does not overwhelm the floodway.



Rittenhouse Detention Basin Relative to EMF
(NTS)

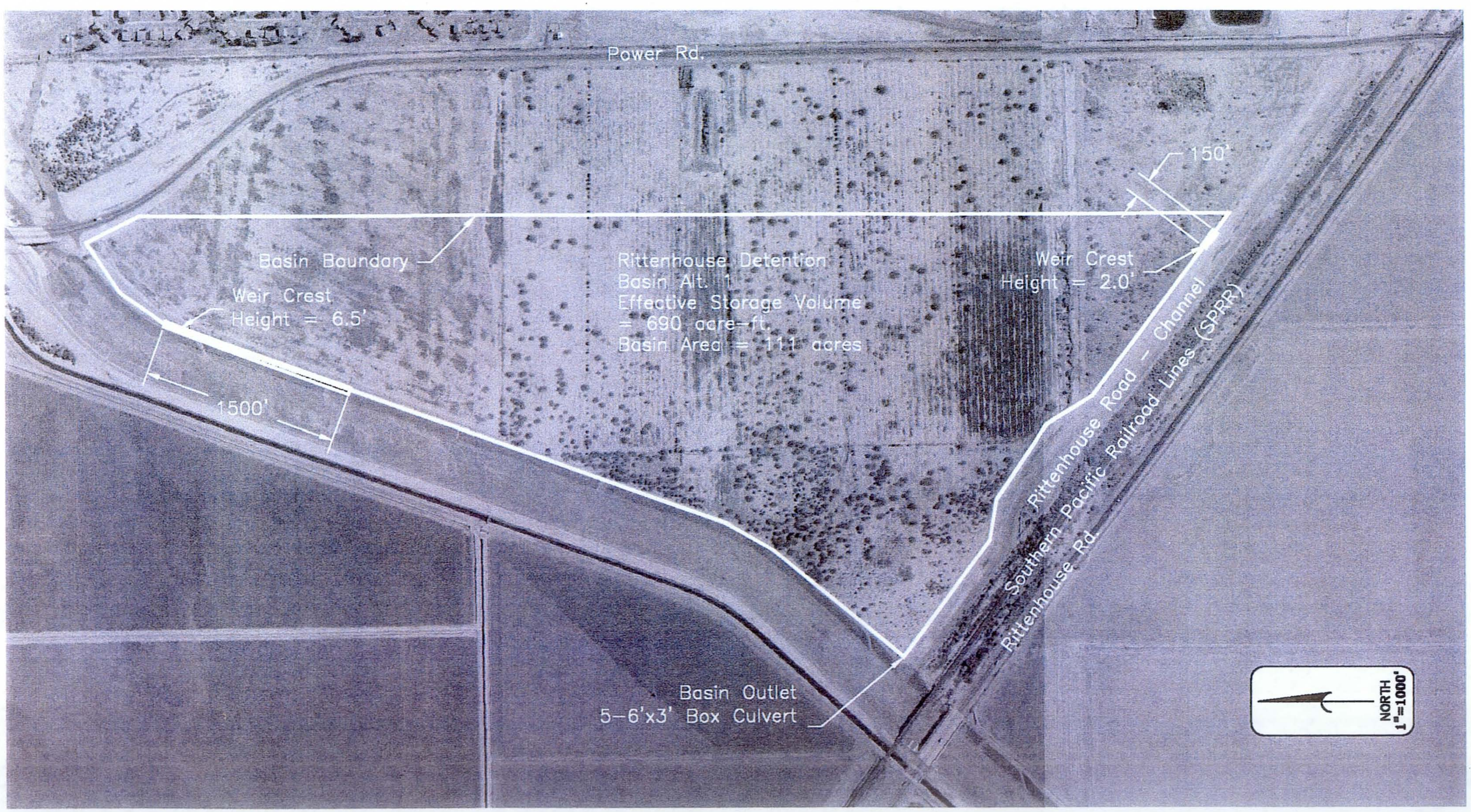


Figure 6
Alternative 1
Rittenhouse
Stormwater
Detention
Basin
Specifications

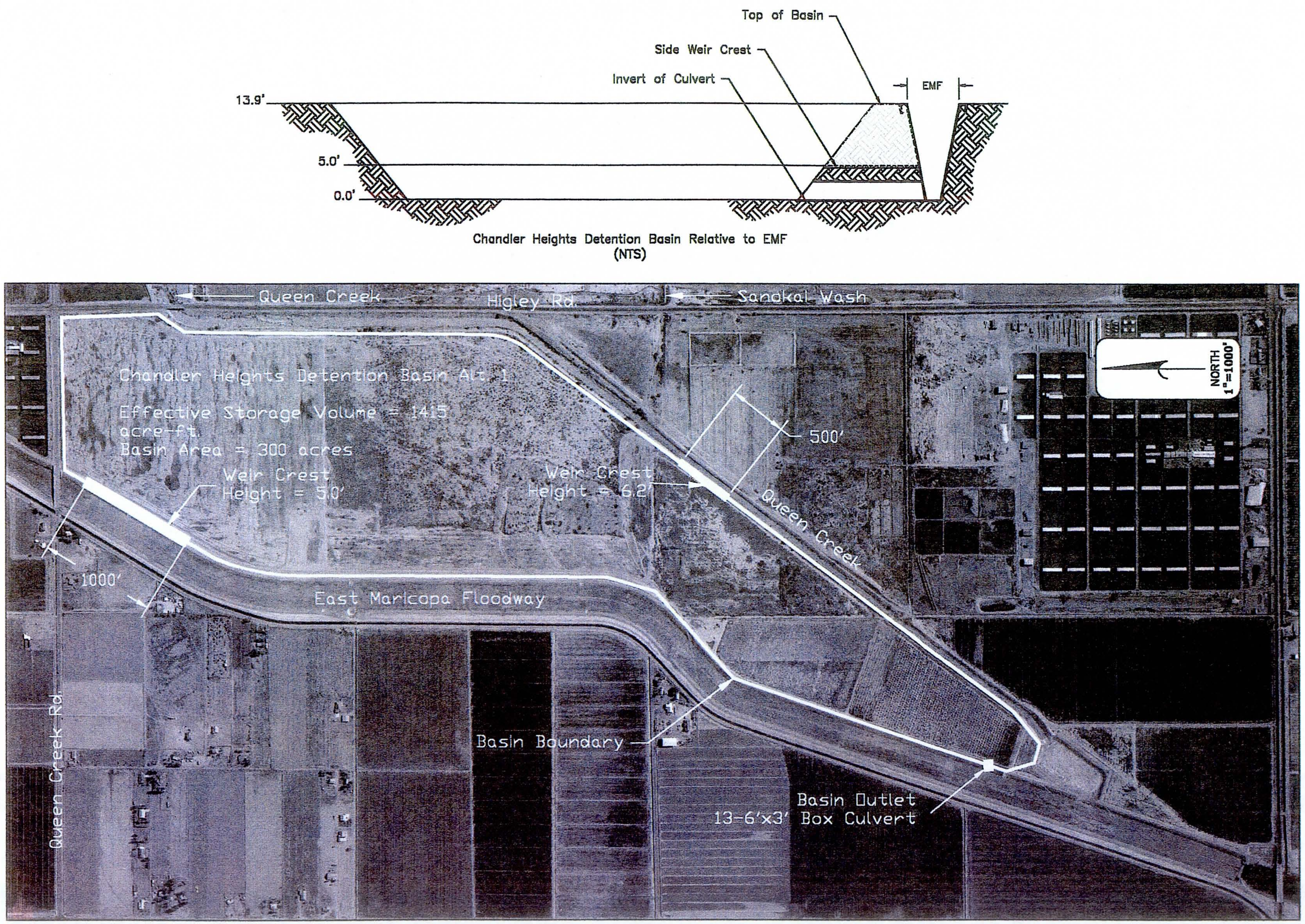


Figure 7
Alternative 1
Chandler
Heights
Stormwater
Detention
Basin
Specifications

Table 5: Alternative 1 Construction and Engineering Costs

Item	Construction	Land Acquisition	Design & Contingency	Total
Ray Detention Basin	$20,652,000	$5,130,000	$4,750,000	$30,532,000
Rittenhouse Detention Basin	$11,878,200	$3,330,000	$2,732,000	$17,940,200
Chandler Heights Detention Basin	$31,900,000	$1,950,000	$7,337,000	$41,187,000
Total	$64,430,200	$10,410,000	$14,819,000	$89,659,200

4.2.1.2 Cost Estimate

Table 5 presents the projected cost estimate for the flood control mitigation associated with Alternative 1. No landscaping was incorporated into this cost estimate.

4.2.1.3 Multi-Use Potential

No multi-use potential has been incorporated into Alternative 1. It is strictly the "engineering alternative".

4.2.1.4 Recommendations for Further Consideration

Alternative 1 is the "no-frills" alternative that strictly addresses flood control within the EMF and its watersheds. It is used as the basis for Alternatives 2 and 3 that incorporate recreational and environmental amenities. It is effective in its flood control objective and meets all the design criteria.

4.2.2 Alternative 2

Alternative 2, known as the recreational alternative, involves stormwater detention basins, both watershed and EMF, and changes to the channel of the EMF. The stormwater detention basins are modeled realistically as they are in Alternative 1. Alternative 2 includes the following changes to the 2002 and Build-out hydrologic and hydraulic models:

1) The EMF offline Guadalupe and Knox stormwater detention basins, originally included in the base hydrologic models, were deleted.

2) The Ray Stormwater Detention Basin, an upstream watershed stormwater detention basin, has been expanded from its proposed volume within the base 2002 and Build-out models. Its design for Alternative 2 is identical as it is in Alternative 1. See Table 2 and Figure 5 in Alternative 1 for a full description.

3) Alternative 2 keeps the EMF offline Rittenhouse Stormwater Detention Basin in place and includes in its design amenities such as ball fields, parking structures, support buildings and additional recreational facilities. Some of these amenities deduct from the available detention storage. Because of these changes to the stage-storage relationships and because of the addition of vegetation to the EMF channel, the Rittenhouse Detention Basin specifications have changed from Alternative 1. See Table 6 for a complete summary of the Alternative 2 Rittenhouse Stormwater Detention Basin's specifications.

In Alternative 2, the area of Rittenhouse Stormwater Detention Basin is increased to 179 acres with a total depth of 15.80 feet. This includes 10.85 feet from the invert of the basin to the minimum weir crest height and 4.95 feet of freeboard. The total storage capacity of the basin, from its invert to the minimum weir crest height is 982 acre-feet. The side weir along the EMF is still 1500 feet long but the crest is only 6.35 feet. A 150 foot long weir with a crest height of 2 feet above the invert of the channel diverts flow from the Rittenhouse Channel.

In the 100-year, 2002 Alternative 2 HEC-1 model, the total flow diverted to the Rittenhouse Stormwater Detention Basin is 898 acre feet. In the 100-year Build-out model, the total flow diverted is 287 acre-feet,

In the geometry and flow rates of the design, a total of 4.5 feet of the Rittenhouse Stormwater Detention Basin is below the invert of the outlet structures draining the basin, which will require the draining of 382 acre-feet by pump. In addition, the water surface of Rittenhouse Channel will have to be raised 1.17 feet above normal depth by The outlet structure to the Rittenhouse Stormwater Detention Basin is proposed to be a 7-cell 6'x3' RCBC. The downstream outlets to the culverts will have flap gates to prevent water from flowing from the EMF to the detention basin through the culverts. A 2-pump station will be required to pump the 382 acre-feet of water in the detention basin below the outlet culverts.

See Figure 8 for the Alternative 2, Rittenhouse Stormwater Detention Basin plan, weir specifications and detention basin cross sections.

4) Alternative 2 keeps the EMF offline Chandler Heights Stormwater Detention Basin in place and includes amenities within the basin itself. As with the Rittenhouse Stormwater Detention Basin, its specifications have changed somewhat because of the stage-storage modifications within the detention basin and different flow requirements within the EMF due to channel modifications. See Table 7 for a complete summary of the specifications of the Alternative 2 proposed Chandler Heights Stormwater Detention Basin.

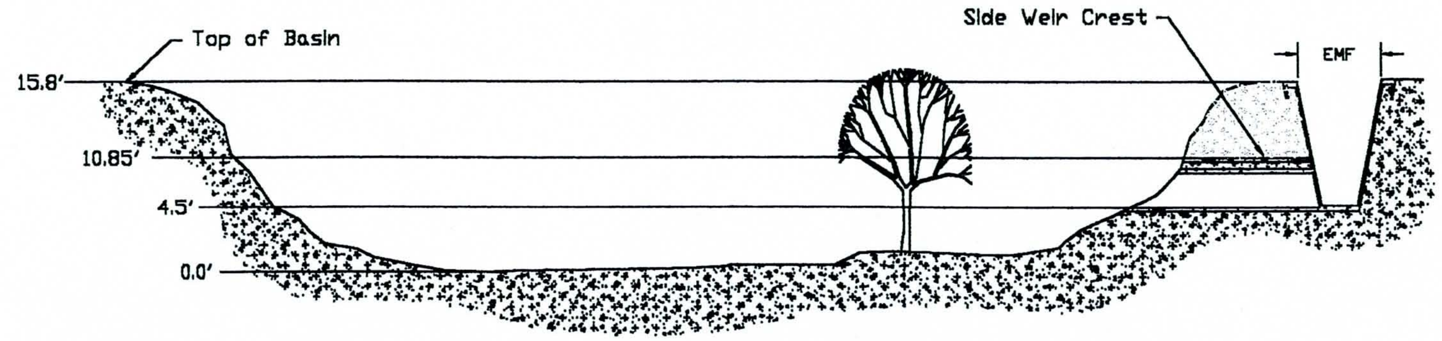
The Alternative 2 Chandler Heights Stormwater Detention Basin has an area of 300 acres with a depth of 23.90 feet, which includes 15 feet of depth from the invert to the minimum weir crest and 8.90 feet of freeboard. The total capacity of the basin from its invert to the minimum weir crest is 1591 acre-feet. A side weir with a length of 1000 feet and a crest height of 5 feet above the invert of the channel diverts flow from the EMF. A side weir with a length 500 feet and a crest height of 6.25 feet above the invert of the channel diverts flow from the Queen Creek.

In the Alternative 2, 2002 HEC-1 model, a total of 1593 acre-feet is diverted into the Chandler Heights Detention Basin. In the Alternative 2, Build-out model, a total of 1239 acre-feet of flow is diverted to the Chandler Heights Stormwater Detention Basin, including 767 acre-feet from the EMF and 472 acre-feet from Queen Creek.

To obtain these geometry and flow rates, the detention basin invert was set 10-feet below the invert of the outlet structures draining the basin. Because of this, 584 acre-feet will have to be pumped from the basin back into the EMF.

Table 6: Alternative 2 Specifications for Rittenhouse SDB

Alternative 2 Rittenhouse Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	10.85	
Top of basin from basin invert (ft) =	15.8	
Total area of detention basin (Acres) =	179	
Total Capacity (Ac-ft) =	982	
Head to pump (ft) =	4.5	
Volume to Pump (Ac-ft) =	382	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	898	287
Total div from EMF	631	231
Total div from Ritt	267	56
Surplus Capacity (Ac-ft) =	84	695
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	6.35	
Weir Length (ft) =	1500	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
From Rittenhouse Channel		
Weir Crest Height (ft) =	2	
Weir Length (ft) =	150	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	1.17	
Drainage of Detention Basin		
Head to Pump (ft) =	4.5	
Volume to Pump (Ac-ft) =	382	
Pump Style =	Screw	
Pump Quantity =	2	
Drainage Culvert =	7-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	516	



Rittenhouse Detention Basin Relative to EMF
(NTS)

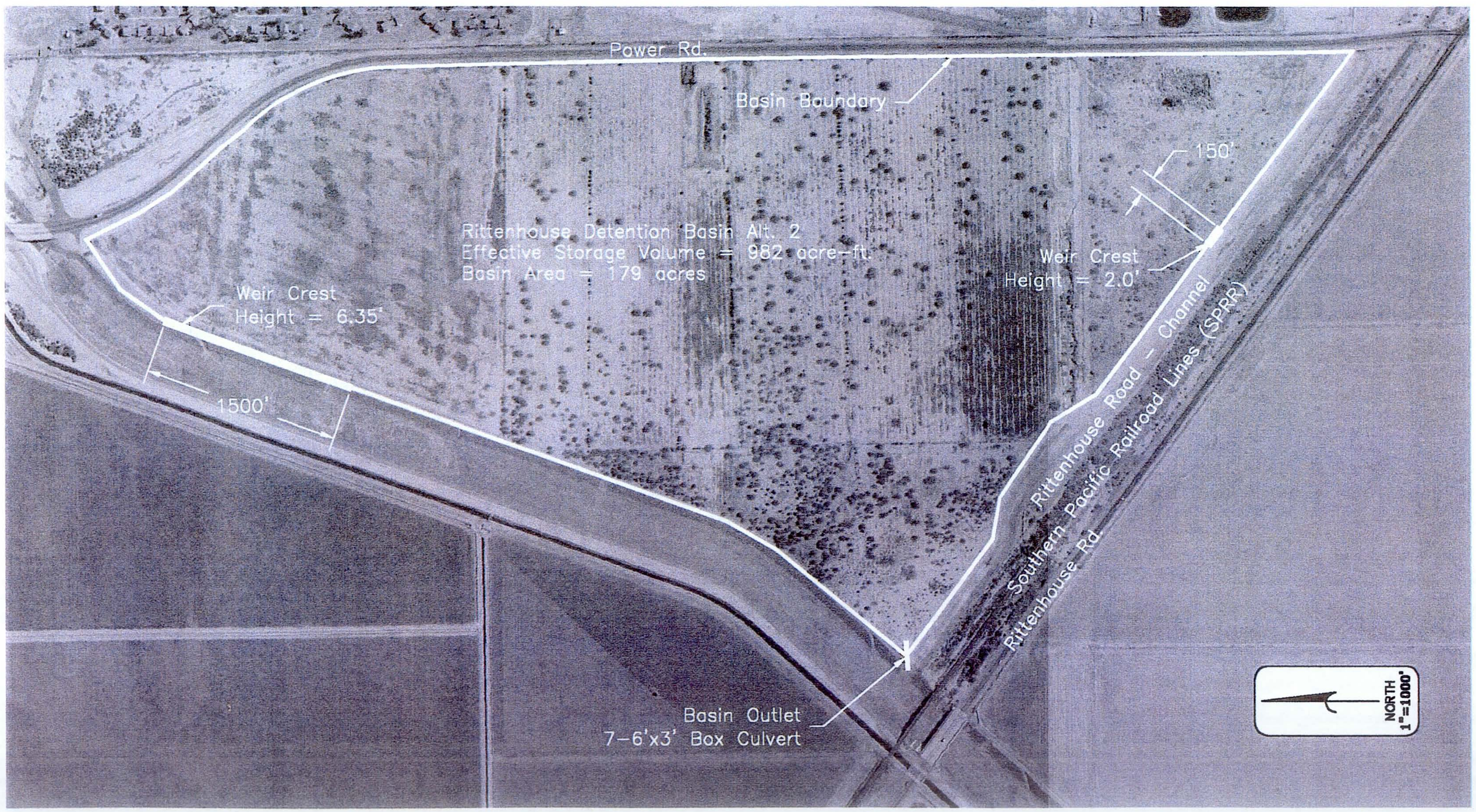


Figure 8
Alternative 2
Rittenhouse
Stormwater
Detention
Basin
Specifications

Table 7: Alternative 2 Specifications for Chandler Heights SDB

Alternative 2 Chandler Heights Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	15	
Top of basin from basin invert (ft) =	23.9	
Total area of detention basin (Acres) =	300	
Total Capacity (Ac-ft) =	1591	
Head to pump (ft) =	10	
Volume to Pump (Ac-ft) =	584	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	1593	1239
Total div from EMF	1093	767
Total div from Queen Creek	500	472
Surplus Capacity (Ac-ft) =	-2	352
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	5	
Weir Length (ft) =	1000	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
From Queen Creek		
Weir Crest Height (ft) =	6.25	
Weir Length (ft) =	500	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
Drainage of Detention Basin		
Head to Pump (ft) =	10	
Volume to Pump (Ac-ft) =	584	
Pump Style =	Screw	
Pump Quantity =	3	
Drainage Culvert =	11-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	1009	

To drain the basin and transport the detained volume back to the EMF in the required 36-hour time period, the outlet structure is proposed to be a 11-cell 6'x3' RCBC. In addition, a 3-pump station will be required to drain the 584 acre-feet of stored water below the invert of the outlet culverts.

See Figure 9 for the Alternative 2, Chandler Heights Stormwater Detention Basin plan, weir specifications and detention basin cross sections.

5) Channel modifications to the EMF were made to the HEC-RAS model. These include a low-flow meandering channel at the following locations: Brown Road to Broadway Road, Guadalupe Road to Ray Road, and Power Road to Hunt Highway (Figure 10). Assuming that the presence of the low-flow channel would concentrate water and lead to increased grass and weed growth, the Manning's coefficient in all low-flow channel sections was increased to 0.03. In order to simulate landscape vegetation within the complete channel, Manning's coefficient was increased to 0.040 in the sections from Brown Road to Broadway Road and from the energy-drop structure just above Germann Road to Chandler Heights Road. The Manning's coefficient was also increased to 0.035 in the section from Guadalupe Road to Ray Road. This section was more sensitive to increased Manning's coefficient, hence the value was increased to only 0.035. Tables in the Appendix show all Manning's roughness values used in each alternative for all river stations above Hunt Highway.

Note that in the landscaping of Alternative 2, vegetation was placed only within specific reaches of the EMF Channel. Moreover, note the placement of landscaping is less extensive in Alternative 2 that in Alternative 3 (Figure 10). The decision to reduce the extent of landscaping in Alternative 2 was tied to the fact that the EMF sections below Chandler Heights Road and between Power Road and the energy-drop structure above Germann Road are hydraulically sensitive to increased Manning's coefficients. Adding vegetation below Chandler Heights Road acts to slow flow and increase the overall peak discharge at Hunt Highway. Similarly, the added vegetation above the drop structure slows passage of the flood, increasing peak discharge and violation channel breakout and freeboard design criteria near Williams Field. One can still make the overall design work with vegetation in these sensitive sections (see Alternative 3); however, the overall capacity of the Rittenhouse and Chandler Heights Detention Basins must be significantly larger to mitigate flood impacts. The cost savings of Alternative 2 over Alternative 3 are directly tied to not placing landscape vegetation in these hydraulically sensitive reaches of the EMF.

To simulate the increased vegetation within the HEC-1 model, roughness coefficients within the routing reaches were increased to match the changes implemented on the HEC-RAS models.

4.2.2.1 Mitigation Effectiveness

The mitigation within Alternative 2 is effective in attenuating the runoff reaching the EMF so that the peak 2002 100-year discharge is contained within the EMF and the peak Build-out 100-year flow is contained within the required freeboard.

4.2.2.2 Cost Estimate

Table 8 presents the projected cost estimate for the mitigation associated with Alternative 2.

4.2.2.3 Multi-Use Potential

Alternative 2 incorporates substantial multi-use potential in the form of recreation amenities; both within the EMF offline stormwater detention basins and within the EMF channel itself. The recreational facilities included in the design are as follows:

1) The design of the Rittenhouse Stormwater Detention Basin incorporates ball fields, hiking trails and park facilities, including covered and uncovered picnic areas and support facilities.
2) The design of the Chandler Heights Stormwater Detention Basin incorporates similar facilities to the Rittenhouse Stormwater Detention Basin.
3) The EMF channel design has a low flow meandering channel along the following stretches:
   - Brown Road to Broadway Road.
   - Guadalupe Road to Ray Road.
   - Power Road to Hunt Highway.

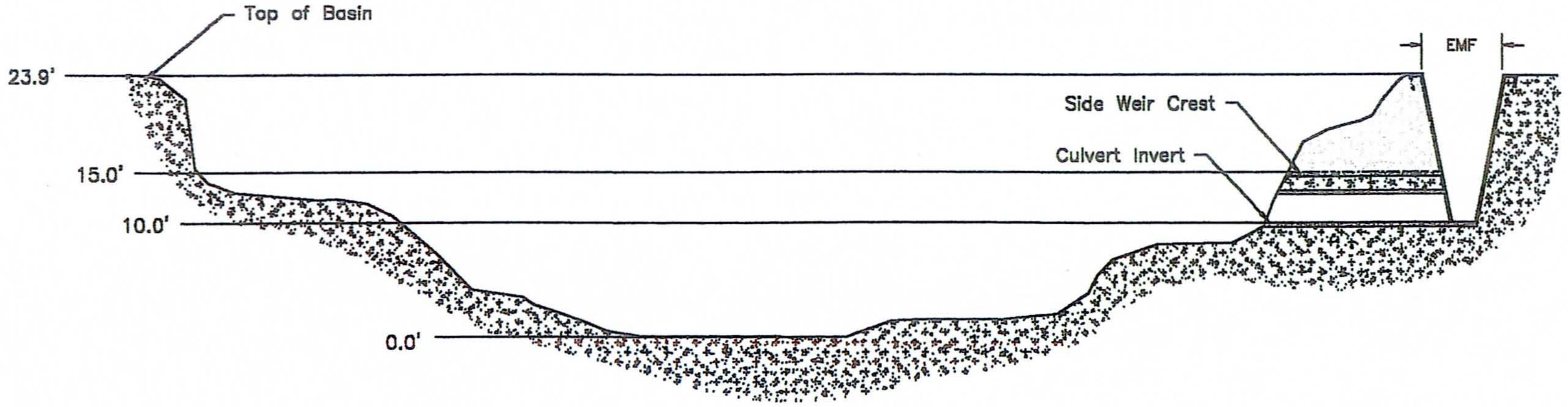
   The meandering channel will be trapezoidal in cross section. Some of the stretches of the EMF that have been proposed to include a low-flow channel, including Brown Road to Broadway, Guadalupe to Ray Road and Germann Road to Chandler Heights Road, will also include revegetation and visual enhancements. See Figure 10 for the channel modification plan.
4) Vegetation, represented by larger roughness coefficients, was added to the models along the following stretches:
   - Brown Road to Broadway Road.
   - Guadalupe Road to Ray Road.
   - Energy-drop structure above Germann Road to Chandler Heights Road.

4.2.2.4 Recommendations for Further Consideration

Alternative 2 is a viable alternative that does not include extensive changes to the channel and no changes that violate the District's requirements.

Table 8: Alternative 2 Construction and Engineering Costs

Item	Construction	Land Acquisition	Design & Contingency	Total
Ray Detention Basin	$20,652,000	$5,130,000	$4,750,000	$30,532,000
Rittenhouse Detention Basin	$23,183,000	$5,370,000	$5,332,000	$33,885,000
Chandler Heights Detention Basin	$53,566,000	$1,950,000	$13,320,000	$68,836,000
Low-Flow Channel	$419,271	$33,542	$62,891	$515,704
Total	$97,820,271	$12,483,542	$23,464,891	$133,768,704



Chandler Heights Detention Basin Relative to EMF
(NTS)

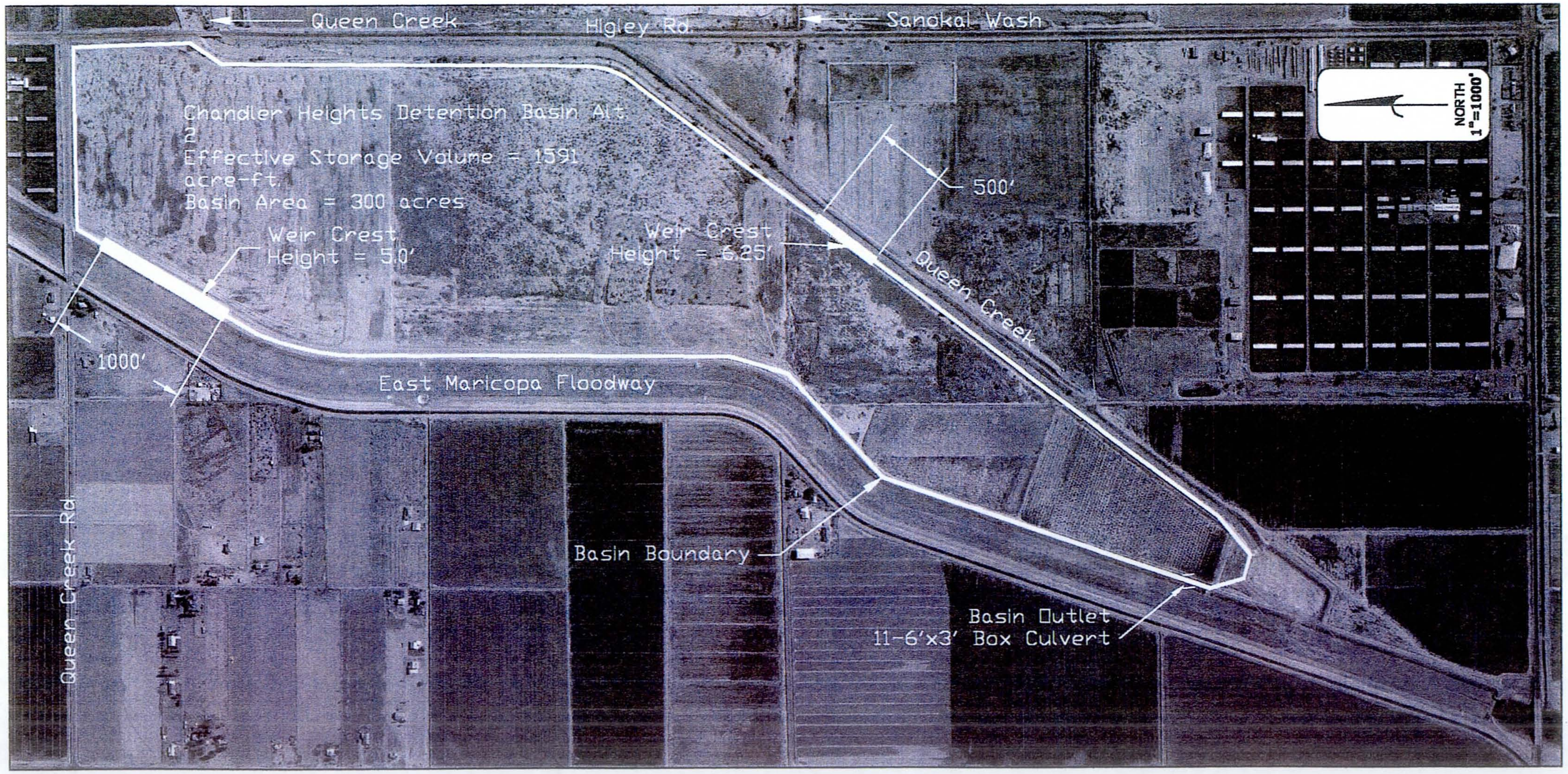


Figure 9
Alternative 2
Chandler Heights Stormwater Detention Basin Specifications

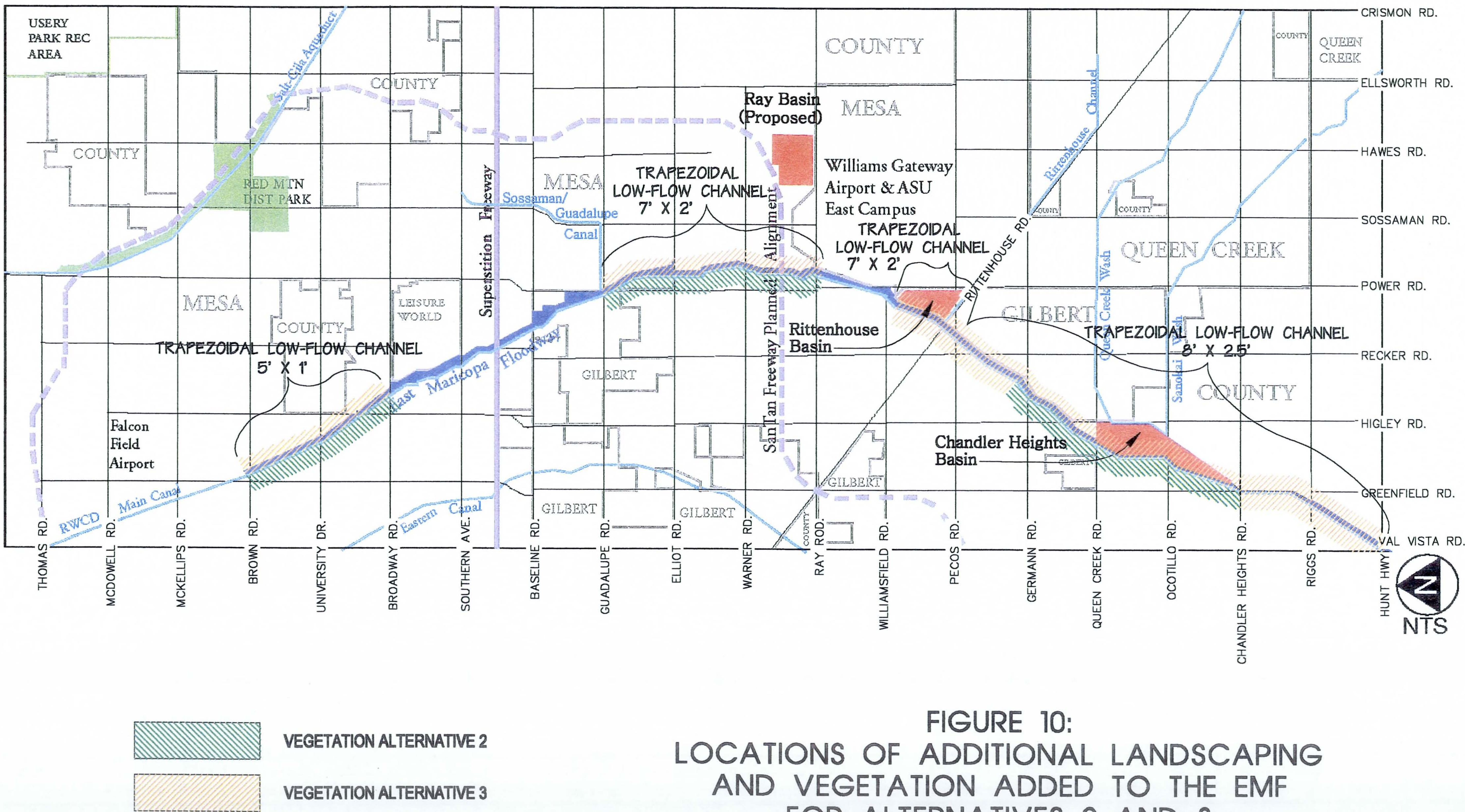


FIGURE 10:
LOCATIONS OF ADDITIONAL LANDSCAPING AND VEGETATION ADDED TO THE EMF FOR ALTERNATIVES 2 AND 3

4.2.3 *Alternative 3*

Alternative 3 is a multi-modal, environmental alternative that offers opportunities for enhanced vegetation in the EMF and detention basins, wetlands, and possible groundwater recharge. The Alternative involves both stormwater detention basins, including upstream watershed and diversions from the EMF itself, and changes to the hydraulics of the EMF. The detention basins are modeled realistically as in Alternatives 1 and 2. Alternative 3 includes the following changes to the hydrologic models:

1) The EMF offline Guadalupe and Knox stormwater detention basins, originally included in the base hydrologic models, were deleted.

2) The Ray Basin, an upstream watershed stormwater detention basin, has been expanded from its proposed volume within the base 2002 and Build-out models and diverts flow from both the San Tan Freeway Channel and the Powerline Floodway, as in Alternatives 1 and 2. The design of the Ray Stormwater Detention Basin for Alternative 3 is identical to its design in Alternatives 1 and 2. See Table 2 and Figure 5 in the Alternative 1 section for a full description.

3) Alternative 3 keeps the EMF offline Rittenhouse Stormwater Detention Basin in place. Included in its Alternative 3 design are amenity features such as hiking trails, park and ride facilities and support buildings. Because of these changes and the modifications to the EMF channel that are incorporated by Alternative 3, the specifications of the Rittenhouse Stormwater Detention Basin are different than either Alternative 1 or 2. See Table 9 for a complete summary of the Rittenhouse Stormwater Detention Basin's specifications.

Table 9: Alternative 3 Specifications for Rittenhouse SDB

Alternative 3 Rittenhouse Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	14.85	
Top of basin from basin invert (ft) =	19.8	
Total area of detention basin (Acres) =	179	
Total Capacity (Ac-ft) =	1344	
Head to pump (ft) =	8.5	
Volume to Pump (Ac-ft) =	735	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	1261	591
Total div from EMF	994	535
Total div from Ritt	267	56
Surplus Capacity (Ac-ft) =	83	753
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	6.35	
Weir Length (ft) =	1500	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
From Rittenhouse Channel		
Weir Crest Height (ft) =	2	
Weir Length (ft) =	150	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	1.17	
Drainage of Detention Basin		
Head to Pump (ft) =	8.5	
Volume to Pump (Ac-ft) =	735	
Pump Style =	Screw	
Pump Quantity =	5	
Drainage Culvert =	7-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	526	

The Rittenhouse Stormwater Detention Basin Alternative 3 Design has an area of 179 acres and a total depth of 19.80 feet. This includes 14.85 feet from the invert of the basin to the minimum weir crest height and 4.95 feet of freeboard. A 1500-foot long weir with a crest height of 6.35 feet above the channel invert diverts flow from the EMF and a 150-foot long weir with a crest of 2-feet above the channel invert diverts flow from the Rittenhouse Channel.

To obtain the necessary volumes and flow rates, the invert of the Rittenhouse Stormwater Detention Basin was set 8.5-feet below the invert of the outlet structures draining the basin. This depth represents the volume of 735 acre-feet that must be pumped from the detention basin into the EMF.

A 7-cell 6'x3' RCBC is proposed as an outlet structure to the Rittenhouse Stormwater Detention Basin to drain it back into the EMF in the requisite 36-hours. The culverts will have flap gates on their downstream ends to prevent water from flowing from the EMF back into the basin. In addition, a 5-pump station will be required to drain the 735 acre-feet below the outlet

See Figure 11 for the Rittenhouse Stormwater Detention Basin plan, weir specifications and detention basin cross sections.

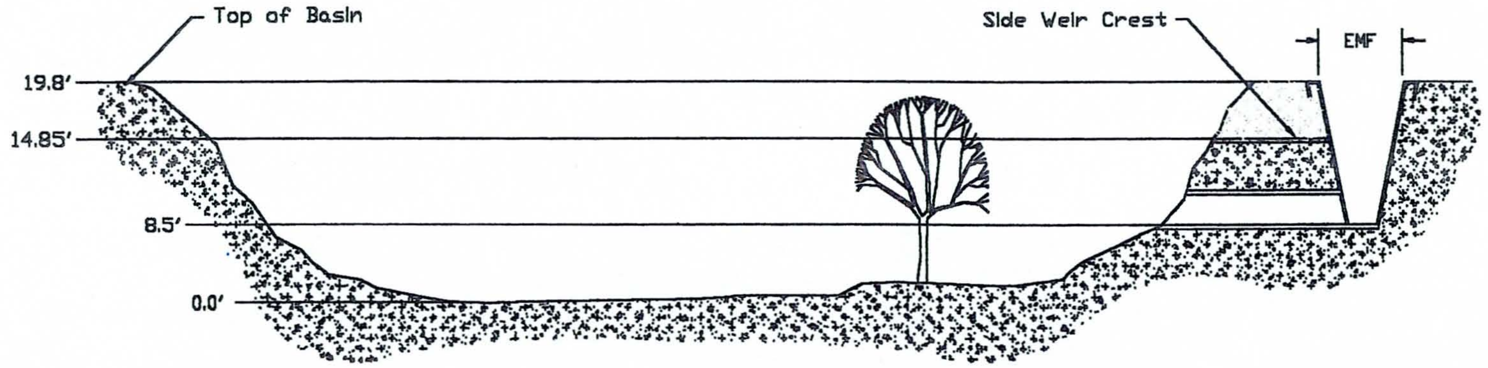
4) Alternative 3 keeps the EMF offline Chandler Heights Stormwater Detention Basin in place. Included in its Alternative 3 design are amenity features such as hiking trails, park and ride facilities and support buildings. Because of these changes and the modifications to the EMF channel that are incorporated in Alternative 3, the specifications of the Chandler Heights Stormwater Detention Basin are different that either Alternative 1 or 2. See Table 10 for a complete summary of the specifications of the Alternative 3 proposed Chandler Heights Stormwater Detention Basin.

The Alternative 3, Chandler Heights Stormwater Detention Basin has an area of 300 acres and a total depth of 25.90 feet. This includes 17.00 feet from the invert of the detention basin to the minimum weir crest height and 8.90 feet of freeboard. The total storage volume of the detention basin, from the invert to the minimum weir crest, is 1807 acre-feet. A 1000-foot long weir with a 5-foot crest height above the channel invert diverts flow from the EMF. A 500-foot long weir with a 5.75 foot crest height above the invert of the channel diverts flow from Queen Creek.

A total of 1724 acre-feet is diverted to the Chandler Heights Stormwater Detention Basin in the Alternative 3, 2002 HEC-1 model. This includes 723 acre-feet from the EMF and 1001 from Queen Creek. A total of 1474 acre-feet is diverted to the Chandler Heights Stormwater Detention Basin in the Alternative 3 build-out HEC-1 model, including 479 acre feet from the EMF and 995 acre-feet from Queen Creek.

Table 10: Alternative 3 Specifications for Chandler Heights SDB

Alternative 3 Chandler Heights Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	17	
Top of basin from basin invert (ft) =	25.9	
Total area of detention basin (Acres) =	300	
Total Capacity (Ac-ft) =	1807	
Head to pump (ft) =	12	
Volume to Pump (Ac-ft) =	767	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	1724	1474
Total div from EMF	723	479
Total div from Queen Creek	1001	995
Surplus Capacity (Ac-ft) =	83	333
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	5	
Weir Length (ft) =	1000	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
From Queen Creek		
Weir Crest Height (ft) =	5.75	
Weir Length (ft) =	500	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
Drainage of Detention Basin		
Head to Pump (ft) =	12	
Volume to Pump (Ac-ft) =	767	
Pump Style =	Screw	
Pump Quantity =	4	
Drainage Culvert =	11-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	957	



Rittenhouse Detention Basin Relative to EMF
(NTS)

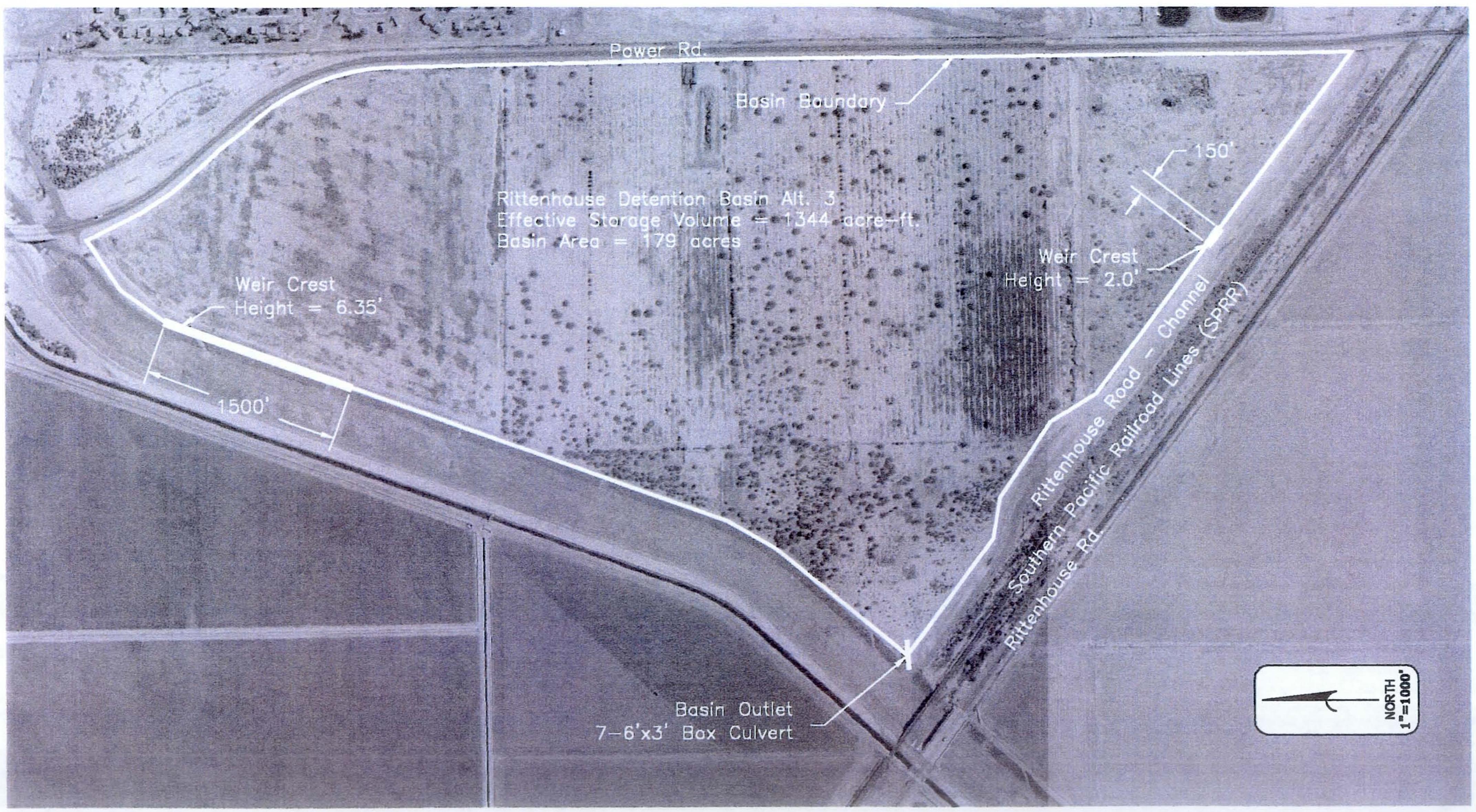


Figure 11
Alternative 3
Rittenhouse
Stormwater
Detention
Basin
Specifications

To obtain the necessary volumes and flow rates, the invert of the Chandler Heights Stormwater Detention Basin was set 12-feet below the invert of the outlet culverts that drain the basin. This will require the pumping of 767 acre-feet of volume from the basin back into the EMF. However, no diversion structures are necessary in either the EMF or Queen Creek in order to obtain the necessary flow depth for the diversion rates.

A 11-cell 6'x3' RCBC is proposed to drain the detention basin in the required 36-hours. A 4-pump station will be required to drain the 767 acre-feet below the invert of the outlet culverts. See Figure 12 for the stormwater detention basin plan, weir specifications and detention basin cross sections.

5) Channel modifications to the EMF have been made to the HEC-RAS model. These include a low flow channel at the following locations: Brown Road to Broadway Road, Guadalupe Road to Ray Road, and Power Road to Hunt Highway. In an approach similar to that taken in Alternative 2, Manning's coefficients in the low-flow channel were everywhere increased to 0.03. Moreover, Manning's coefficients in the EMF adjacent the low-flow sections were increased to either 0.035 or 0.04. Figure 10 shows all sections with increase vegetation, and the Appendix gives roughness coefficient details.

4.2.3.1 Mitigation Effectiveness

The mitigation within Alternative 3 is effective in attenuating the runoff reaching the EMF so that the peak 100-year 2002 discharge is contained within the EMF and the peak Build-out conditions, 100-year peak flow is contained with adequate freeboard.

4.2.3.2 Cost Estimate

Table 11 presents the projected cost estimate for the mitigation associated with Alternative 3.

4.2.3.3 Multi-Use Potential

Alternative 3 is the multi-modal, environmental enhancement alternative. It incorporates park and ride facilities and hiking trails with vegetative enhancement into the Rittenhouse and Chandler Heights Detention Basin and a low flow channel within the EMF along identical stretches as in Alternative 2. There is significantly more vegetative enhancement in the EMF incorporated into Alternative 3 above what is incorporated into Alternative 2 (Figure 10).

4.2.3.4 Recommendations for Further Consideration

Alternative 3 is a viable alternative that does not require substantial changes to the EMF channel. Because it proposes such a high degree of revegetation within the EMF, the detention basins are required to be significantly larger than in Alternative 2. Consequently the costs for Alternative 3 are significantly higher than those for Alternative 2. It is neither the engineering nor multi-modal recommended alternative.

Table 11: Alternative 3 Construction and Engineering Costs

Item	Construction	Land Acquisition	Design & Contingency	Total
Ray Detention Basin	$20,652,000	$5,130,000	$4,750,000	$30,532,000
Rittenhouse Detention Basin	$30,365,000	$5,370,000	$6,983,950	$42,718,950
Chandler Heights Detention Basin	$58,056,000	$1,950,000	$13,352,880	$73,358,880
Low-Flow Channel	$419,271	$33,542	$62,891	$515,704
Total	$109,492,271	$12,483,542	$25,132,661	$147,125,534

4.2.4 *Alternative 4*

The basic premise of Alternative 4 is to eliminate the Ray Stormwater Detention Basin from the Williams Airport area. Because the Ray Basin is effective in decreasing the peak flows from the Powerline Floodway and, as in Alternatives 1-3, the San Tan Freeway channel, its elimination makes it difficult to contain the 100-year flood within the EMF in several locations. Unaltered, the 100-year peak will break out of the EMF in the downstream portion of Reach 5 and several portions of Reach 4. The stretch of the EMF through the Williams Golf Course in Reach 4 is especially overburdened without the reduction in peak flow accomplished by the Ray Stormwater Detention Basin. In order to eliminate the Ray Basin, contain the existing conditions peak flow within the EMF, and contain the future Build-out conditions in the EMF with the required freeboard (i.e. 0.2 x the energy head in subcritical conditions), the channel of the EMF has to be modified beyond the specifications of the study criteria. Namely, the stretch of the EMF through the Williams Airport has to be modified.

The following changes were made to the 2002 and Build-out hydrologic models and the HEC-RAS model of the EMF:

1) Knox and Guadalupe Basins were removed from the routing model.

2) Ray Basin was removed completely from the SE watershed model.

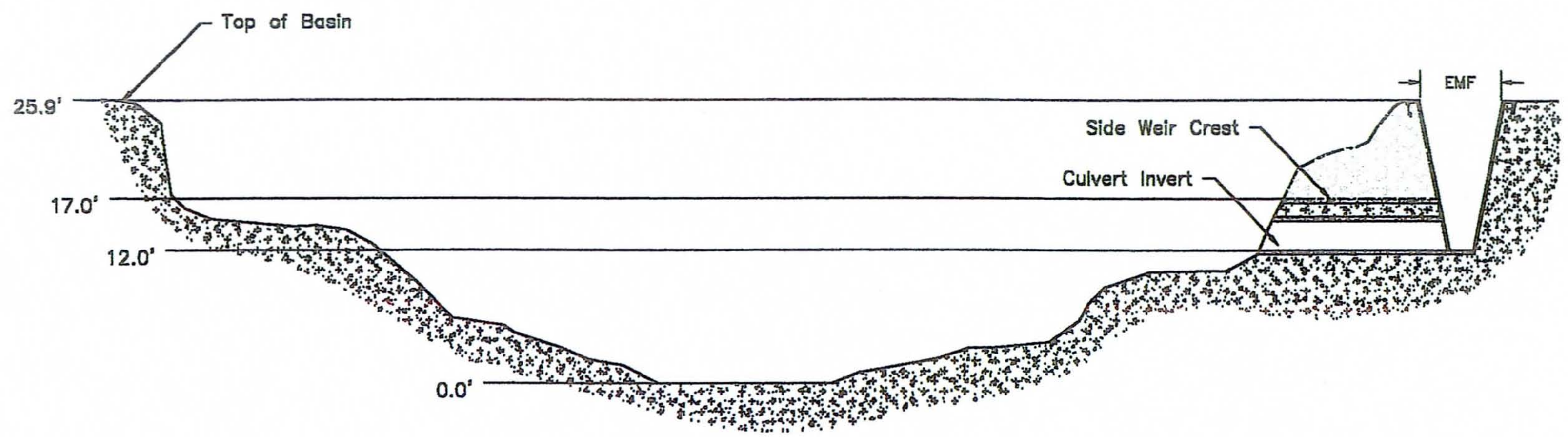
3) Volume was added to the Rittenhouse Stormwater Detention Basin in Alternative 4 to increase its effectiveness in decreasing the design peak flow. The area of the basin was set to 179 acres with a total depth of 16.80 feet above it invert. This includes 9.00 feet from the invert to the minimum weir crest and 7.8 feet of freeboard. The weir within the Rittenhouse Channel was set to a height of 2.5 above the channel invert and has a length of 500 feet. Within the EMF, the diversion weir has a crest 3.50 feet above the channel invert and a length of 2000 feet. See Table 12 for the specifications of the Alternative 4 design of the Rittenhouse Stormwater Detention Basin.

A total of 1538 acre-feet is diverted into the Rittenhouse Stormwater Detention Basin in the 2002 model, including 1316 acre-feet from the EMF and 222 acre-feet from the Rittenhouse Channel. In the Build-out model, a total of 395 acre-feet is diverted to the Rittenhouse Stormwater Detention Basin, including 357 acre-feet from the EMF and 38 acre-feet from the Rittenhouse Channel. The Build-out conditions will include 1195 acre-feet of surplus storage. The basin invert is 5.50 feet below the invert of the outlet culvert. This will require the pumping of 952 acre-feet of water from the basin to the EMF.

A 7-cell 6'x3' RCBC is proposed as the outlet culvert to the stormwater detention basin to drain the 100-year 2002 volume in the required 36 hour period. In addition, a 5-cell screw pump structure is necessary to drain the water below the culvert in the necessary time-frame. See Figure 13 for the Alternative 3 Rittenhouse Stormwater Detention Basin Specifications.

Table 12: Alternative 4 Specifications for Rittenhouse SDB

Alternative 4 Rittenhouse Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	9	
Top of basin from basin invert (ft) =	16.8	
Total area of detention basin(Acres) =	179	
Total Capacity (Ac-ft) =	1590	
Head to pump (ft) =	5.5	
Volume to Pump (Ac-ft) =	952	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	1538	395
Total div from EMF	1316	357
Total div from Ritt	222	38
Surplus Capacity (Ac-ft) =	52	1195
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	3.5	
Weir Length (ft) =	2000	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
From Rittenhouse Channel		
Weir Crest Height (ft) =	2.5	
Weir Length (ft) =	500	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
Drainage of Detention Basin		
Head to Pump (ft) =	5.5	
Volume to Pump (Ac-ft) =	952	
Pump Style =	Screw	
Pump Quantity =	5	
Drainage Culvert =	7-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	586	



Chandler Heights Detention Basin Relative to EMF
(NTS)

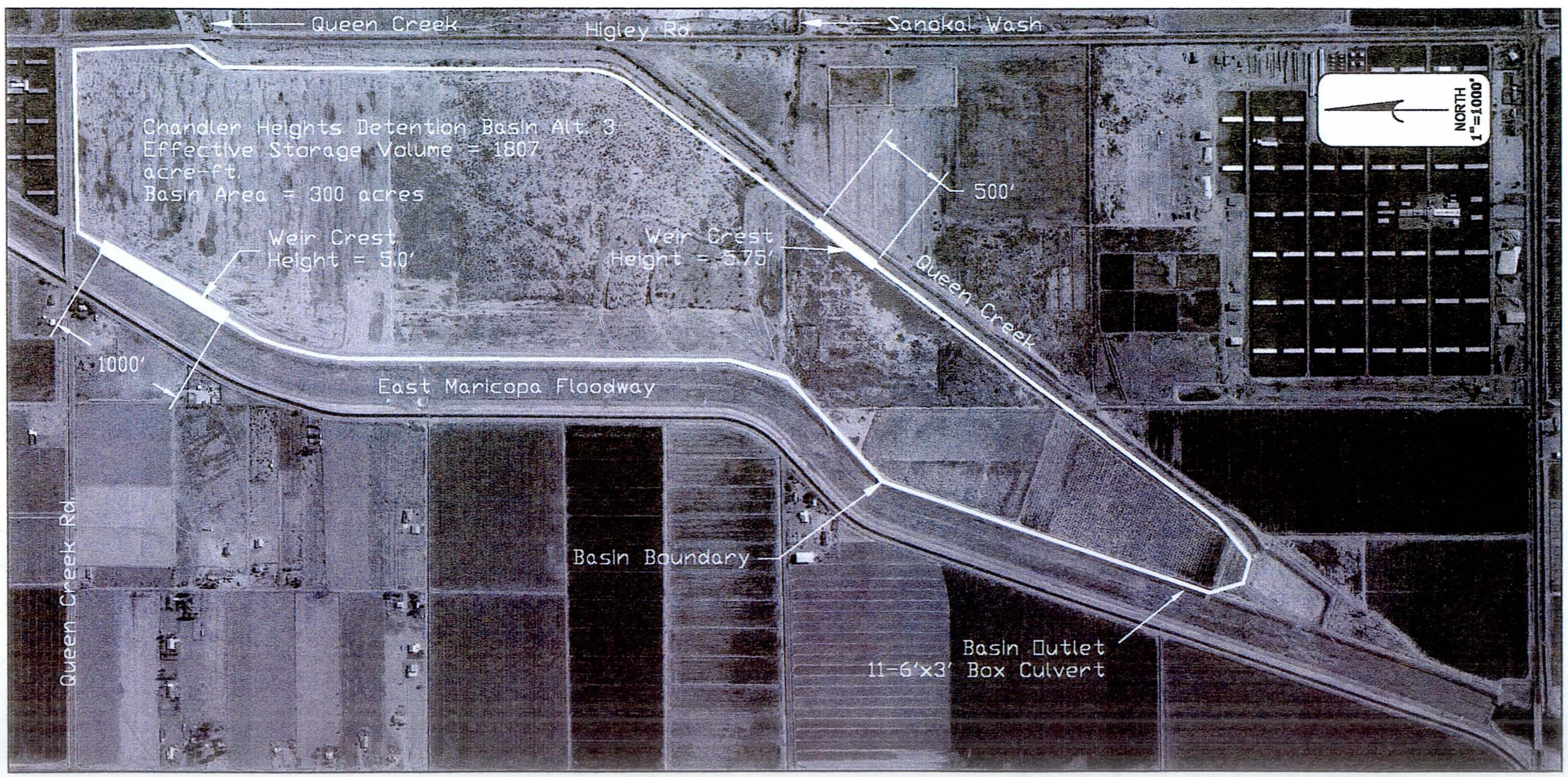
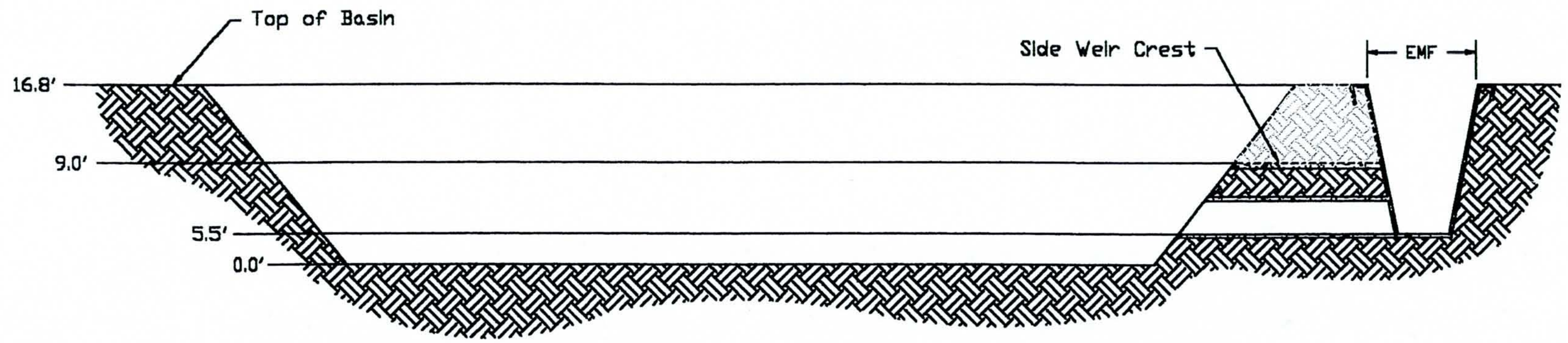


Figure 12
Alternative 3
Chandler
Heights
Stormwater
Detention
Basin
Specifications



Rittenhouse Detention Basin Relative to EMF
(NTS)

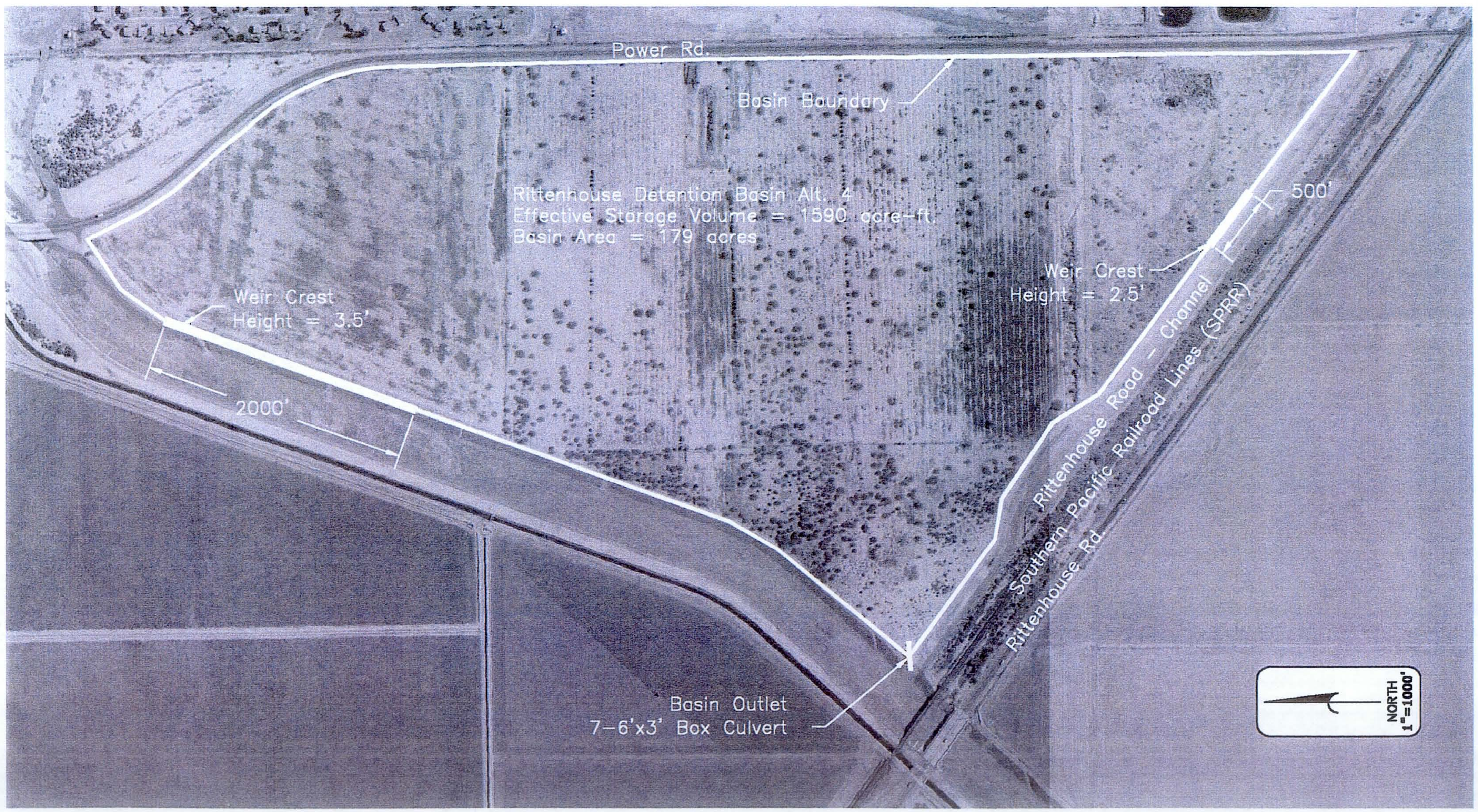


Figure 13
Alternative 4
Rittenhouse
Stormwater
Detention
Basin
Specifications

Table 13: Alternative 4 Specifications for Chandler Heights SDB

Alternative 4 Chandler Heights Stormwater Detention Basin		
Minimum weir crest from basin invert (ft) =	9	
Top of basin from basin invert (ft) =	18.4	
Total area of detention basin (Acres) =	300	
Total Capacity (Ac-ft) =	2616	
Head to pump (ft) =	4.5	
Volume to Pump (Ac-ft) =	1269	
	2002	Buildout
Total Flow Diverted (Ac-ft) =	2510	2084
Total div from EMF	868	448
Total div from Queen Creek	1642	1636
Surplus Capacity (Ac-ft) =	106	532
Side Weirs		
From EMF Channel		
Weir Crest Height (ft) =	4.5	
Weir Length (ft) =	1000	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
From Queen Creek		
Weir Crest Height (ft) =	5.5	
Weir Length (ft) =	1000	
Weir Width (ft) =	10	
Weir Coeff. =	2.63	
Diversion Structure Height (ft)	None	
Drainage of Detention Basin		
Head to Pump (ft) =	4.5	
Volume to Pump (Ac-ft) =	1269	
Pump Style =	Screw	
Pump Quantity =	7	
Drainage Culvert =	13-6'x3' RCBC	
Volume to Drain by Culvert (Ac-ft)	1241	

4) The Chandler Heights Stormwater Detention Basin in Alternative 4 has an area of 300 acre and a total depth of 18.40 feet. This includes 9.00 feet of depth from the invert of the basin to the minimum weir crest and 9.40 feet of freeboard. The weir within the EMF was set to a crest height of 4.5 above the channel invert with a length of 1000 feet. The weir within the Queen Creek channel was set to a crest height of 5.5 feet above the channel invert with a length of 1000 feet. See Table 13 for a full description of Alternative 4, Chandler Heights Stormwater Detention Basin.

The Chandler Heights Stormwater Detention Basin diverts a total of 2510 acre-feet in the Alternative 4, 2002 HEC-1 model, including 868 acre-feet from the EMF and 1642 acre-feet from the Queen Creek. In the Build-out model, the basin diverts a total of 2084 acre-feet, including 448 acre-feet from the EMF and 1636 acre-feet from Queen Creek. However, the basin does require a depth of 4.5-feet below the invert of the outlet culvert. This will require the pumping of 1269 acre-feet of water back into the EMF.

In Alternative 4, a 13-cell 6'x3' RCBC is proposed as the outlet structure to Chandler Heights Stormwater Detention Basin in order to drain it within 24-hours. In addition, a 7-pump screw pump structure will be required to pump the water below the invert of the outlet structure. See Figure 14 for the Alternative 3 Chandler Heights Stormwater Detention Basin Specifications.

5) The grade-control drop within Reach 3 just north of the Chandler Heights Bridge crossing (between cross sections 11.321 and 11.308) was removed completely. The grade control drop within Reach 4 between the Higley Road and Rittenhouse Road crossings (between cross sections 14.754 and 14.738) was also removed, along with its associated width constriction. The drop was relocated upstream to Reach 5 just south of the Elliot Road Bridge (between cross sections 19.863 and 20.058). Between the new drop structure in Reach 5 and the removed drop structure in Reach 3 (cross section 19.863 in Reach 5 and cross section 11.308 in Reach 3), the channel invert slope was set to a constant 0.00057 ft/ft. To make this change, each cross section was altered by dropping its bottom width to the prescribed depth keeping the existing slide slopes and Manning's coefficients constant (see Figure 15 for explanation). These modifications were made to the cross sections through the bridge structures.

6) The bottom width of each cross section through the William Golf Course constriction (from 17.071 to 18.193) in Reach 4 was increased by 30-feet, again keeping the existing side slopes constant (see Figure 16 for explanation). Again, these modifications were made to the cross sections through the bridge structures.

4.2.4.1 Mitigation Effectiveness

With the removal of the Ray Stormwater Detention Basin, the burden on the Rittenhouse and Chandler Heights Stormwater Detention Basins is increased above the base model. In the Alternative 4 2002 model, the total diversion to the Rittenhouse Stormwater Detention Basin is 1538 acre-feet. This is an increase of 512 acre-feet of total required storage volume presented in the base 2002 model. In the Alternative 4 Build-out model, the required volume of the Rittenhouse is 395 acre-feet. This is an increase of 352 acre-feet from the total required storage volume in the base Build-out model.

The Chandler Heights Stormwater Detention Basin is required to detain a total of 2510 acre feet in the Alternative 4 model. This is an increase of 172 acre-feet from the base 2002 model. In the Alternative 4 Build-out model, the Chandler Heights Stormwater Detention Basin is required to detain a total of 2084 acre-feet. This is actually a decrease of 44 acre-feet from the base Build-out model.

The channel alterations allow for full conveyance of the design event in the 2002 conditions and full conveyance with freeboard in Future/Buildout conditions. Obviously, this alternative violates the criteria of leaving the EMF channel unaltered through the Williams Golf Course. The channel is changed substantially, both horizontally and vertically. The proposed channel modifications would require reconstruction and/or alteration of six bridge structures in Reaches 3, 4 and 5. The bridge reconstruction was not been included in the cost estimate for Alternative 4.

4.2.4.2 Cost Estimate

Table 14 presents the projected cost estimate for the mitigation associated with Alternative 4.

4.2.4.3 Multi-Use Potential

There is no multi-use potential associated with Alternative 4.

4.2.4.4 Recommendations for Further Consideration

Because Alternative 4 violates the District's criteria of no channel modifications in the Williams Golf Course, it has limited applicability in this study. In addition, the Alternative 4 would require reconstruction of several bridge crossings, which would be cost prohibitive. This alternative is generally a comparison alternative to demonstrate the effectiveness of the Ray Stormwater Detention Basin.

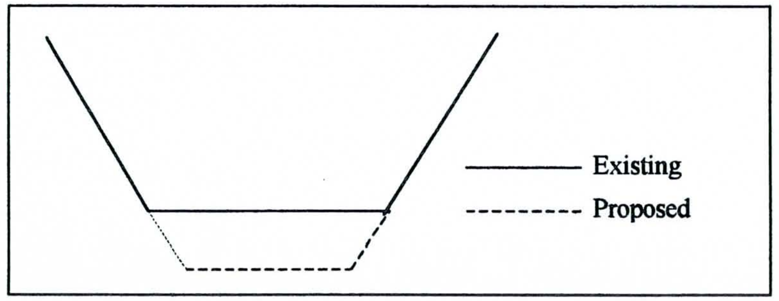


Figure 15: Typical cross section change from cross section 11.321 in Reach 3 to 19.863 in Reach 5.

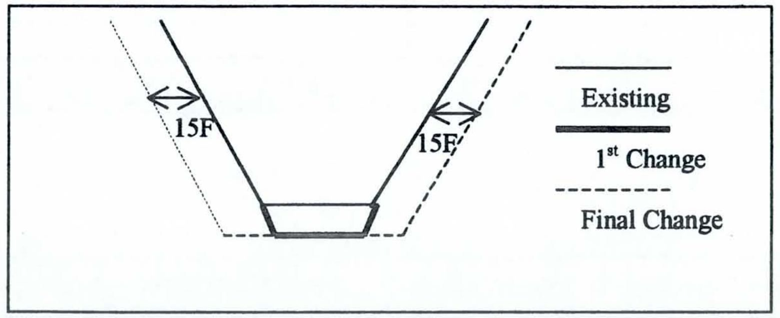
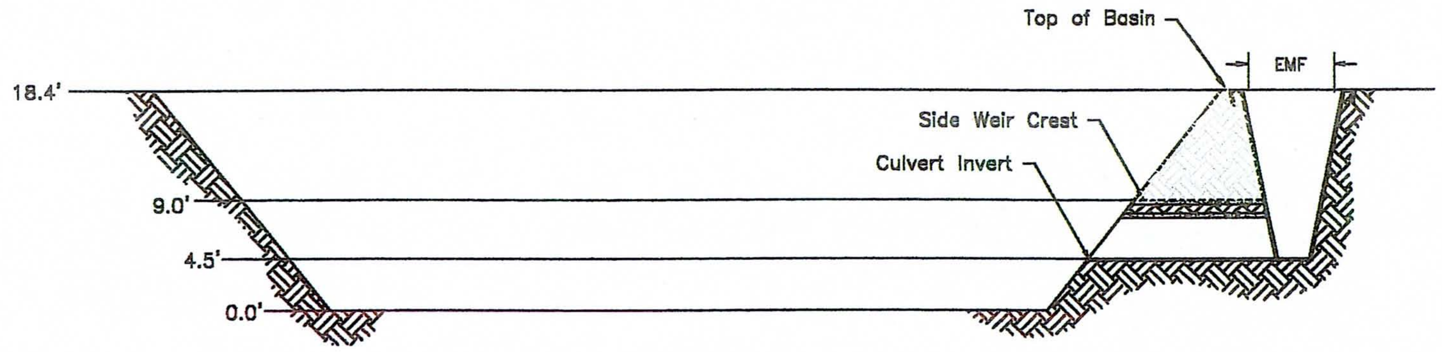


Figure 16: Channel Bottom Width Extension in Alternative 4 from cross section 17.071 to 18.193.



Chandler Heights Detention Basin Relative to EMF
(NTS)

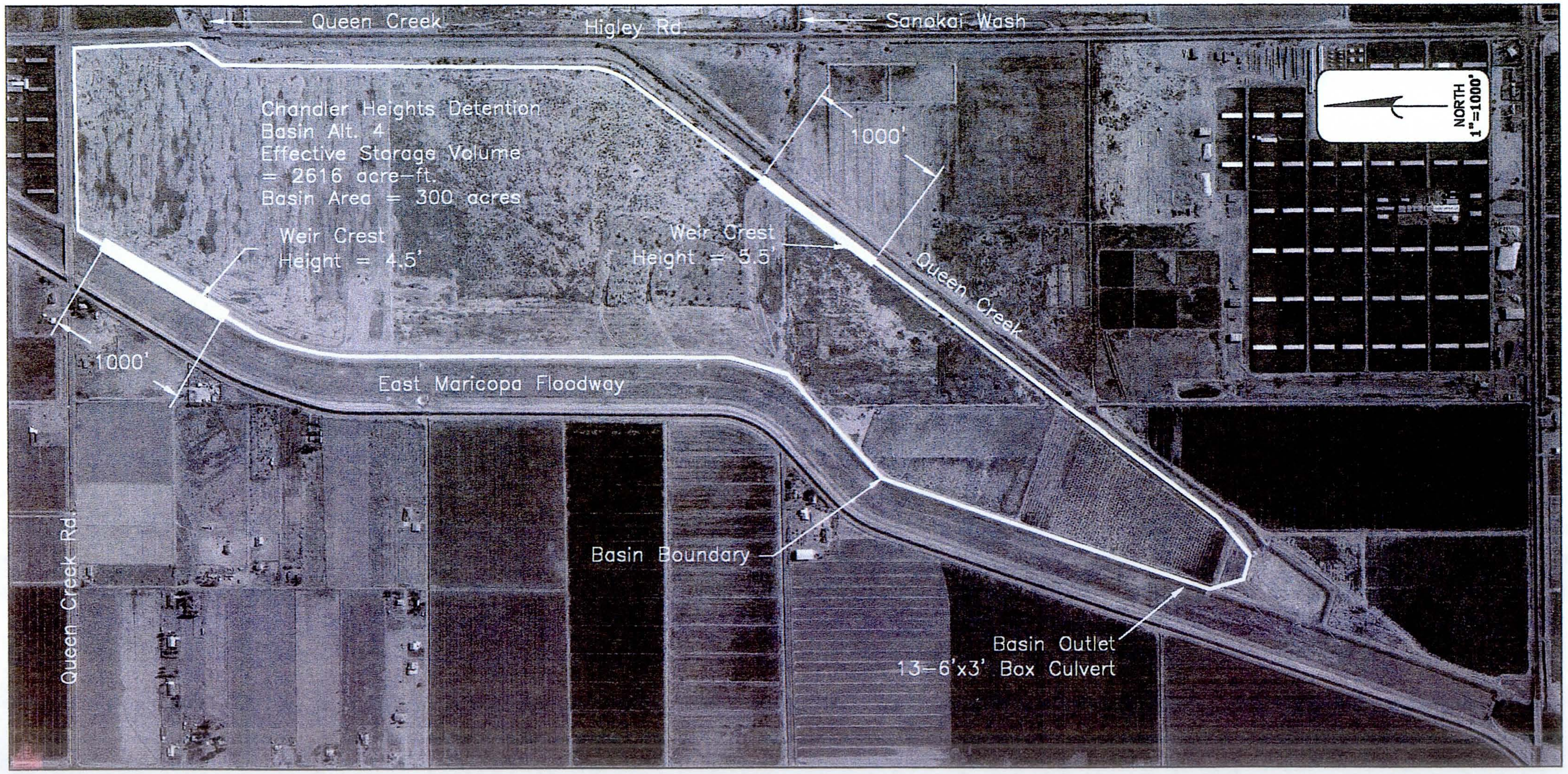


Figure 14
Alternative 4
Chandler
Heights
Stormwater
Detention
Basin
Specifications

5.0 FURTHER CONSIDERATION

There are several conservative aspects to the design affecting each alternative. First, because the build-out/freeboard criterion is more critical than the 2002 conveyance in most cases, the crest heights and lengths of the diversion weirs are usually based on the Built-out event. However, the resulting volume of diversion, which mandates the size of each stormwater detention basin, is based on the 2002 event (the volume of water that is diverted from the channels with the designed diversion weir crest height and length). Therefore, each basin is forced to store a larger volume of water to satisfy both criteria. In addition, the required detention volumes are conservative because the entire top of the flow hydrograph within the channel is lopped off at each diversion, diverting all flow above a certain rate, with no storage considered above the invert of the weir crest. The volume of detention storage could be minimized if water was allowed to be stored above the weir crest height to an elevation that, when reached, would allow the remainder of the flow in the channel to pass by. This would also allow water to flow from the stormwater detention basin back into the channel over the weir after the peak has passed, lessening the burden on the outlet culverts. These conservative design elements should be carefully considered in the final design of the basins. Their alteration could substantially reduce the cost of each stormwater detention basin.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The challenge facing planners working with the East Maricopa Floodway is to devise flood control solutions that effectively mitigate 100-year flood waters safely out of Maricopa County without adversely impacting residents of Pinal County and the Gila River Indian Reservation. Moreover, strong community sentiment favors utilizing the space within the EMF channel and the detention basins for multiple purposes including recreation. Unfortunately, implementation of these multiple uses can, at times, require larger or more complicated flood control structures.

Table 14: Alternative 4 Construction and Engineering Costs (without bridge modification costs)

Item	Construction	Land Acquisition	Design & Contingency	Total
Rittenhouse Detention Basin	$27,160,000	$5,370,000	$6,247,000	$38,777,000
Chandler Heights Detention Basin	$42,929,000	$1,950,000	$9,874,000	$54,753,000
Channel Alterations	$5,151,310	NA	$1,184,801	$6,336,111
Total	$75,240,310	$7,320,000	$17,305,801	$99,866,111

Designers ultimately have two tools to mitigate flooding throughout the system: modifications to the EMF channel and the use of off-line storm-water detention basins to capture and retain peak discharge. However, the EMF system is dynamic and highly non-linear. Seemingly minor changes to the channel or detention basin in one section of the system can have major effects on another part of the system. In contrast, major changes in some areas may have little impact in the overall system. In meeting the stated criteria for the study during each alternative, the designers focussed on three critical river sections of the EMF: at Hunt Highway, just below Riggs Road(within Reach 3), and within the Williams Golf Course. The issues within the golf course were best addressed by making changes to the Ray Detention Basin. Moreover, the relatively large size of Ray Basin in Alternatives 1-3 points directly to the need to bring the water surface in the golf course down to an acceptable level. Downstream, near Hunt Highway, the water surface was most effectively lowered by changing both Rittenhouse and Chandler Height Detention Basins. While the EMF was clean and free of vegetation in Alternative 1, both of the lower basins were relatively modest in size. As vegetation was added to the EMF under the objective of creating a multi-use EMF corridor in Alternatives 2-3, the two lower basins needed to be larger. Table 15 compares the costs, features, and detention basin size for each of the four alternatives.

Of the specific recommendations from the study, the first is to include the Ray Detention Basin. One of the best ways to meet project goals for any set of alternatives is to include the Ray Basin. It can be designed to remain dry in all but the largest floods and it offers the designer great leverage in balancing the entire system. As shown in Alternative 4, eliminating the Ray Basin shifts the burden of flood mitigation to downstream basins and modifications of the EMF itself. The second recommendation is the addition of a low-flow channel to the EMF. Such a channel would confine frequent, small discharge events to the low-flow channel, thus reducing maintenance within the EMF. It is recommended that the EMF Channel not be vegetated; however, if the District does chose to add vegetation to the EMF, there are some sections of the channel that are more hydraulically sensitive. These sensitive sections are located between Power Road and the energy-drop structure above Germann Road as well as the section below Chandler Heights Road. The difference between not adding and adding vegetation to these sensitive sections is represented in the study by Alternative 2 and Alternative 3 (see Figure 10).

Also recommended is for the District to develop careful and accurate techniques to evaluate the hydraulic features in the design. This study explicitly chose to take many conservative approaches to designing the detention basins and side weirs. When in doubt, the more conservative option was always chosen. There is opportunity for careful and detailed engineering design that could allow more accurate hydraulic analysis. With the careful analysis, some, if not all, the basins could be reduced in size and complexity. The extra engineering could develop a more "optimum" design and save significant costs in the final project. In addition, balancing modifications in the channel with modification in the basins, while incorporating less conservative assumptions, could yield a less costly solution.

The final recommendation is for the District to size the retention capacity of each basin for the future build-out conditions rather than the 2002 conditions as done in this study. Such an approach would meet future flood mitigation needs at a significantly lower cost. To properly cost this approach, a designer would need to completely redesign optimize each basin with changes to basin size, drainage culverts, and pumping stations. Such an exercise is beyond the scope of the current study. Nonetheless, simple calculations with smaller basin excavation costs were completed to evaluate potential savings. When the excavation casts for the basins were reduced to meet buildout conditions, the alternative costs were calculated to by no more than $46 million, $55 million, $63 million, and $58 million for Alternatives 1 through 4 respectively. Further savings would come from design optimization.

Table 15: Comparative costs and scope of each alternative. Note—Alternative 4 costs do not include bridge modifications.

			Ray DB		Rittenhouse DB		Chandler Heights DB	
	Desc.	Total project cost	Area (Ac)	Volume (Ac-ft)	Area (Ac)	Volume (Ac-ft)	Area (Ac)	Volume (Ac-ft)
Alt 1	Strictly Engr	$89.7 M	171	1537	111	690	300	1415
Alt 2	Low-flow; some veg.	$132.8 M	171	1537	179	982	300	1591
Alt 3	Low-flow; more veg.	$147.1 M	171	1537	179	1344	300	1807
Alt 4	No Ray DB; no veg.	$99.9 M	-	-	179	1590	300	2616

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APPENDIX

Detailed Cost Estimates

Alternative 1

Ray Detention Basin

Area (acre)	171	1st Weir Length	750	1st Weir Crest	9.25
Depth (ft)	15.7	2nd Weir Length	200	2nd Weir Crest	11.25
Volume (yd^3)	4,331,316	Outlet Pipes	9-6'x3' RCBC		

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	171	$2,000.00	$342,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	4,331,316	$3.75	$16,242,435
4	Seeding	Acres	171	$2,500.00	$427,500
5	Excavation for Side Weir (Powerline-1)	CY	13,083	$4.50	$58,875
6	Excavation for Side Weir (San Tan Freeway Channel-2)	CY	3,489	$4.50	$15,700
7	Concrete Side Weir (Powerline-1)	CY	2,569	$425.00	$1,092,014
8	Concrete Side Weir (San Tan-2)	CY	833	$425.00	$354,167
9	Grouted Riprap for Erosion Protection (Powerline-1)	CY	13,875	$82.50	$1,144,688
10	Grouted Riprap for Erosion Protection (San Tan-2)	CY	4,500	$82.50	$371,250
11	Outlet Pipes*	L.F.	120	$1,570.00	$188,400
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$12,028.00	$12,028
13	Trash Rack/Access Barrier	Each	9	$1,200.00	$10,800
14	Major Utility Relocations	L. Sum	1	$45,000.00	$45,000
15	Pump Station	L. Sum	2	$123,600.00	$247,200
	Subtotal Construction Costs				$20,652,056
	Land Acquisition	Acres	171	$30,000.00	$5,130,000
	Engineering (8%)	L. Sum	1		$1,652,164
	Contingency (15%)	L. Sum	1		$3,097,808
	Total				$30,532,029

Alternative 1

Rittenhouse Detention Basin

Area (acre)	111	1st Weir Length	1500	1st Weir Crest	6.5
Depth (ft)	11.3	2nd Weir Length	150	2nd Weir Crest	7
Volume (yd^3)	2023604	Outlet Pipes	5-6'x3' RCBC		

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	111	$2,000.00	$222,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	2,023,604	$3.75	$7,588,515
4	Seeding	Acres	111	$2,500.00	$277,500
5	Excavation for Side Weir (EMF-1)	CY	18,833	$4.50	$84,750
6	Excavation for Side Weir (Rittenhouse Channel-2)	CY	1,883	$4.50	$8,475
7	Concrete Side Weir (EMF-1)	CY	3,611	$425.00	$1,534,722
8	Concrete Side Weir (Rittenhouse-2)	CY	389	$425.00	$165,278
9	Grouted Riprap for Erosion Protection (Powerline-1)	CY	19,500	$82.50	$1,608,750
10	Grouted Riprap for Erosion Protection (San Tan-2)	CY	2,100	$82.50	$173,250
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	5	$1,200.00	$6,000
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
					$11,879,289
	Subtotal Construction Costs				$11,879,289
	Land Acquisition	Acres	111	$30,000.00	$3,330,000
	Engineering (8%)	L. Sum	1		$950,343
	Contingency (15%)	L. Sum	1		$1,781,893
	Total				$17,941,525

Alternative 1

Area (acre)	300	1st Weir Length	1000	1st Weir Crest	5
Depth (ft)	13.9	2nd Weir Length	500	2nd Weir Crest	20.95
Volume (yd^3)	6727600	Outlet Pipes	13-6'x3' RCBC		

Chandler Heights Detention Basin

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	300	$2,000.00	$600,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	6,727,600	$3.75	$25,228,500
4	Seeding	Acres	300	$2,500.00	$750,000
5	Excavation for Side Weir (EMF-1)	CY	15,444	$4.50	$69,500
6	Excavation for Side Weir (Queen Creek-2)	CY	7,722	$4.50	$34,750
7	Concrete Side Weir (EMF-1)	CY	1,852	$425.00	$787,037
8	Concrete Side Weir (Queen Creek-2)	CY	3,880	$425.00	$1,648,843
9	Grouted Riprap for Erosion Protection (EMF-1)	CY	10,000	$82.50	$825,000
10	Grouted Riprap for Erosion Protection (Queen Creek-2)	CY	20,950	$82.50	$1,728,375
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	13	$1,200.00	$15,600
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
	Subtotal Construction Costs				$31,897,654
	Land Acquisition	Acres	65	$30,000.00	$1,950,000
	Engineering (8%)	L. Sum	1		$2,551,812
	Contingency (15%)	L. Sum	1		$4,784,648
	Total				$41,184,114
		0.08			

Total DB Cost for Alt 1 = $89,657,668

Alternative 2

Ray Detention Basin

Area (acre)	171	1st Weir Length		750	1st Weir Crest	9.25
Depth (ft)	15.7	2nd Weir Length		200	2nd Weir Crest	11.3
Volume (yd^3)	4,331,316	Outlet Pipes		9-6'x3' RCBC		

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	171	$2,000.00	$342,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	4,331,316	$3.75	$16,242,435
4	Seeding	Acres	171	$2,500.00	$427,500
5	Excavation for Side Weir (Powerline-1)	CY	13,083	$4.50	$58,875
6	Excavation for Side Weir (San Tan Freeway Channel-2)	CY	3,489	$4.50	$15,700
7	Concrete Side Weir (Powerline-1)	CY	2,569	$425.00	$1,092,014
8	Concrete Side Weir (San Tan-2)	CY	833	$425.00	$354,167
9	Grouted Riprap for Erosion Protection (Powerline-1)	CY	13,875	$82.50	$1,144,688
10	Grouted Riprap for Erosion Protection (San Tan-2)	CY	4,500	$82.50	$371,250
11	Outlet Pipes*	L.F.	120	$1,570.00	$188,400
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$12,028.00	$12,028
13	Trash Rack/Access Barrier	Each	9	$1,200.00	$10,800
14	Major Utility Relocations	L. Sum	1	$45,000.00	$45,000
15	Pump Station	L. Sum	2	$123,600.00	$247,200
	Subtotal Construction Costs				$20,652,056
	Land Acquisition	Acres	171	$30,000.00	$5,130,000
	Engineering (8%)	L. Sum	1		$1,652,164
	Contingency (15%)	L. Sum	1		$3,097,808
	Total				$30,532,029

Alternative 2

Rittenhouse Detention Basin

Area (acre)	179	1st Weir Length	1435	1st Weir Crest	10.35
Depth (ft)	15.3	2nd Weir Length	150	2nd Weir Crest	9
Volume (yd^3)	4418436	Outlet Pipes	5-6'x3' RCBC		

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	179	$2,000.00	$358,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	4,418,436	$3.75	$16,569,135
4	Seeding	Acres	179	$2,500.00	$447,500
5	Excavation for Side Weir (EMF-1)	CY	24,395	$4.50	$109,778
6	Excavation for Side Weir (Rittenhouse Channel-2)	CY	2,550	$4.50	$11,475
7	Concrete Side Weir (EMF-1)	CY	5,501	$425.00	$2,337,854
8	Concrete Side Weir (Rittenhouse-2)	CY	500	$425.00	$212,500
9	Grouted Riprap for Erosion Protection (Powerline-1)	SY	29,705	$82.50	$2,450,621
10	Grouted Riprap for Erosion Protection (San Tan-2)	SY	2,700	$82.50	$222,750
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	5	$1,200.00	$6,000
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
15	Pump Station	L. Sum	2	$123,600.00	$247,200
	Subtotal Construction Costs				$23,182,862
	Land Acquisition	Acres	179	$30,000.00	$5,370,000
	Engineering (8%)	L. Sum	1		$1,854,629
	Contingency (15%)	L. Sum	1		$3,477,429.29
	Total				$33,884,920

Alternative 2

Area (acre)	300	1st Weir Length	1000	1st Weir Crest	15
Depth (ft)	23.9	2nd Weir Length	325	2nd Weir Crest	31
Volume (yd^3)	11567600	Outlet Pipes	11-6'x3' RCBC		

Chandler Heights Detention Basin

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	300	$2,000.00	$600,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	11,567,600	$3.75	$43,378,500
4	Seeding	Acres	300	$2,500.00	$750,000
5	Excavation for Side Weir (EMF-1)	CY	26,556	$4.50	$119,500
6	Excavation for Side Weir (Queen Creek-2)	CY	8,631	$4.50	$38,838
7	Concrete Side Weir (EMF-1)	CY	5,556	$425.00	$2,361,111
8	Concrete Side Weir (Queen Creek-2)	CY	3,731	$425.00	$1,585,880
9	Grouted Riprap for Erosion Protection (EMF-1)	SY	30,000	$82.50	$2,475,000
10	Grouted Riprap for Erosion Protection (Queen Creek-2)	SY	20,150	$82.50	$1,662,375
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	11	$1,200.00	$13,200
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
15	Pump Station	L. Sum	3	$123,600.00	$370,800
	Subtotal Construction Costs				$53,565,252
	Land Acquisition	Acres	65	$30,000.00	$1,950,000
	Engineering (8%)	L. Sum	1		$4,285,220
	Contingency (15%)	L. Sum	1		$8,034,788
	Total				$67,835,260

Total DB Cost for Alt 2 =	$132,252,209
Channel Modifications =	$515,704
	$132,767,913

Alternative 3

Ray Detention Basin

Area (acre)	171	1st Weir Length	750	1st Weir Crest	9.25
Depth (ft)	15.7	2nd Weir Length	200	2nd Weir Crest	11.3
Volume (yd^3)	4,331,316	Outlet Pipes	9-6'x3' RCBC		

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	171	$2,000.00	$342,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	4,331,316	$3.75	$16,242,435
4	Seeding	Acres	171	$2,500.00	$427,500
5	Excavation for Side Weir (Powerline-1)	CY	13,083	$4.50	$58,875
6	Excavation for Side Weir (San Tan Freeway Channel-2)	CY	3,489	$4.50	$15,700
7	Concrete Side Weir (Powerline-1)	CY	2,569	$425.00	$1,092,014
8	Concrete Side Weir (San Tan-2)	CY	833	$425.00	$354,167
9	Grouted Riprap for Erosion Protection (Powerline-1)	CY	13,875	$82.50	$1,144,688
10	Grouted Riprap for Erosion Protection (San Tan-2)	CY	4,500	$82.50	$371,250
11	Outlet Pipes*	L.F.	120	$1,570.00	$188,400
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$12,028.00	$12,028
13	Trash Rack/Access Barrier	Each	9	$1,200.00	$10,800
14	Major Utility Relocations	L. Sum	1	$45,000.00	$45,000
15	Pump Station	L. Sum	2	$123,600.00	$247,200
	Subtotal Construction Costs				$20,652,056
	Land Acquisition	Acres	171	$30,000.00	$5,130,000
	Engineering (8%)	L. Sum	1		$1,652,164
	Contingency (15%)	L. Sum	1		$3,097,808
	Total				$30,532,029

Alternative 3

Rittenhouse Detention Basin

Area (acre)	179	1st Weir Length	1500	1st Weir Crest	14.4
Depth (ft)	19.3	2nd Weir Length	150	2nd Weir Crest	15
Volume (yd^3)	5573582.667	Outlet Pipes	5-6'x3' RCBC		

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	179	$2,000.00	$358,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	5,573,583	$3.75	$20,900,935
4	Seeding	Acres	179	$2,500.00	$447,500
5	Excavation for Side Weir (EMF-1)	CY	32,167	$4.50	$144,750
6	Excavation for Side Weir (Rittenhouse Channel-2)	CY	3,217	$4.50	$14,475
7	Concrete Side Weir (EMF-1)	CY	7,972	$425.00	$3,388,194
8	Concrete Side Weir (Rittenhouse-2)	CY	833	$425.00	$354,167
9	Grouted Riprap for Erosion Protection (Powerline-1)	CY	43,050	$82.50	$3,551,625
10	Grouted Riprap for Erosion Protection (San Tan-2)	CY	4,500	$82.50	$371,250
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	5	$1,200.00	$6,000
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
15	Pump Station	L. Sum	5	$123,600.00	$618,000
	Subtotal Construction Costs				$30,364,945
	Land Acquisition	Acres	179	$30,000.00	$5,370,000
	Engineering (8%)	L. Sum	1		$2,429,196
	Contingency (15%)	L. Sum	1		$4,554,742
	Total				$42,718,882

Alternative 3

Area (acre)	300	1st Weir Length	1000	1st Weir Crest	17
Depth (ft)	25.9	2nd Weir Length	500	2nd Weir Crest	20.5
Volume (yd^3)	12535600	Outlet Pipes	11-6'x3' RCBC		

Chandler Heights Detention Basin

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	300	$2,000.00	$600,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	12,535,600	$3.75	$47,008,500
4	Seeding	Acres	300	$2,500.00	$750,000
5	Excavation for Side Weir (EMF-1)	CY	28,778	$4.50	$129,500
6	Excavation for Side Weir (Queen Creek-2)	CY	14,389	$4.50	$64,750
7	Concrete Side Weir (EMF-1)	CY	6,296	$425.00	$2,675,926
8	Concrete Side Weir (Queen Creek-2)	CY	3,796	$425.00	$1,613,426
9	Grouted Riprap for Erosion Protection (EMF-1)	CY	34,000	$82.50	$2,805,000
10	Grouted Riprap for Erosion Protection (Queen Creek-2)	CY	20,500	$82.50	$1,691,250
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	11	$1,200.00	$13,200
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
15	Pump Station	L. Sum	4	$123,600.00	$494,400
	Subtotal Construction Costs				$58,056,001
	Land Acquisition	Acres	65	$30,000.00	$1,950,000
	Engineering (8%)	L. Sum	1		$4,644,480
	Contingency (15%)	L. Sum	1		$8,708,400
	Total				$73,358,881

Total DB Cost for Alt 3 =	$146,609,792
Channel Modifications =	$515,704
	$147,125,496

Alternative 4

Rittenhouse Detention Basin

Area (acre)	179	1st Weir Length	2000	1st Weir Crest	9
Depth (ft)	16.8	2nd Weir Length	500	2nd Weir Crest	8.13
Volume (yd^3)	4851616	Outlet Pipes	7-6'x3' RCBC		

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	179	$2,000.00	$358,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	4,851,616	$3.75	$18,193,560
4	Seeding	Acres	179	$2,500.00	$447,500
5	Excavation for Side Weir (EMF-1)	CY	37,333	$4.50	$168,000
6	Excavation for Side Weir (Rittenhouse Channel-2)	CY	9,333	$4.50	$42,000
7	Concrete Side Weir (EMF-1)	CY	6,667	$425.00	$2,833,333
8	Concrete Side Weir (Rittenhouse-2)	CY	1,506	$425.00	$639,861
9	Grouted Riprap for Erosion Protection (Powerline-1)	CY	36,000	$82.50	$2,970,000
10	Grouted Riprap for Erosion Protection (San Tan-2)	CY	8,130	$82.50	$670,725
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	7	$1,200.00	$8,400
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
15	Pump Station	L. Sum	5	$123,600.00	$618,000
	Subtotal Construction Costs				$27,159,428
	Land Acquisition	Acres	179	$30,000.00	$5,370,000
	Engineering (8%)	L. Sum	1		$2,172,754
	Contingency (15%)	L. Sum	1		$4,073,914
	Total				$38,776,097

Alternative 4

Area (acre)	300	1st Weir Length	1000	1st Weir Crest	8.5
Depth (ft)	16.4	2nd Weir Length	1000	2nd Weir Crest	24.25
Volume (yd^3)	7937600	Outlet Pipes	12-6'x3' RCBC		

Chandler Heights Detention Basin

	Item	Unit	Quantity	Unit Price	Cost
1	Mobilization	L. Sum	1	$100,000.00	$100,000
2	Clearing and Grubbing	Acres	300	$2,000.00	$600,000
3	Basin Excavation (Includes removal and disposal of waste)	CY	7,937,600	$3.75	$29,766,000
4	Seeding	Acres	300	$2,500.00	$750,000
5	Excavation for Side Weir (EMF-1)	CY	18,222	$4.50	$82,000
6	Excavation for Side Weir (Queen Creek-2)	CY	18,222	$4.50	$82,000
7	Concrete Side Weir (EMF-1)	CY	3,148	$425.00	$1,337,963
8	Concrete Side Weir (Queen Creek-2)	CY	8,981	$425.00	$3,817,130
9	Grouted Riprap for Erosion Protection (EMF-1)	CY	17,000	$82.50	$1,402,500
10	Grouted Riprap for Erosion Protection (Queen Creek-2)	CY	48,500	$82.50	$4,001,250
11	Outlet Pipes*	L.F.	120	$864.00	$103,680
12	Inlet and Outlet Headwalls for Outlet Pipes	L. Sum	1	$6,369.00	$6,369
13	Trash Rack/Access Barrier	Each	12	$1,200.00	$14,400
14	Major Utility Relocations	L. Sum	0	$45,000.00	$0
15	Pump Station	L. Sum	7	$123,600.00	$865,200
	Subtotal Construction Costs				$42,928,492
	Land Acquisition	Acres	65	$30,000.00	$1,950,000
	Engineering (8%)	L. Sum	1		$3,434,279
	Contingency (15%)	L. Sum	1		$6,439,273.74
	Total				$54,752,045

Total DB Cost for Alt 4 =	$93,528,142
Channel Modifications =	$6,336,111
	$99,864,253

Alternative 4
Channel Excavation and Lining

Item	Unit	Quantity	Unit Price	Cost
Mobilization	L. Sum	1	$100,000.00	$100,000
Excavation	CY	1,347,016	$3.75	$5,051,310
Channel Relining	SY			
Subtotal Construction Costs				$5,151,310
Engineering (8%)	L. Sum	1		$412,105
Contingency (15%)	L. Sum	1		$772,697
Total				$6,336,111

Culvert Analysis

Barrel	Culvert Concrete CY	Steel LBS	Headwall Concrete CY	Steel LBS	Cost/LF	Headwall Costs
2	1.172	172.1	14.61	1173	$379.05	$4,239
3	1.687	237.6	17.02	1388	$540.55	$4,949
4	2.203	305.2	19.43	1603	$703.35	$5,659
5	2.718	369.1	21.84	1818	$864.05	$6,369
6	3.234	431.8	24.25	2033	$1,024.40	$7,079

Culvert Sizes
Barrel

Alt	Ray	Ritt	Chand
1	9	5	13
2	9	5	11
3	9	5	11
4	Na	7	12

Costs Per LF

Alt	Ray	Ritt	Chand
1	$1,567.40	$864.05	$2,267.50
2	$1,567.40	$864.05	$1,888.45
3	$1,567.40	$864.05	$1,888.45
4	Na	$1,243.10	$2,048.80

Costs Per HW

Alt	Ray	Ritt	Chand
1	$12,028	$6,369	$17,687
2	$12,028	$6,369	$13,448
3	$12,028	$6,369	$13,448
4	Na	$10,608	$14,158

Low-Flow Channel Excavation

Starting RS	Ending RS	Straight Length	Total Length	X-Sectional Area	Clearing Area	Total Volume
Mile	Mile	FT	FT	SF	SF	CY
27.244	24.936	12186.24	12264.72	6	367942	2,725.49
21.402	18.345	16140.96	16244.91	18	487347	10,829.94
16.94	16.205	3880.8	3905.79	18	117174	2,603.86
16.205	9.001	38037.12	38282.08	26.25	1148462	37,218.69
						53,377.98

Costs

1	Clearing and Grubbing	Acres	49	$2,000	$97,379
2	Channel Excavation (Includes removal and disposal of waste)	CY	53,378	$4	$200,167
3	Seeding	Acres	49	$2,500	$121,724
				Total	$419,271
				Design	$33,542
				Contingency	$62,891
				Total	$515,704

Pump Stations

Pumps Used: 96-inch diameter, 3-flight open screw pumps with a 48-in diameter by 3/8 in thick wall -lift of 14-ft at 30 percent inclination

Flow Rate = 29752 gal/minute

Pumping Information (Screw Pumps)									
Alt	Basin	Max Head (ft)	Volume (acre/feet)	Rate (cfs)	Rate (gal/m)	Pump Rate (gal/m)	# of Pumps (roundup)	Cost/pump	Total Cost
1	Ray	2	318	106.85	47,957	29752	2	$123,600	$247,200
1	Rittenhouse	0	0	0.00	0	29752	0	$123,600	$0
1	Chandler Heights	0	0	0.00	0	29752	0	$123,600	$0
2	Ray	2	318	106.85	47,957	29752	2	$123,600	$247,200
2	Rittenhouse	4.5	382	128.35	57,608	29752	2	$123,600	$247,200
2	Chandler Heights	10	584	196.22	88,071	29752	3	$123,600	$370,800
3	Ray	2	318	106.85	47,957	29752	2	$123,600	$247,200
3	Rittenhouse	8.5	795	267.12	119,892	29752	5	$123,600	$618,000
3	Chandler Heights	12	767	257.71	115,669	29752	4	$123,600	$494,400
4	Rittenhouse	5.5	952	319.87	143,569	29752	5	$123,600	$618,000
4	Chandler Heights	4.5	1241	416.98	187,152	29752	7	$123,600	$865,200

Hard Coding the HC Cards

Hard Coding the HC Cards

In order to reduce variation in calculated areas throughout the HEC-1 models, the District asked Collins/Pina Engineering to "hard code" the area input of the HC cards within each routing model. In other words, at each concentration point in the routing model, the sub-basin areas were specified instead of allowing HEC-1 to calculate the areas. To determine the area of all sub-basins influencing a given concentration point, the areas of each sub-basin, taken from the BA cards, were summed throughout the watershed of interest. These areas, calculated and supplied by the District, are given in the table below. CPE hard coded only the concentration points in the routing model and not in the watershed models.

Note that all values of area upstream of the EMF at Rittenhouse Road (EMFRIT) are identical between the 2002 existing conditions and the build-out conditions. In contrast, the areas differ in the EMF from Germann Road (EMFGRM) to Hunt Highway South (EMFHTS). Areas at EMFGRM, EMFQCN, and EMFQCS were all reduced by 3.34 square miles in the build-out conditions due to the assumption that flow from sub-basins 310, 314, and 318 would eventually be diverted into the EMF at Chandler Heights North (EMFCHN). These sub-basins currently flow into the channel at EMFGRM. In addition, the computed areas below EMFCHN were all increased by 0.194 square miles due to the eventual closure of a landfill in the Queen Creek/Sanokai Watershed. Finally, the areas of the model HC cards associated with the routing of flow through each detention basin were all set to 1.0, which adds more conservatism to the basin volumes or amount diverted.

Table—Values of concentration areas used for hard coding the HC cards within the routing model. All values supplied by the District.

Concentration Point	Sub-basin areas (sq. miles) Year 2002 Conditions	Sub-basin areas (sq. miles) Buildout Conditions
EMFBRN	7.75	7.75
EMFCLB	8.26	8.26
EMFUNI	12.71	12.71
EMFAPC	13.24	13.24
EMFBRD	15.14	15.14
EMFSTH	17.75	17.75
EMFSPR	20.99	20.99
EMFGUA	38.86	38.86
EMFELT	39.70	39.70
EMFWRN	44.21	44.21
EMFKNX	45.23	45.23
EMFPWR	107.61	107.61
EMFWRD	108.73	108.73
EMFWIL	111.38	111.38
EMFRIT	124.72	124.72
EMFGRM	129.19	125.85
EMFQCN	130.19	126.85
EMFQCS	131.16	127.82
EMFCHN	237.23	237.42
EMFCHS	239.15	239.34
EMFRG	240.68	240.87
EMFHTN	241.65	241.84
EMFHTS	249.28	249.47

Manning's Roughness Coefficients

		Alt 1			Alt 2			Alt 3			Alt 4		
	River Sta	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4
1	13.471		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
2	13.439		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
3	13.431	Queen's Creek Road											
4	13.426		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
5	13.374		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
6	13.28		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
7	13.232		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
8	13.179		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
9	13.084		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
10	13.037		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
11	12.981		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
12	12.905		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
13	12.884		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
14	12.789		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
15	12.694		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
16	12.6		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
17	12.552		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
18	12.488		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
19	12.441		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
20	12.349		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
21	12.302		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
22	12.245		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
23	12.177		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
24	12.082		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
25	11.988		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
26	11.893		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
27	11.798		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
28	11.703		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
29	11.609		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
30	11.572		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
31	11.531		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
32	11.486		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
33	11.391		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
34	11.328		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
35	11.321		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
36	11.308		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
37	11.297		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
38	11.26		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
39	11.254	Chandler Hgts. Rd.											
40	11.249		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
41	11.231		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
42	11.189		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
43	11.127		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
44	11.033		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
45	10.938		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
46	10.843		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
47	10.749		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
48	10.654		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
49	10.566		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
50	10.518		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
51	10.441		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
52	10.346		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
53	10.252		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
54	10.218		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
55	10.207	Riggs Rd.											
56	10.195		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
57	10.171		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
58	10.134		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
59	10.039		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
60	9.944		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
61	9.897		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
62	9.854		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
63	9.849		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
64	9.844		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
65	9.802		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
66	9.708		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
67	9.613		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
68	9.518		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
69	9.424		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
70	9.334		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
71	9.24		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
72	9.145		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
73	9.053		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
74	9.036		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
75	9.018		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
76	9.001		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	

		Alt 1			Alt 2			Alt 3			Alt 4		
	River Sta	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4
1	18.345		0.03			0.03			0.03			0.03	
2	18.237		0.015			0.015			0.015			0.015	
3	18.193		0.015			0.015			0.015			0.015	
4	18.087		0.015			0.015			0.015			0.015	
5	17.898		0.015			0.015			0.015			0.015	
6	17.803		0.015			0.015			0.015			0.015	
7	17.672		0.015			0.015			0.015			0.015	
8	17.571		0.015			0.015			0.015			0.015	
9	17.48		0.015			0.015			0.015			0.015	
10	17.339		0.015			0.015			0.015			0.015	
11	17.288		0.015			0.015			0.015			0.015	
12	17.269		0.015			0.015			0.015			0.015	
13	17.261	William's Field Rd.											
14	17.254		0.015			0.015			0.015			0.015	
15	17.25		0.015			0.015			0.015			0.015	
16	17.213		0.015			0.015			0.015			0.015	
17	17.17		0.015			0.015			0.015			0.015	
18	17.101		0.015			0.015			0.015			0.015	
19	17.086	Power Rd.											
20	17.082		0.015			0.015			0.015			0.015	
21	17.071		0.03			0.03			0.03			0.03	
22	17.039		0.03			0.03			0.03			0.03	
23	16.94		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
24	16.819		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
25	16.63		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
26	16.468		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
27	16.389		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
28	16.321		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
29	16.251		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
30	16.242		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
31	16.24	SPRR Bridge											
32	16.238		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
33	16.221		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
34	16.213	Rittenhouse											
35	16.205		0.03		0.03	0.03	0.03	0.04	0.03	0.04		0.03	
36	16.19		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
37	16		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
38	15.811		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
39	15.606		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
40	15.483		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
41	15.414		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
42	15.225		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
43	15.035		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
44	14.926		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
45	14.823		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
46	14.764		0.025		0.025	0.03	0.025	0.04	0.03	0.04		0.025	
47	14.754		0.025		0.025	0.03	0.025	0.04	0.03	0.04			
48	14.738		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
49	14.637		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
50	14.412		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
51	14.3		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
52	14.191		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
53	14		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
54	13.961		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
55	13.954	Higley Rd.											
56	13.946		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
57	13.911		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
58	13.842		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
59	13.755		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
60	13.661		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
61	13.566		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
62	13.471		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	

HEC-RAS Manning's n values through Reach 5

			Alt 1			Alt 2			Alt 3			Alt 4	
	River Sta	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4
1	24.723		0.03			0.03			0.03			0.03	
2	24.621		0.035			0.035			0.035			0.035	
3	24.535		0.035			0.035			0.035			0.035	
4	24.454		0.035			0.035			0.035			0.035	
5	24.32		0.035			0.035			0.035			0.035	
6	24.247		0.035			0.035			0.035			0.035	
7	24.13		0.035			0.035			0.035			0.035	
8	24.1		0.035			0.035			0.035			0.035	
9	24.06		0.035			0.035			0.035			0.035	
10	23.962		0.035			0.035			0.035			0.035	
11	23.911		0.035			0.035			0.035			0.035	
12	23.827		0.015			0.015			0.015			0.015	
13	23.825		0.015			0.015			0.015			0.015	
14	23.815	Southern Ave.											
15	23.804		0.015			0.015			0.015			0.015	
16	23.803		0.015			0.015			0.015			0.015	
17	23.758		0.035			0.035			0.035			0.035	
18	23.712		0.035			0.035			0.035			0.035	
19	23.604		0.035			0.035			0.035			0.035	
20	23.45		0.035			0.035			0.035			0.035	
21	23.315		0.035			0.035			0.035			0.035	
22	23.274		0.035			0.035			0.035			0.035	
23	23.219		0.035			0.035			0.035			0.035	
24	23.206	US 60											
25	23.193		0.035			0.035			0.035			0.035	
26	23.166		0.035			0.035			0.035			0.035	
27	23.135		0.035			0.035			0.035			0.035	
28	23.087		0.035			0.035			0.035			0.035	
29	23.009		0.035			0.035			0.035			0.035	
30	22.944		0.035			0.035			0.035			0.035	
31	22.866		0.035			0.035			0.035			0.035	
32	22.797		0.035			0.035			0.035			0.035	
33	22.74		0.035			0.035			0.035			0.035	
34	22.68		0.035			0.035			0.035			0.035	
35	22.615		0.035			0.035			0.035			0.035	
36	22.608	Baseline											
37	22.599		0.035			0.035			0.035			0.035	
38	22.557		0.035			0.035			0.035			0.035	
39	22.475		0.035			0.035			0.035			0.035	
40	22.436		0.035			0.035			0.035			0.035	
41	22.348		0.035			0.035			0.035			0.035	
42	22.263		0.035			0.035			0.035			0.035	
43	22.189		0.035			0.035			0.035			0.035	
44	22.107		0.035			0.035			0.035			0.035	
45	22.022		0.035			0.035			0.035			0.035	
46	21.947		0.035			0.035			0.035			0.035	
47	21.839		0.035			0.035			0.035			0.035	
48	21.745		0.035			0.035			0.035			0.035	
49	21.713		0.035			0.035			0.035			0.035	
50	21.656		0.035			0.035			0.035			0.035	
51	21.633		0.035			0.035			0.035			0.035	
52	21.591		0.035			0.035			0.035			0.035	
53	21.531		0.035			0.035			0.035			0.035	
54	21.485		0.035			0.035			0.035			0.035	
55	21.439		0.035			0.035			0.035			0.035	
56	21.418		0.03			0.03			0.03			0.03	
57	21.413		0.03			0.03			0.03			0.03	
58	21.407	Guadalupe											
59	21.402		0.03		0.035	0.03	0.035	0.035	0.03	0.035		0.03	
60	21.391		0.03		0.035	0.03	0.035	0.035	0.03	0.035		0.03	
61	21.355		0.03		0.035	0.03	0.035	0.035	0.03	0.035		0.03	
62	21.339	Power Rd.											
63	21.326		0.03		0.035	0.03	0.035	0.035	0.03	0.035		0.03	
64	21.282		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
65	21.188		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
66	21.093		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
67	21		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
68	20.894		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
69	20.799		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
70	20.704		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
71	20.61		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
72	20.517		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
73	20.428		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
74	20.353		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
75	20.334		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
76	20.328		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
77	20.323	Elliot Rd.											
78	20.316		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
79	20.306		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
80	20.247		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
81	20.058		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
82	19.863		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
83	19.745		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
84	19.592		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
85	19.541		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
86	19.489		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
87	19.458		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
88	19.408		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
89	19.298		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
90	19.109		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
91	18.97		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
92	18.782		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
93	18.535		0.025		0.035	0.03	0.035	0.035	0.03	0.035		0.025	
94	18.345		0.03		0.035	0.03	0.035	0.035	0.03	0.035		0.03	

HEC-RAS Manning's n values through Reach 6

		Alt 1			Alt 2			Alt 3			Alt 4		
	River Sta	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4	n #2	n #3	n #4
1	27.39		0.03			0.03			0.03			0.03	
2	27.371		0.03			0.03			0.03			0.03	
3	27.295		0.03			0.03			0.03			0.03	
4	27.278		0.013			0.013			0.013			0.013	
5	27.274		0.013			0.013			0.013			0.013	
6	27.261	Brown Rd.											
7	27.248		0.013			0.013			0.013			0.013	
8	27.244		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
9	27.189		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
10	27.095		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
11	27		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
12	26.905		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
13	26.811		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
14	26.762		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
15	26.727		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
16	26.722		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
17	26.712	Adobe St.											
18	26.702		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
19	26.697		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
20	26.678		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
21	26.595		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
22	26.5		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
23	26.385		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
24	26.311		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
25	26.228		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
26	26.187		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
27	26.171		0.035		0.04	0.035	0.04	0.04	0.035	0.04		0.035	
28	26.163	University Dr.											
29	26.154		0.035		0.04	0.035	0.04	0.04	0.035	0.04		0.035	
30	26.133		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
31	26.047		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
32	25.939		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
33	25.844		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
34	25.749		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
35	25.647		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
36	25.598		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
37	25.597		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
38	25.578		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
39	25.565	Main St./Higley Rd.											
40	25.508		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
41	25.49		0.013		0.013	0.013	0.013	0.013	0.013	0.013		0.013	
42	25.463		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
43	25.4		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
44	25.3		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
45	25.2		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
46	25.1		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
47	25.028		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
48	24.987		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
49	24.957		0.025		0.04	0.03	0.04	0.04	0.03	0.04		0.025	
50	24.946		0.03		0.04	0.03	0.04	0.04	0.03	0.04		0.03	
51	24.936	Broadway											
52	24.926		0.03			0.03			0.03			0.03	
53	24.9		0.03			0.03			0.03			0.03	
54	24.862		0.03			0.03			0.03			0.03	
55	24.723		0.03			0.03			0.03			0.03	

HEC-1 Generated Discharge Values

Discharge values (cfs) generated by HEC-1 Routing models at EMF locations.

		Y2002					Buildout			
RS		Alt 1	Alt 2	Alt 3	Alt 4		Alt 1	Alt 2	Alt 3	Alt 4
27.39	HOBART	497	497	497	497		782	782	782	782
27.278	EMFBRN	578	578	578	578		583	583	583	583
	EMFCLB	580	536	536	580		477	475	475	477
26.187	EMFUNI	1056	1032	1032	1056		1225	1093	1093	1225
25.598	EMFAPC	2188	2054	2054	2188		1463	1293	1293	1463
	EMFBRD	3069	2707	2707	3069		2443	2220	2220	2443
24.723	EMFSTH	3094	2841	2841	3094		2443	2195	2195	2443
23.219	EMFSPR	3374	3117	3117	3374		2739	2498	2498	2739
22.615	EMFGUA	4669	4299	4299	4669		3891	3689	3689	3891
21.282	EMFELT	4551	4125	4125	4551		3714	3400	3400	3714
19.298	EMFWRN	4594	4174	4174	4626		3738	3377	3377	3778
18.97	EMFKNX	4612	4166	4166	4745		3700	3254	3254	3825
18.237	EMFPWR	6245	5658	5658	10241		4923	4441	4441	6816
17.17	EMFWFD	6245	5665	5665	10087		4904	4436	4436	6775
	EMFWIL	6322	5728	5638	10187		4957	4483	4427	6892
	EMFRTT	4284	4651	5221	10133		2661	2985	3494	6049
16.251	EMFRIT	4637	3542	2865	4820		4438	3397	2718	4587
14.637	EMFGRM	4839	3768	3070	5046		4423	3391	2711	4579
13.471	EMFQCN	5007	4000	3267	5221		4415	3384	2708	4574
13.374	EMFQCS	5123	4107	3343	5342		4421	3391	2717	4581
	EMFQUE	5184	5984	5464	10186		2780	3159	3078	4135
11.26	EMFCHN	6870	5862	5358	5926		6550	5682	5212	5761
11.231	EMFCHS	7516	6498	5998	6598		6874	6031	5562	6109
	EMFRG	7513	6493	5992	6591		6870	6027	5555	6103
	EMFHTN	7508	6490	5986	6590		6860	6023	5551	6095
9.018	EMFHTS	7980	6935	6402	7355		7144	6273	5765	6795

Engineering Tools and Techniques

Engineering Tools and Techniques used in EMF Design

Creating, changing, and analyzing data within the EMF project can be complex. To facilitate and, in some cases, automate the generation of data used in the modeling, CPE developed several tools and techniques that are described below.

Diversion Card Input

Designing appropriately sized detention basins and side-weirs that efficiently mitigate flood potential within the EMF can be time consuming. Firstly, the designer must choose a crest and length for each side-weir within the EMF study area (as many as six side-weirs associated with three detention basins). Next, to accurately estimate the side-weir diversion characteristics as the water surface in the channel crests the side-weir, the designer is tasked with constructing a diversion profile over the designed side-weir. Ultimately, the designer must input this non-linear diversion profile into the HEC-1 model with the use of the DT/DI/DQ cards, describing side-weir diversion as a function of flow rate in the channel. To run the HEC-1 models, the designer must also input routing information for each detention basin with SE/SV/SQ cards, describing the water surface verses water volume in the basin. For each change in side-weir crest, side-weir length, or detention basin geometry, the designer needs to go back to recalculate the appropriate data for the DT/DI/DQ and the SE/SV/SQ cards used in the HEC-1 model. This process is quite time consuming. To expedite this iterative design process, CPE developed a Excel Spreadsheet, titled "diversion_card_input_altX.xls" that automates the calculation process and generates a summary sheet that collects and sorts all the diversion and routing data that the designer needs to enter into the HEC-1 model. The following pages show a sample of data generated by the Spreadsheet, and the text below briefly describes how to use the tool. Note that colors are used in the Spreadsheet that cannot be photocopied in the Appendix pages.

The entire spreadsheet contains four different styles of worksheets: 1) the side-weir data input sheet, 2) the detention basin data calculation sheet, 3) the summary sheet, and 4) a side-weir flow calculator that is used internally by the spreadsheet macros to calculate flow over a given side-weir (not displayed in this report). Displayed in the following sheets of the Appendix are example worksheets from the spreadsheet titled "diversion_card_input_alt1.xls." Three example sheets, representing each of the first three styles, are displayed. Within the spreadsheet, all cells colored with bright yellow represent input cells where the designer must supply data (visible in Excel only).

Data describing the geometry of each side-weir is entered through the side-weir data input sheet, shown in the following pages under the label "EMF to RITTDB." This particular worksheet calculates the diversion curve for the side-weir diverting flow from the EMF into the Rittenhouse Detention Basin. The cells on the upper right of the spreadsheet are an output table giving the side-weir rating curve. For example, in this sheet, the user entered a side-weir length of 1500 ft and a side-weir crest of 6.5 ft above the invert of the channel. The user also entered the values of a = 78.018 and b = 2.0991 for the power curve coefficients of the channel (see the section on Power Curve Generation). Finally, the user entered the elevation (Rel. Channel Invert = 0 ft) of the channel bottom relative to the drainage outlet of the detention basin. This value has no effect on the diversion values, but it is used to determine the lowest side-weir crest on the detention basin, which, in turn, determines the total capacity of the detention basin. If any of the input values of the spreadsheet differ from the values used to generate the output ratings curve, a red flag titled "**recalc**" appears in cell C-20 to alert the user that the rating curves may need to be recalculated. Finally, the designer presses the macro button titled "Calc diversions" at cell A-20 to automatically generate the side-weir rating curve. Note that although present in the spreadsheet, this macro button does not appear on the printed examples in the following pages. Also note that upon launching the Excel spreadsheet "diversion_card_input_alt1.xls," the user must select "enable macros" to have the macro functionality.

Next, the designer can enter the dimensions of any of the three detention basins to return the volume verses water surface data for the given basin. These sheets calculate the capacity and ratings curves for a prismatic basin. An example of this style of sheet is also shown ("Worksheet = Ray Basin"). Calculation of capacity of the contoured, multi-modal basin designs used in alternatives two and three are discussed later (in the Multi-modal Detention Basin Sizing section). All summary data from each of the calculation sheets is then collected and displayed on the "summary" worksheet. The "summary" worksheet is formatted in a fashion that allows the designer to print a single summary sheet, from which data can be directly entered into the HEC-1 model.

A typical process for the designer involves systematically stepping through each side-weir calculation sheet. On each sheet, the user enters the appropriate data of the new design (i.e. wide-weir crest, side-weir length, or power curve coefficients), then presses the "Calc diversions" macro button. This macro step is extremely important because output values are not automatically updated with changes in input values. The user then goes to the detention basin geometry worksheet to update any necessary input data. Finally, the user reads or prints the summary sheet to get the input data used for the HEC-1 model.

Power Curve Generation

Channel power curves are used by the side-weir diversion calculator to determine water depth in the channel as a function of channel flow rate. By plugging in diversion rates as a function of channel flow rate, HEC-1 can calculate the total diverted discharge over the side-weir.

To calculate the channel power curves, the user must first generate a curve of water depth as a function of channel flow rate. This curve can be generated by using several different flow rates as simulation data in a HEC-RAS hydraulic analysis of the channel. In addition, the designer can simulate a prismatic channel in Flow Master under normal depth conditions. If available, the designer can use experimental data from the channel itself. Once a curve of water depth verses channel flow rate is available, the designer can simply perform a power-based regression of the data to generate an equation of the form

$$WS = [Q/a]^{(1/b)}$$

where WS is the water depth in the channel, Q is the channel flow rate, and a and b are power curve coefficients obtained with the regression techniques. In Excel, the designer can quickly generate this regression relationship by first plotting flow rate (y-axis) verses water depth (x-axis) in an x-y graph. Then by applying a power trend line to the data, the display regression equation will generate the power coefficients, a and b where

$$Q = a\,(WS)^b$$

An example of the channel power curve for the EMF channel at Rittenhouse Basin is given in the following pages. The data were generated using HEC-RAS and the resulting regression curves generates coefficient values of a = 78.018 and b = 2.0991.

Culvert Sizing

The culverts were designed under the assumption that flap gates on each culvert prevented any draining of the detention basin until the flood peak passes. Once the peak passes the culvert flap gates are assumed to open, and the basin begins to drain. The culverts were all assumed to be 120 feet long and the conveyance through the culverts (a function of the pressure head behind the culvert and the energy losses through the culvert) was calculated with the Hydrocalc hydraulics software package. The Hydrocalc software predicts discharge from a culvert using FHWA Charts.

After generating a relationship describing culvert discharge verses water depth in the detention basin, a spreadsheet calculating water depth in the basin as a function of time was built. This spreadsheet is titled "culvertd.xls" and is displayed in the following pages as an example. Typically, the designer varies the quantity of culverts until the water surface at 36 hours is below 0.50 feet.

Pump Sizing

Based on the recommendation of Lakeside Equipment Corporation, Bartlett, Illinois, a pump gallery was selected for each basin and each alternative that required pumping. Lakeside recommended the District use 96-inch diameter, 3-flight open screw pumps with a 48-inch diameter by 3/8-inch thick wall while operating at 30° inclination. The unit price of the screw pump is $123,600. Each screw pump is capable of delivering 29,752 gallons per minute, and the quantity of pumps for each basin was selected to ensure all pumping was completed within 36 hours. The original faxed quote from Lakeside is included in the following pages.

Multi-modal Detention Basin Sizing

Using topographically contoured designs provided by Collins/Pina Landscaping Department, the overall capacity, as well as the volume verses depth relationships, were generated with a spreadsheet. Shown in the following pages, "contours.xls," the spreadsheet uses the average depth verses area method to calculate the volumetric capacity of the detention basin. The spreadsheet also automatically calculates the volume verses elevation used as HEC-1 input for the SE/SV/SQ cards. The user may need to adjust the curve factor to tune the capacity calculation to the desired detention basin size. Excel's goal seek algorithm allows the user to quickly converge on an appropriate curve factor. Note that this "contour.xls" Spreadsheet was only used on Rittenhouse and Chandler Heights Basin in Alternatives 2 and 3. All other basins were sized as a rectangular prism.

Diversion Card Input

Summary of Diversion Card Input Calculator.

	Santan into Ray 4.5 Crest (rtc) 200 Length		Power into Ray 3.25 Crest 750 Length		EMF into Rittenhouse 6.5 Crest 1500 Length		Ritt into Rittenhouse 2 Crest 150 Length		EMF into CHDB 5 Crest 1000 Length		Queen into CHDB 6.2 Crest 500 Length	
	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)
1	750	0	528	0	3968	0	269	0	2897	0	3118	0
2	772	7	544	11	4087	64	277	2	2984	39	3212	27
3	819	31	576	40	4332	253	294	9	3163	163	3404	121
4	892	78	628	89	4722	590	320	23	3448	392	3711	308
5	999	154	704	162	5289	1108	358	48	3862	751	4156	617
7	1356	439	955	410	7177	2902	486	140	5241	2018	5640	1767
9	8138	6509	5730	5174	43074	38309	2919	2207	31452	27602	33848	26376

Ray DB

Max Depth = 9.25 ft
Area = 171 Acres
Max Vol = 1,537 Ac-ft
Perim = 11,104 ft

SV	SE	SQ
0	0.00	0
244	1.54	1
493	3.08	2
747	4.63	3
1005	6.17	4
1269	7.71	5
1537	9.25	6

Total head pumped (ft) = 2

Rittenhouse DB

Max Depth = 6.5 ft
Area = 111 Acres
Max Vol = 690 Ac-ft
Perim = 11,146 ft

SV	SE	SQ
0	0.00	0
109	1.08	1
220	2.17	2
334	3.25	3
450	4.33	4
569	5.42	5
690	6.50	6

Total head pumped (ft) = 0

Chandler Heights DB

Max Depth = 5 ft
Area = 300 Acres
Max Vol = 1,415 Ac-ft
Perim = 20,807 ft

SV	SE	SQ
0	0.00	0
229	0.83	1
461	1.67	2
696	2.50	3
933	3.33	4
1173	4.17	5
1415	5.00	6

Total head pumped (ft) = 0

Date and time = 10/30/00 14:56
Design = Vers. 7.06
Alt = Alt 1
Designer = CSM

Pwr curve Coeffs.	sandb	pwrdb	rittdb	rrddb	nchdb	chqcdb
WS-a	35.224	66.102	78.018	77.579	152.47	67.416
WS-b	2.0331	1.7627	2.0991	1.7936	1.8296	2.1014

Summary of Diversion Card Input Calculator.

	Santan into Ray 4.5 Crest (rtc) 200 Length		Power into Ray 3.25 Crest 750 Length		EMF into Rittenhouse 6.35 Crest 1500 Length		Ritt into Rittenhouse 2 Crest 150 Length		EMF into CHDB 5 Crest 1000 Length		Queen into CHDB 6.25 Crest 500 Length	
	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)
1	750	0	528	0	2995	0	269	0	1941	0	3171	0
2	772	7	544	11	3085	57	277	2	1999	34	3266	27
3	819	31	576	40	3270	215	294	9	2119	130	3462	122
4	892	78	628	89	3564	486	320	23	2310	301	3774	313
5	999	154	704	162	3992	894	358	48	2587	559	4227	626
7	1356	439	955	410	5417	2285	486	140	3511	1450	5736	1794
9	8138	6509	5730	5174	32513	29248	2919	2207	21070	18873	34424	26788

Ray DB

Max Depth = 9.25 ft
Area = 171 Acres
Max Vol = 1,537 Ac-ft
Perim = 11,104 ft

SV	SE	SQ
0	0.00	0
244	1.54	1
493	3.08	2
747	4.63	3
1005	6.17	4
1269	7.71	5
1537	9.25	6

Total head pumped (ft) = 2

Rittenhouse DB

Max Depth = 6.35 ft
Area = 179 Acres
Max Vol = 982 Ac-ft
Perim = 11,146 ft

SV	SE	SQ
0	0.00	0
149	1.80	1
305	3.60	2
465	5.40	3
632	7.20	4
804	9.00	5
982	10.90	6

Total head pumped (ft) = 4.5

Values calculated using "contour.xls"

Chandler Heights DB

Max Depth = 5 ft
Area = 300 Acres
Max Vol = 1,591 Ac-ft
Perim = 20,807 ft

SV	SE	SQ
0	0.00	0
14	1.30	1
47	2.70	2
101	4.00	3
379	7.70	4
871	11.30	5
1591	15.00	6

Total head pumped (ft) = 10

Values calculated using "contour.xls"

Date and time = 10/30/00 14:53
Design = Vers. 8.06
Alt = Alt 2
Designer = CSM

Pwr curve Coeffs.	sandb	pwrdb	rittdb	rrddb	nchdb	chqcdb
WS-a	35.224	66.102	77.923	77.579	105.5	67.416
WS-b	2.0331	1.7627	1.9741	1.7936	1.8095	2.1014

Summary of Diversion Card Input Calculator.

	Santan into Ray 4.5 Crest (rtc) 200 Length		Power into Ray 3.25 Crest 750 Length		EMF into Rittenhouse 6.35 Crest 1500 Length		Ritt into Rittenhouse 2 Crest 150 Length		EMF into CHDB 5 Crest 1000 Length		Queen into CHDB 5.75 Crest 500 Length	
	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)
1	750	0	528	0	2356	0	269	0	1941	0	2662	0
2	772	7	544	11	2427	50	277	2	1999	34	2741	24
3	819	31	576	40	2572	180	294	9	2119	130	2906	106
4	892	78	628	89	2804	400	320	23	2310	301	3167	270
5	999	154	704	162	3140	727	358	48	2587	559	3547	539
7	1356	439	955	410	4261	1832	486	140	3511	1450	4814	1536
9	8138	6509	5730	5174	25574	23092	2919	2207	21070	18873	28891	22798

Ray DB

Max Depth =	**9.25**	**ft**
Area =	**171**	**Acres**
Max Vol =	**1,537**	**Ac-ft**
Perim =	**11,104**	**ft**

SV	SE	SQ
0	0.00	0
244	1.54	1
493	3.08	2
747	4.63	3
1005	6.17	4
1269	7.71	5
1537	9.25	6

Total head pumped (ft) = 2

Rittenhouse DB

Max Depth =	**6.35**	**ft**
Area =	**179**	**Acres**
Max Vol =	**1,344**	**Ac-ft**
Perim =	**11,146**	**ft**

SV	SE	SQ
0	0.00	0
204	2.48	1
417	4.95	2
637	7.43	3
865	9.90	4
1110	12.38	5
1344	14.85	6

Total head pumped (ft) = 8.5

Values calculated using "contour.xls"

Chandler Heights DB

Max Depth =	**5**	**ft**
Area =	**300**	**Acres**
Max Vol =	**1,807**	**Ac-ft**
Perim =	**20,807**	**ft**

SV	SE	SQ
0	0.00	0
16	1.50	1
53	3.00	2
114	4.50	3
430	8.67	4
989	12.83	5
1807	17.00	6

Total head pumped (ft) = 12

Values calculated using "contour.xls"

Date and time = 10/30/00 14:56
Design = Vers. 8.04
Alt = Alt 3
Designer = CSM

Pwr curve Coeffs.	sandb	pwrdb	rittdb	rrddb	nchdb	chqcdb
WS-a	35.224	66.102	60.841	77.579	105.5	67.416
WS-b	2.0331	1.7627	1.9781	1.7936	1.8095	2.1014

Summary of Diversion Card Input Calculator.

	Santan into Ray	Power into Ray	EMF into Rittenhouse		Ritt into Rittenhouse		EMF into CHDB		Queen into CHDB	
			3.5 Crest		2.5 Crest		4.5 Crest		5.5 Crest	
	N/A	N/A	2000 Length		500 Length		1000 Length		1000 Length	
			Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)	Channel flow	Diversion (cfs)
1			3877	0	401	0	2955	0	2424	0
2			3994	52	413	6	3044	40	2497	35
3			4233	217	438	26	3226	165	2647	144
4			4614	523	478	60	3516	397	2885	342
5			5168	1002	535	112	3938	761	3231	648
7			7013	2697	726	294	5344	2051	4385	1723
9			42088	36995	4356	3883	32075	28189	26315	23183

Ray DB

N/A

Rittenhouse DB

Max Depth =	**9**	**ft**
Area =	**179**	**Acres**
Max Vol =	**1,590**	**Ac-ft**
Perim =	**11,146**	**ft**

SV	SE	SQ
0	0.00	0
253	1.50	1
512	3.00	2
774	4.50	3
1041	6.00	4
1313	7.50	5
1590	9.00	6

Total head pumped (ft) = 5.5

Chandler Heights DB

Max Depth =	**9**	**ft**
Area =	**300**	**Acres**
Max Vol =	**2,616**	**Ac-ft**
Perim =	**20,807**	**ft**

SV	SE	SQ
0	0.00	0
414	1.50	1
838	3.00	2
1269	4.50	3
1709	6.00	4
2158	7.50	5
2616	9.00	6

Total head pumped (ft) = 4.5

Date and time = **10/30/00 14:28**
Design = **Vers. 9.02**
Alt = **Alt 4**
Designer = **CSM**

Pwr curve Coeffs.	sandb	pwrdb	rittdb	rrddb	nchdb	chqcdb
WS-a	35.224	66.102	466.3	77.579	244.84	67.416
WS-b	2.0331	1.7627	1.6907	1.7936	1.6559	2.1014

Stage diversion relationship curve for the diversion from the EMF into Rittenhouse Basin at River Station 16.321

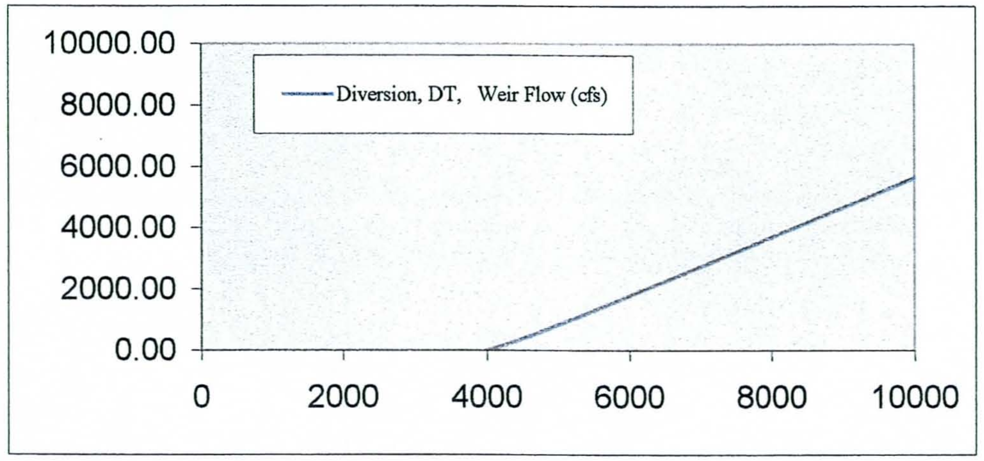
	Input Values	
Weir Length, L =	1500	ft
Weir Crest =	6.5	ft
Rel. Channel Invert (ft) =	0	ft
channel elevation	0	

Channel Flow rate	Water Surface (relative)	Diversion, DT, Weir Flow (cfs)
3968	6.50	0.00
4087	6.59	63.63
4332	6.78	252.52
4722	7.06	589.98
5289	7.45	1107.62
6082	7.97	1853.88
7177	8.62	2901.96
8684	9.44	4361.13
43074	20.24	38308.85

Power Curve Coeffs.
[WS or Vel = (Q/a)^(1/b)]

	WS
a =	*78.018*
b =	*2.0991*

recalc



Given a detention basin area, height, perimeter, and side slope,
the following sheet calculates the volume of Ray detention basin.
Included is a geometric calculator that sizes a non-rectangular Ray Basin.
The spreadsheet accounts for the decrease in volume due to the side slope of the detention basin walls and presents a volume verses elevation relationship.

Top of Basin rel to culvert outlet (ft) =	**13.7**
DB Side Slope =	**4**
Perimeter (ft) =	**11,104.00**
Total effective DB depth [min weir to basin invert] (ft) =	**9.25**
Area at top of basin (Acres) =	**171**
Area at Base (Acres) =	**156.72**

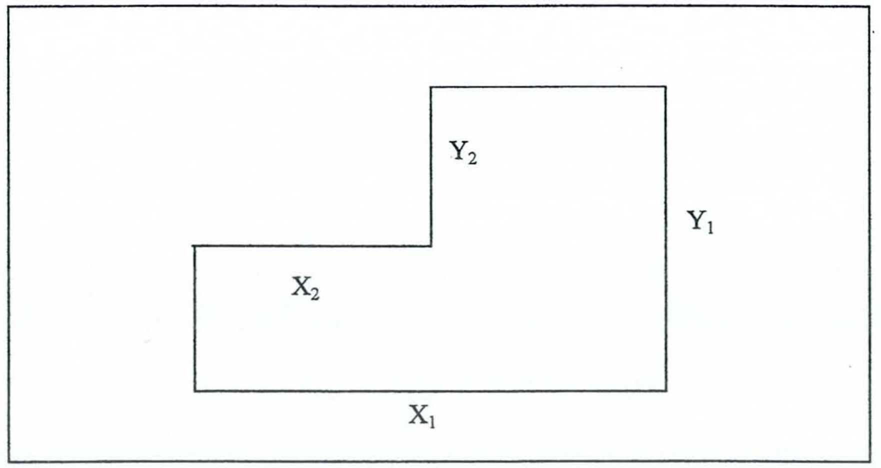
Min Weir Hgt relative to culvert outlet **7.25**

	Chandler Heights	Elevation	Volume (Ac-ft)	
0	0.00	0.00	-	
1	0.17	1.54	244	
2	0.33	3.08	493	
3	0.50	4.63	747	
4	0.67	6.17	1,005	
5	0.83	7.71	1,269	
6	1.00	9.25	1,537	DB Capacity

PUMPING	
Total Head Pumped (ft) =	2
Total Volume Pumped (Ac-ft) =	318

	590
X1 =	2902
X2 =	1700
Y1 =	2650
Y2 =	150

Original H-Z Dimensions	
X1 =	2240
X2 =	1110
Y1 =	2650
Y2 =	875

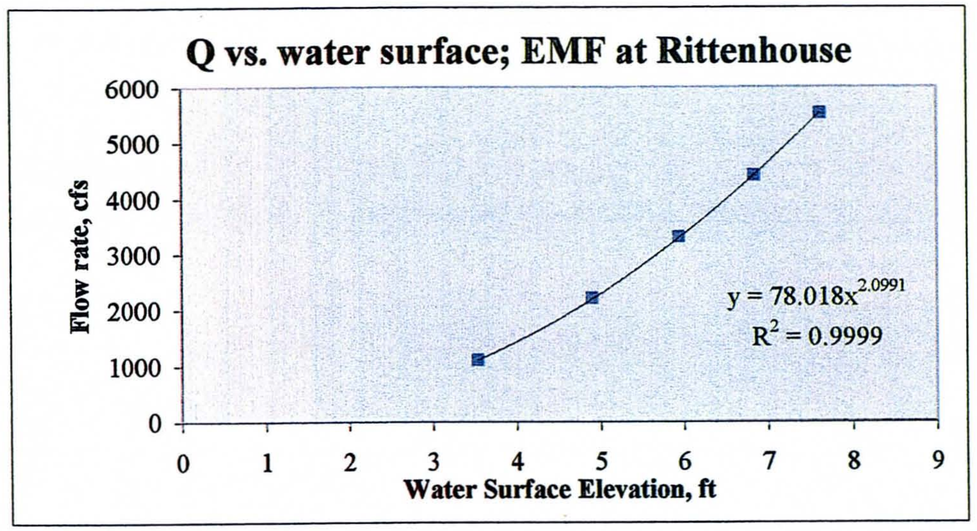


Power Curve Generation

The data below were generated with HEC-RAS using flow conditions at 20%, 40%, 60%, 80%, and 100% of the 2002 flows. Values taken from reach 4, at station 16.94
June 14, 2000 csm

EMF at Rittenhouse

WS	Q Total
7.62	5515
6.84	4412
5.95	3309
4.9	2206
3.54	1103
2.58	552

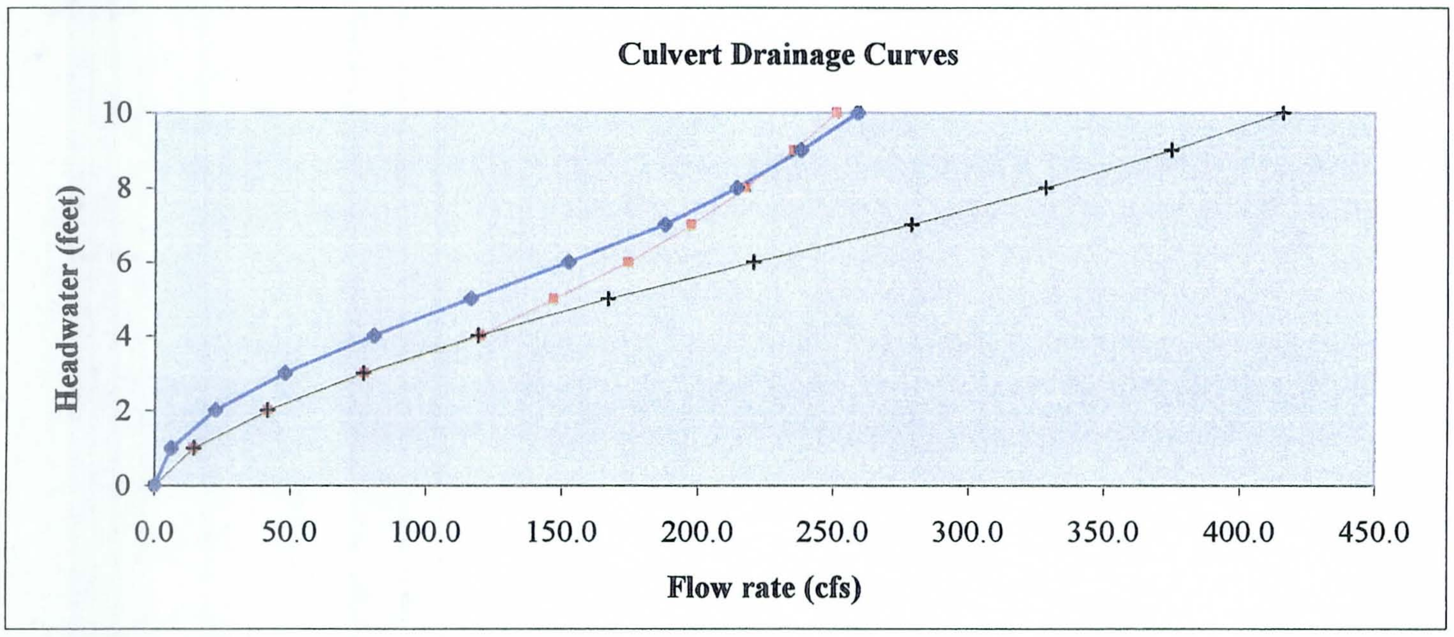


Culvert Sizing

The Table and Chart below calculates the ratings curves for concrete culverts, 120 feet in length.
Flow rate curves were based on FHWA Charts using the HYDROCALC Hydraulics software package.
A linear flow model was used at the lower end of headwater values to ground the curves to 0,0.

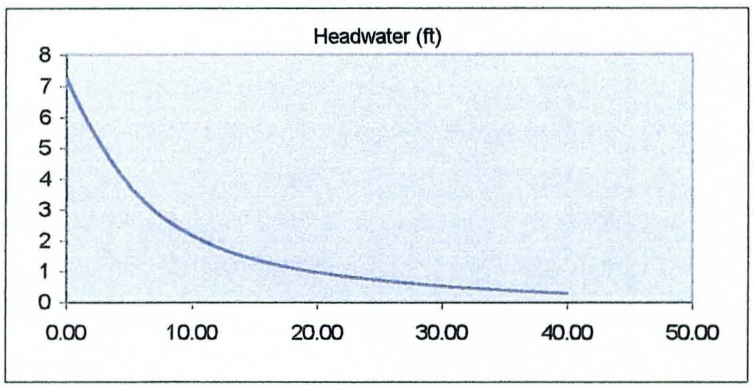
	6'x3' Box	6'x6' Box	60" Circular
Linear	12.65	12.65	5.32
Valid cutoff	0.79	0.79	0.94
Box outlet a	14.428	14.531	6.0428
Box outlet b	1.5282	1.5182	1.8832
Valid cutoff	4	7.94	3.7
Mid Linear m	0	0	35.84
Mid Linear b	0	0	-62.59
Valid cutoff	4	7.94	6.6
Outlet a	149.96	393.04	199.77
Outlet b	94.449	489.04	201.13

Head (feet)	Flow (cfs) [6x3 Box]	Flow (cfs) [6x6 Box]	Flow (cfs) [60 inch circular]
0	0.0	0.0	0.0
1	14.4	14.5	6.0
2	41.6	41.6	22.3
3	77.3	77.0	47.8
4	120.0	119.2	80.8
5	146.9	167.3	116.6
6	174.2	220.6	152.5
7	197.4	278.8	187.6
8	217.4	328.3	214.3
9	235.0	374.6	237.8
10	250.8	416.0	258.9



WS at 36 hours =	0.37
Number of Culverts =	9
Starting HW (ft) =	7.25
Area (Acres) =	171
DB Height (ft) =	13.7
Perimeter (ft) =	10960
DB Side Slope =	4
Area at the base (Acres) =	157.21

	6'x3' Box
Linear	12.65
Valid cutoff	0.79
Box outlet a	14.428
Box outlet b	1.5282
Valid cutoff	4
Mid Linear m	0
Mid Linear b	0
Valid cutoff	4
Outlet a	149.96
Outlet b	94.449



Time (min)	Time (hours)	Headwater (ft)	DB Volume (Ac-ft)	Outflow rate (cfs	Vol outflow (Ac-ft)	New DB Volume (Ac-ft)	**New Headwater (ft)**
0	0.00	7.25	1,192.69	1823.6	12.559	1,180.13	**7.18**
5	0.08	7.18	1180.13	1809.9	12.465	1,167.66	**7.10**
10	0.17	7.10	1167.66	1796.2	12.370	1,155.29	**7.03**
15	0.25	7.03	1155.29	1782.4	12.276	1,143.02	**6.96**
20	0.33	6.96	1143.02	1768.6	12.180	1,130.84	**6.89**
25	0.42	6.89	1130.84	1754.7	12.085	1,118.75	**6.82**
30	0.50	6.82	1118.75	1740.8	11.989	1,106.76	**6.75**
35	0.58	6.75	1106.76	1726.8	11.893	1,094.87	**6.68**
40	0.67	6.68	1094.87	1712.8	11.796	1,083.07	**6.61**
45	0.75	6.61	1083.07	1698.8	11.700	1,071.37	**6.54**
50	0.83	6.54	1071.37	1684.7	11.603	1,059.77	**6.47**
55	0.92	6.47	1059.77	1670.6	11.505	1,048.27	**6.41**
60	1.00	6.41	1048.27	1656.4	11.408	1,036.86	**6.34**
65	1.08	6.34	1036.86	1642.2	11.310	1,025.55	**6.27**
70	1.17	6.27	1025.55	1627.9	11.212	1,014.34	**6.21**
75	1.25	6.21	1014.34	1613.6	11.113	1,003.22	**6.14**
80	1.33	6.14	1003.22	1599.3	11.015	992.21	**6.08**
85	1.42	6.08	992.21	1585.0	10.916	981.29	**6.01**
90	1.50	6.01	981.29	1570.6	10.817	970.48	**5.95**
95	1.58	5.95	970.48	1556.1	10.717	959.76	**5.88**
100	1.67	5.88	959.76	1541.7	10.618	949.14	**5.82**
105	1.75	5.82	949.14	1527.2	10.518	938.62	**5.76**
110	1.83	5.76	938.62	1512.7	10.418	928.21	**5.70**
115	1.92	5.70	928.21	1498.1	10.318	917.89	**5.64**
120	2.00	5.64	917.89	1483.5	10.217	907.67	**5.57**
125	2.08	5.57	907.67	1468.9	10.117	897.56	**5.51**
130	2.17	5.51	897.56	1454.3	10.016	887.54	**5.45**
135	2.25	5.45	887.54	1439.7	9.915	877.62	**5.40**
140	2.33	5.40	877.62	1425.0	9.814	867.81	**5.34**
145	2.42	5.34	867.81	1410.3	9.713	858.10	**5.28**
150	2.50	5.28	858.10	1395.6	9.612	848.49	**5.22**
155	2.58	5.22	848.49	1380.9	9.510	838.98	**5.17**
160	2.67	5.17	838.98	1366.1	9.409	829.57	**5.11**
165	2.75	5.11	829.57	1351.4	9.307	820.26	**5.05**
170	2.83	5.05	820.26	1336.6	9.205	811.05	**5.00**
175	2.92	5.00	811.05	1321.9	9.104	801.95	**4.94**
180	3.00	4.94	801.95	1307.1	9.002	792.95	**4.89**
185	3.08	4.89	792.95	1292.3	8.900	784.05	**4.84**
190	3.17	4.84	784.05	1277.5	8.798	775.25	**4.78**
195	3.25	4.78	775.25	1262.7	8.696	766.55	**4.73**
200	3.33	4.73	766.55	1247.9	8.595	757.96	**4.68**
205	3.42	4.68	757.96	1233.1	8.493	749.47	**4.63**
210	3.50	4.63	749.47	1218.4	8.391	741.08	**4.58**
215	3.58	4.58	741.08	1203.6	8.289	732.79	**4.53**
220	3.67	4.53	732.79	1188.8	8.187	724.60	**4.48**
225	3.75	4.48	724.60	1174.1	8.086	716.51	**4.43**
230	3.83	4.43	716.51	1159.3	7.984	708.53	**4.38**
235	3.92	4.38	708.53	1144.6	7.883	700.65	**4.34**
240	4.00	4.34	700.65	1129.9	7.782	692.86	**4.29**
245	4.08	4.29	692.86	1115.2	7.681	685.18	**4.24**
250	4.17	4.24	685.18	1100.6	7.580	677.60	**4.20**
255	4.25	4.20	677.60	1086.0	7.479	670.12	**4.15**
260	4.33	4.15	670.12	1071.4	7.378	662.75	**4.11**
265	4.42	4.11	662.75	1056.8	7.278	655.47	**4.06**
270	4.50	4.06	655.47	1042.3	7.178	648.29	**4.02**
275	4.58	4.02	648.29	1027.8	7.078	641.21	**3.98**
280	4.67	3.98	641.21	1070.9	7.375	633.84	**3.93**
285	4.75	3.93	633.84	1052.6	7.249	626.59	**3.89**
290	4.83	3.89	626.59	1034.7	7.126	619.46	**3.85**
295	4.92	3.85	619.46	1017.2	7.005	612.46	**3.80**
300	5.00	3.80	612.46	1000.1	6.887	605.57	**3.76**
305	5.08	3.76	605.57	983.3	6.772	598.80	**3.72**
310	5.17	3.72	598.80	966.9	6.659	592.14	**3.68**
315	5.25	3.68	592.14	950.9	6.549	585.59	**3.64**
320	5.33	3.64	585.59	935.2	6.441	579.15	**3.60**
325	5.42	3.60	579.15	919.9	6.336	572.81	**3.56**
330	5.50	3.56	572.81	904.9	6.232	566.58	**3.52**
335	5.58	3.52	566.58	890.2	6.131	560.45	**3.49**
340	5.67	3.49	560.45	875.9	6.032	554.42	**3.45**
345	5.75	3.45	554.42	861.8	5.935	548.48	**3.41**
350	5.83	3.41	548.48	848.0	5.840	542.64	**3.38**

Pump Sizing

LAKESIDE

Water Purification Since 1928

www.enquip.com/LAKESIDE-1.htm

LAKESIDE EQUIPMENT CORPORATION
FAX MEMORANDUM

1022 E. Devon Ave * Box 8448 * Bartlett, IL 60103 * 630-837-5640 * Fax: 630-837-5647 * E-Mail: sales@lakeside-equipment.com

DATE: July 19, 2000 **FAX #:** 520-884-5278

TO: Collins Pina Engineers **SUBJECT:** Phoenix, Arizona

ATTN: Mr. Magirl Storm Water Retention Basin

FROM: Larry Lehnert

Total number of pages, including cover sheet: 6

Dear Mr. Magirl:

In accordance with your request, we are pleased to provide our Open Screw Pump recommendations for the referenced project.

INTRODUCTION

Lakeside Open Screw Pumps offer the following advantages over conventional pumps:

- Variable Pumping Capacity - The open screw pump has built-in variable capacity that automatically adjusts the pumping rate and power consumption to the depth of the liquid in the inlet chamber while operating at a constant speed.

- High-Efficiency - Screw pumps provide efficient pumping over a wide range and operate economically down to 30% of maximum design pumping capacity. The high-efficiency pumping results in lower electrical costs over the entire life of the equipment.

- Non-Clogging - Open screw pumps require no screening and pass debris as large as the gap between the screw flights or the wall and torque tube.

- Minimum Maintenance - Open screw pumps operate at slow speed to reduce friction that causes parts damage and heat generation. Only periodic maintenance is required for oil changes, greasing the upper bearing and adding grease to the lower bearing automatic lubrication system.

- No Wetwell - Open screw pumps do not require a wet well, pump house or housing.

OPEN SCREW PUMPS

To handle the peak flow of 115,000 gal/min, we recommend four (4) 96-in. diameter, 3-flight open screw pumps with a 48-in. diameter by 3/8-in. thick wall for a lift (H) of 14 ft while operating at 30° inclination. Each pump will operate at 27 rev/min while delivering 29,752 gal/min and will draw 148 bhp at the motor. We recommend a 200 hp drive rated at a minimum of 490,665 in-lb torque for the maximum pumping condition. See SPD-150 for our calculations.

Budget pricing is as follows:

Unit Price:	- $123,600
Total Price:	- $494,400
Approximate Shipping Weight/Unit:	- 30,000lb

Budget pricing includes:

- Shop prime painting
- Four (4) days of start-up service and operator training in two (2) trips to the project site.
- Freight allowed FOB our factory in Chariton, Iowa to the project site.

DRAWINGS AND SPECIFICATIONS

Refer to drawing D-45005-S and the dimensional data sheet for our suggested layout of the screw pumps which have been selected for this project. As this project moves forward, we can provide drawings on disk suitable for incorporation into most CAD systems. Likewise, specifications can also be furnished on disk.

If you have any questions, feel free to contact our local representative, Rod Johnson , or this office.

Best regards,

Larry Lehnert

Larry Lehnert
/lal

cc: Rod Johnson, Goble Sampson Associates, Inc.

LAKESIDE EQUIPMENT CORPORATION
1022 East Devon Avenue
BARTLETT, ILLINOIS 60103
(312) 837-5640

JOB : PHOENIX, AZ
NO. OF PUMPS REQ'D IS 4
ENGINEER : COLLINS PINA ENGR
CALCULATED BY LAL DATE 07/18/00
CHECKED BY ________ DATE ________
SHEET NO. ______ OF ______ SPD-150

GIVEN INFORMATION

PUMP DIAM. = D = 96 In.
CAPACITY = Q'= 28750 GPM
LIFT = H = 14 Ft.
NO. FLIGHTS = 3
SLOPE = alpha = 30 Degrees
OTHER :

RECOMMENDED PUMP(S)

(1) Angle, Alpha = 30 Degrees
(2) MAX. Q = 30854 GPM
(3) MAX. RPM = N = 28
(4) LIFT "H" = 14.00 Ft.
(5) "d" = 48.0 In.
(6) "D" = 96.000 In.
(7) REDUCER S.F. = 1.50 (BHP)
(8) TOTAL LOAD = 1534 Lb/Ft
(9) MOMENT INERTIA = 15908 IN ^4
(10) DEAD LOAD = 387 Lb/ft.
(11) MOTOR HP = 200
(12) BRG. CENTERS = 43.20 Ft.

RADIAL BEARING LOAD = 23916 #/BRG.
THRUST BEARING LOAD = 32664 #/BRG.

TORQUE TUBE = 48.0" x 0.3750"
REQ'D RPM N' = NQ'/Q = 27
ACTUAL CAPACITY = N'Q/N= Q" = 29752 GPM

BHP = (Q" x H x 8.33)/(33000 x 0.75) = 140.19
BHP at Motor =140.19/ 0.95 = 147.57
USE 200.0 HP MOTOR
REQ'D REDUCER TORQUE = 490665 In-Lb

BHP x 1.50 x 63,000/ N' = 490665 In-Lb
OR MOTOR HP x1.00 x .95 x 63000/ N' = 443333 In-L

FLIGHT LENGTH = 35.99 Ft.

ALLOWABLE DEFLECTION = 0.259 In.
TOTAL DEFLECTION = 0.188 In.
DEAD LOAD DEFLECTION = 0.047 In.
HORIZONTAL DEFLECTION = 0.055 In.

MISC. NOTES & COMMENTS

Upper Bearing : 9"
Lower Bearing : 8 3/4"

BASE MOUNT Reducer : NORPA 355

LAKESIDE EQUIPMENT CORPORATION
1022 East Devon Avenue
BARTLETT, ILLINOIS 60103

JOB : PHOENIX, AZ
NO. OF PUMPS REQ'D IS 4
ENGINEER : COLLINS PINA ENGR
CALCULATED BY LAL DATE 07/18/00
CHECKED BY ________ DATE ________

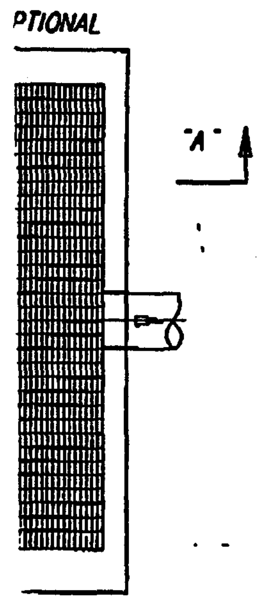
Dimensions for use on Drawing________________

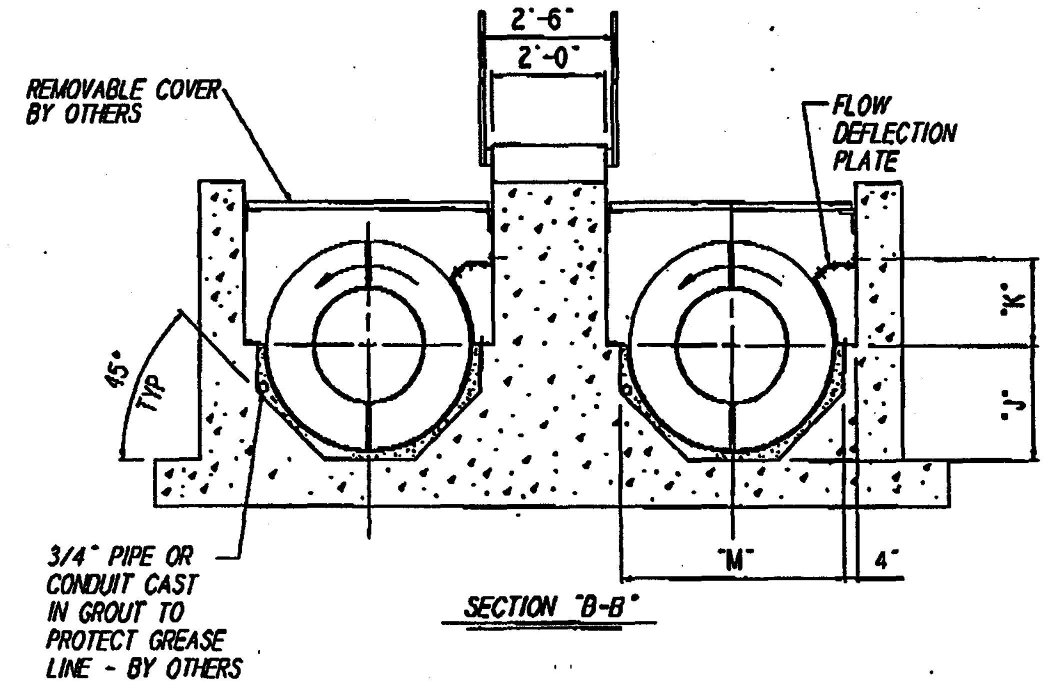
DIA.		alpha	A	B	C	E	F
96"		30 deg.	5' 6-1/4"	3' 6"	As Reqd.	7' 10"	18' 1-1/4"

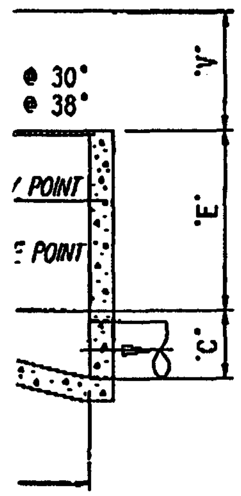
G	H	J	K	L	M	N	P
2'-0"	14'-0"	4' 3-1/2"	2' 3-1/2"	36' 1"	8' 8"	As Reqd.	1' 2-1/2"

Q	R	S	T	U	V	W	X	Y
1' 8"	1' 5"	2' 3-3/4"	9'-0"	8' 9"	4' 10-1/2"	3' 8"	3' 6"	31' 4-1/2"

MOTOR (HP)	SCREW (RPM)	CAPACITY (GPM)	NO. OF FLIGHTS	TORQUE TUBE	WALL THICK.
200	27	28750	3	48"	0.375"







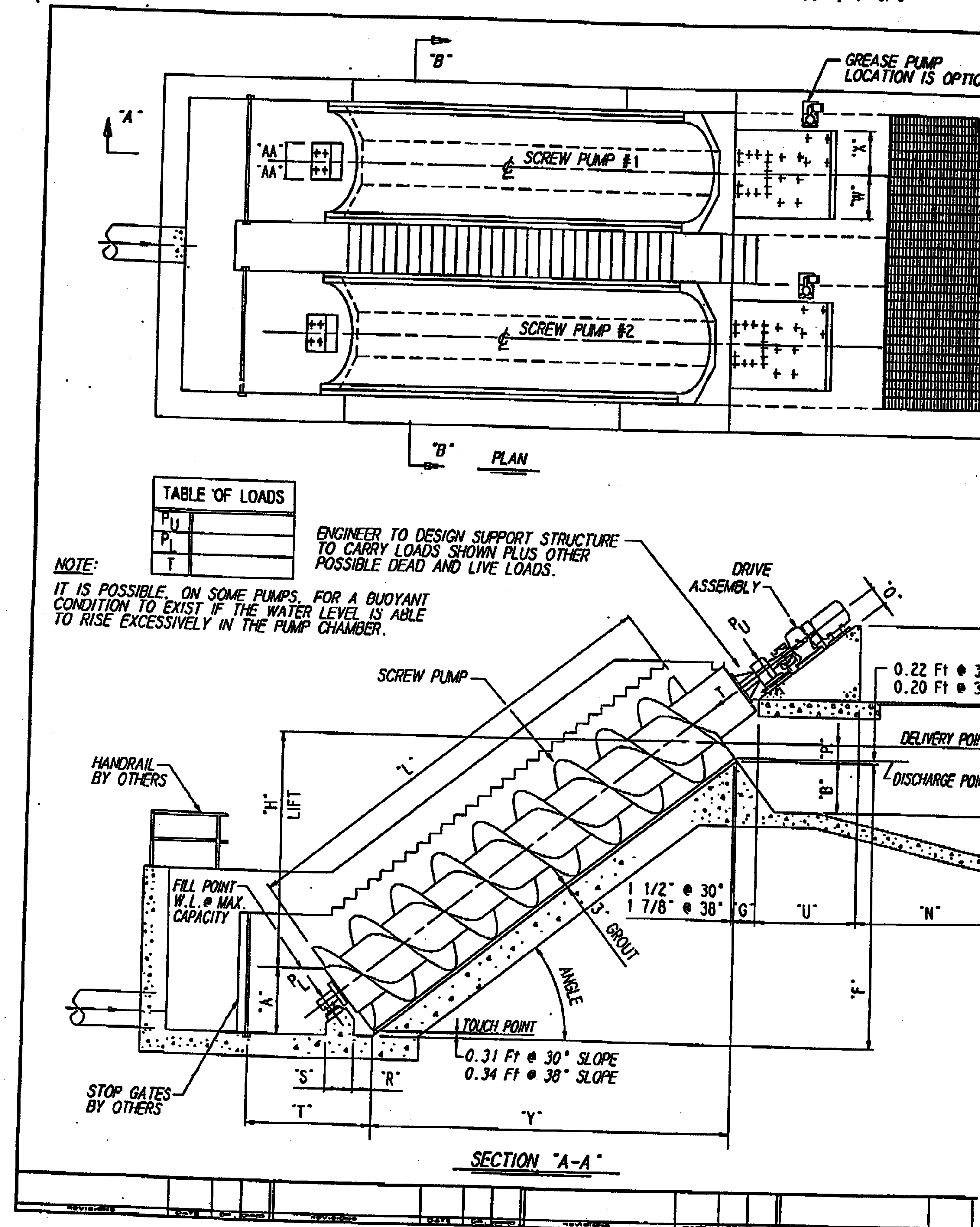
PUMP DIA	ANGLE	A	B	C	E	F	G	H	J

K	L	M	N	P	Q	R	S	T	U	V

W	X	Y	AA	MOTOR HP	SCREW RPM	CAPACITY GPM	QTY. of FLIGHTS	TORQUE TUBE DIA	WALL THICKNESS

LAKESIDE EQUIPMENT CORPORATION | DR. GS | CHKD. TODD | DATE 02-02-94 | FILE NO. 251 | OPEN SCREW PUMP PARALLEL SHAFT DRIVE | DWG. NO. D-45005-S | REV.

ORIGINAL


"B"
GREASE PUMP LOCATION IS OPTION
"A"
"AA"
"AA"
SCREW PUMP #1
SCREW PUMP #2
"X"
"W"
"B"
PLAN
TABLE OF LOADS
P_U
P_L
T
NOTE:
IT IS POSSIBLE, ON SOME PUMPS, FOR A BUOYANT CONDITION TO EXIST IF THE WATER LEVEL IS ABLE TO RISE EXCESSIVELY IN THE PUMP CHAMBER.
ENGINEER TO DESIGN SUPPORT STRUCTURE TO CARRY LOADS SHOWN PLUS OTHER POSSIBLE DEAD AND LIVE LOADS.
DRIVE ASSEMBLY
"O"
P_U
T
0.22 Ft @ 3
0.20 Ft @ 3
SCREW PUMP
DELIVERY POI
DISCHARGE POI
"P"
"B"
"L"
"H"
LIFT
HANDRAIL BY OTHERS
FILL POINT W.L.@ MAX. CAPACITY
1 1/2" @ 30°
1 7/8" @ 38°
"G"
"U"
"N"
3" GROUT
ANGLE
"F"
"A"
P_L
TOUCH POINT
0.31 Ft @ 30° SLOPE
0.34 Ft @ 38° SLOPE
"S"
"R"
"T"
"Y"
STOP GATES BY OTHERS
SECTION "A-A"

Multi-modal Detention Basin Sizing

EMF Weir

Top (weir crest) =	6.35
Base (pond)=	-4.5

From Rittenhouse Channel

Top (weir crest) =	6.35
Base (pond)=	-4.5

	Contour		Area (Acres)		Mean Area	Elevation	Delta Volume (Ac-ft)	Subtotals
Subbasin 1	A1		89.96			6.35		
		B1		16.79		-4.50		
		B2		21.70		-4.50		
		B3		26.56		-4.50		
		B4		8.96	*81.99*	-4.50	*889.53725*	
								889.53725
Subbasin 3	A5		9.97			6.35		
		B5		7.04	*8.51*	-4.50	*92.27925*	
								92.27925

Rittenhouse DB Total Volume (Ac-ft) = 982 need to be 974 Ac-ft

A Area	99.93
B Area	81.05

Elevation to pump =	0.0
Fraction elevation =	0.41
Fraction Area =	88.88
Volume to pump =	**382**

Elevation Contours

curve factor = 1

	Elev	Area
Top=	10.85	99.93
Bottom=	0	81.05

Elevation	Fract Elev	Elev	Area	Volume
0	0.00	0.00	81.05	0
1	0.17	1.81	84.20	149.41
2	0.33	3.62	87.34	155.10
3	0.50	5.43	90.49	160.79
4	0.67	7.23	93.64	166.48
5	0.83	9.04	96.78	172.17
6	1.00	10.85	99.93	177.86
				981.82
			match to =	*982*

Card Input	
SV	SE
0	0.0
149	1.8
305	3.6
465	5.4
632	7.2
804	9.0
982	10.9

North Basin (EMF Weir)

Top (weir crest) =	5
Middle =	-6
Base (pond)=	-10

South Basin (Queen's Creek Weir)

Top (weir crest) =	5
Middle =	-6
Base (pond)=	-10

	Contour	Area (Acres)	Mean Area	Elevation	Delta Volume (Ac-ft)	Subtotals	
Subbasin 1	A1	85.39		5.00			
	B1a	9.41		-6.00			
	C1a	0.26	*4.84*	-10.00	*19.34*		
	B1b	11.81	*53.31*	-6.00	*586.355*		
	C1b	1.92	*6.87*	-10.00	*27.46*		
						633.155	
Subbasin 2	A2	87.4		5.00			
	B2a	7.49		-6.00			
	B2b	4.62	*49.76*	-6.00	*547.305*		
	C2b	0.56	*2.59*	-10.00	*10.36*		
						557.665	
Subbasin 3	A3	55.33		5.00			
	B3a	9.02		-6.00			
	B3b	5.93	*35.14*	-6.00	*386.54*		
	C3b	0.91	*3.42*	-10.00	*13.68*		
						400.22	
	Chandler Heights Total Volume (Ac-ft) =					1591	1570

A Area	228.12
B Area	48.28
C Area	3.65
Elevation to pump =	0
Fraction elevation =	0.55
Fraction Area =	146.37
Volume to pump =	**584**

Elevation Contours

curve factor = 1.0744225

	Elev	Area
Top=	15	228.12
middle =	4	48.28
Bottom=	0	3.65

Elevation	Fract Elev	Elev	Area	Volume
0	0.00	0.00	3.65	0
1	0.33	1.33	17.36	14.01
2	0.67	2.67	32.52	33.25
3	1.00	4.00	48.28	53.87
4	0.33	7.67	103.52	278.30
5	0.67	11.33	164.61	491.57
6	1.00	15.00	228.12	720.00
				1591.00
			match to =	*1591*

Card Input

SV	SE
0	0.0
14	1.3
47	2.7
101	4.0
379	7.7
871	11.3
1591	15.0

EMF Weir

Top (weir crest) = 6.35

Base (pond)= -8.5

From Rittenhouse Channel

Top (weir crest) = 6.35

Base (pond)= -8.5

	Contour		Area (Acres)		Mean Area	Elevation	Delta Volume (Ac-ft)	Subtotals
Subbasin 1	A1		89.96			6.35		
		B1		16.79		-8.50		
		B2		21.70		-8.50		
		B3		26.56		-8.50		
		B4		8.96	*81.99*	-8.50	*1217.47725*	
								1217.4773
Subbasin 3	A5		9.97			6.35		
		B5		7.04	*8.51*	-8.50	*126.29925*	
								126.29925

Rittenhouse DB Total Volume (Ac-ft) = 1344 need to be **1345** Ac-ft

A Area 99.93

B Area 81.05

Elevation to pump = 0.0

Fraction elevation = 0.57

Fraction Area = 91.86

Volume to pump = 735

Elevation Contours

curve factor = 1

	Elev	Area
Top=	14.85	99.93
Bottom=	0	81.05

Elevation	Fract Elev	Elev	Area	Volume
0	0.00	0.00	81.05	0
1	0.17	2.48	84.20	204.49
2	0.33	4.95	87.34	212.28
3	0.50	7.43	90.49	220.07
4	0.67	9.90	93.64	227.86
5	0.83	12.38	96.78	235.64
6	1.00	14.85	99.93	243.43
				1343.78
			match to =	*1344*

Card Input	
SV	**SE**
0	**0.0**
204	**2.5**
417	**5.0**
637	**7.4**
865	**9.9**
1100	**12.4**
1344	**14.9**

	Contour	Area (Acres)	Mean Area	Elevation	Delta Volume (Ac-ft)	Subtotals
Subbasin 1	A1	85.39		5.00		
	B1a	9.41		-7.50		
	C1a	0.26	*4.84*	-12.00	*21.7575*	
	B1b	11.81	*53.31*	-7.50	*666.3125*	
	C1b	1.92	*6.87*	-12.00	*30.8925*	
						718.9625
Subbasin 2	A2	87.4		5.00		
	B2a	7.49		-7.50		
	B2b	4.62	*49.76*	-7.50	*621.9375*	
	C2b	0.56	*2.59*	-12.00	*11.655*	
						633.5925
Subbasin 3	A3	55.33		5.00		
	B3a	9.02		-7.50		
	B3b	5.93	*35.14*	-7.50	*439.25*	
	C3b	0.91	*3.42*	-12.00	*15.39*	
						454.64
					Chandler Heights Total Volume (Ac-ft) =	1807.195

1950

North Basin (EMF Weir)

Top (weir crest) =	5
Middle =	-7.5
Base (pond)=	-12

South Basin (Queen's Creek Weir)

Top (weir crest) =	5
Middle =	-7.5
Base (pond)=	-12

A Area	228.12
B Area	48.28
C Area	3.65
Elevation to pump =	0
Fraction elevation =	0.60
Fraction Area =	156.18
Volume to pump =	767

Elevation Contours

curve factor = 1.074022711

	Elev	Area
Top=	17	228.12
middle =	4.5	48.28
Bottom=	0	3.65

Elevation	Fract Elev	Elev	Area	Volume
0	0.00	0.00	3.65	0
1	0.33	1.50	17.36	15.76
2	0.67	3.00	32.52	37.42
3	1.00	4.50	48.28	60.60
4	0.33	8.67	103.54	316.30
5	0.67	12.83	164.63	558.69
6	1.00	17.00	228.12	818.23
				1807.00
			match to =	*1807*

Card Input	
SV	**SE**
0	**0.0**
16	**1.5**
53	**3.0**
114	**4.5**
430	**8.7**
989	**12.8**
1807	**17.0**

Side-weir Discharge Calculation Approach

Side-Weir Discharge Calculations Used in the East Maricopa Floodway Capacity Mitigation and Multi-Use Corridor Study

August 2000

Christopher Magirl, Principal Author
Michael E. Zeller, P.E., P.H., Technical Reviewer
Collins/Pina Engineering & Tetra Tech, Inc., ISG

Introduction:

In order to alleviate flooding potential for residents along the East Maricopa Floodway (EMF), Collins/Pina Engineering (CPE) completed a hydrologic study and conceptual design of mitigation structures for the watershed and drainage canals. Due to changes in the watershed mostly related to urbanization, the EMF, which was constructed by the Soil Conservation Service in 1989, can no longer convey a 100-year flood event safely away from Eastern Maricopa County. Using existing hydrologic and hydraulic models, CPE first evaluated, and then subsequently designed, offline detention basins to capture peak flows in the EMF and four major tributaries. By capturing the flood peak, these basins would reduce the overall flood potential within the EMF. Three detention basins are proposed along the EMF, with each basin accepting flow diverted from adjacent channels. Because these basins are offline, and located adjacent to the channels, all diversions were accomplished with broad-crested side-weirs. As a flood rises and flows past the detention basin, the strategically placed side-weirs divert the top of the flood peak into the retention basin (Figure 1). The following discussion reviews the hydraulic assumptions made by CPE in predicting this diverted flow over each side-weir, and proposes potential improvements to the model.

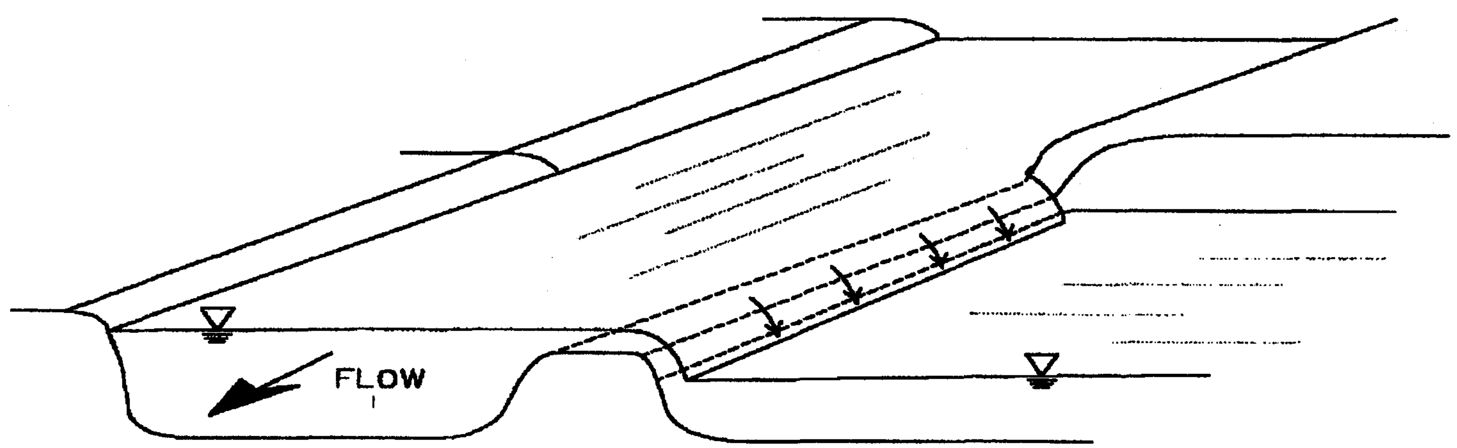


Figure 1 – Schematic of flow over a side-weir from the main channel.

Previous Research:

Numerous investigators have studied the flow characteristics of open-channel side-weirs. One of the first was Engels, who, in 1917, conducted a series of flow experiments with a flume and a sharp-crested side-weir cut into the wall of the flume (Collinge 1956). Engels was able to empirically derive an equation describing lateral discharge as a function of side-weir length, *L*, and the depth of flow, *h*, over the weir crest at the downstream end of the side-weir. The Engel's Equation is:

$$Q = 3.32L^{0.83}h^{1.67}. \qquad (1)$$

Where Q is the total flow over the side-weir in cfs; *L* and *h* are both in the units of feet. Engels' equation is applicable to subcritical flow in a rectangular channel with a sharp-crested side-weir that is relatively short in length. The equation is still commonly used in the wastewater industry. Interestingly, Engels found that for subcritical flow, at some distance before the side-weir, the water surface along the wall of the side-weir drops to a minimum value at the leading edge of the side-weir cut-out. Furthermore, the water-surface depth along the crest of the side-weir actually increases in the downstream direction. Though rarely explained in the literature, this seemingly odd behavior is probably an artifact of the high radial acceleration of flow around and into the leading edge of the side-weir opening. In much the same way that a whirlpool draws the water surface downward, the concentration of vorticity at the leading edge of the side-weir decreases fluid pressure and pulls the water surface down. Moving downstream, vorticity dissipates and the water surface returns to the normal depth expected from the channel geometry.

In 1934, De Marchi published a theoretical model describing discharge from a side-weir. Among his assumptions in the model, two are particularly important:

1. The unit discharge (q') over any given length of the side-weir can be calculated from the normal weir formula,

   $$q' = C_d H^{1.5}. \qquad (2)$$

   Where C_d is the weir coefficient and H is the head above the side-weir, and

2. The total *specific energy* [i.e., $y + V^2/(2g)$] in the channel through the side-weir section remains constant.

This second assumption restricts the model to short side-weirs, or those side-weirs that cause negligible change to depth of flow in the channel (Collinge 1956). De Marchi's theory was later summarized by Chow (1959). Many additional researchers, including Collinge; Subramanya and Awasthy (1972); Ranga Raju, et al. (1979); and Borghei, et al. (1998) have since attempted to improve De Marchi's approach. De Marchi's theory proposes the following equation for discharge over a side-weir:

$$q' = 2/3\ C_m\,(2g)^{0.5}\,(H)^{1.5}. \qquad (3)$$

Where q' is unit discharge (cfs/ft), g is gravity, and C_m is the De Marchi coefficient of discharge. Collinge, Borghei, et al., and Agaccioglu and Yuksel (1998) all gave excellent summaries of previous side-weir work. One approach that receives wide acceptance in practice is that of Hager (1987). Hager built upon the De Marchi theory (3) by proposing the following equation for C_m:

$$C_m = 0.485[(2+F^2)/(2+3F^2)]^{0.5}. \qquad (4)$$

Where F is the Froude Number. Recently, Majaj released a Basin Analysis System (BAS) software package that simulates an entire detention basin with side-weirs and drainage culverts. However, the BAS software, as well as most other classic side-weir studies, relies on a De Marchi-type approach by assuming constant specific energy throughout the side-weir section.

Current Approach:

While investigations of side-weirs with negligible energy loss are legion, the current engineering problem attempts to select a side-weir for which CPE believes that the assumption of constant specific energy throughout the side-weir section does not apply. In fact, the ultimate goal of the designed side-weirs in the EMF is to significantly remove the peak of the 100-year flood, hence the energy, from the EMF. Typically, the side-weirs in the current project are designed to remove roughly approximately 60-65% of the discharge, with a corresponding reduction of from 20-30% of the energy in the flow. CPE believes that, in a channel like the EMF, this also means the water surface would be lowered by approximately 20-30%. In contrast, classic side-weir studies typically report specific energy losses of less than 5%.

The EMF problem differs from classic side-weir analyses in three fundamental areas. Firstly, the EMF and its tributaries are wide, flat, trapezoidal channels. In contrast, classical approaches almost always use rectangular channels. Secondly, the side-weirs used in the EMF project are broad-crested (at least 10 feet in width), while classical investigations use almost exclusively sharp-crested side-weirs. Finally, due to the very long length of the side-weirs, CPE believes that the energy in the channel does not remain constant throughout the reach of the channel. Due do these differences between classical side-weir theory and the current engineering design problem, CPE took a modified approach to solving the problem of flow over a relatively long, broad-crested side-weir.

In solving the problem of the long, broad-crested side-weir diverting flow from a wide trapezoidal channel, CPE made the following assumptions:

1. Flow is incompressible, steady, and one-dimensional in the channel.

2. The unit discharge over any given length of the side-weir can be calculated from the normal weir formula, namely:

$$q' = C_d H^{1.5}. \quad (5)$$

(Note: This is one of the original assumptions used by De Marchi.)

3. The weir coefficient for flow over the side-weir is assumed to take the broad-crested value, $C_d = 2.63$.

4. The crest of the side-weir remains parallel to the channel bottom.

5. Channel flow rate as a function of depth of flow in the channel follows a power-curve relationship of the form:

$$Q = a\,(y)^b. \quad (6)$$

Where y is the depth of flow in the channel, Q is the flow rate within the channel, and "a" and "b" are constants determined through regression techniques.

6. The power curve for channel flow rate versus depth of flow in the channel is constant throughout the reach of the side-weir.

7. Specific energy in the channel is not necessarily constant through the length of the side-weir, but may drop as a function of the decrease of flow along the reach of the channel containing the side-weir.

8. Due to the relative length of the side-weirs, any influence that the downstream component of momentum in the channel might have upon the diversion of flow is neglected, thus the driving force for flow over the side-weir is the head of the water in the main channel at any particular point along the side-weir.

In calculating the discharge over the side-weir, CPE used a numerical approach. First, the side-weir was discretized into many small sections, each section or node small in comparison to the overall length of the side-weir (CPE used 1000 nodal sections). Starting at the most upstream location of the side-weir and working downstream, a prediction of diverted flow per unit length over the side-weir for this small section was calculated using the broad-crested weir equation (5). CPE assumed that the driving head, H, was simply the difference between the depth of flow in the channel at the upstream end of discretized section and the height of the side-weir crest. Moreover, the depth of flow was calculated using the channel flow rate versus depth of flow power-curve relationship (Equation 6). The total diverted flow, dQ, from a given node was then calculated as the differential unit discharge, Δq, multiplied by the incremental length of the node, Δs. Once the side-weir discharge was calculated for a particular section, continuity was used to recalculate the flow rate in the main channel at the next section downstream (Figure 2). Using the new downstream flow rate in conjunction with the channel power curves, a new depth of flow was computed—leading to the calculation of the side-weir discharge over the next section downstream. This process was continually repeated in the downstream direction along the entire length of the side-weir, ultimately resulting in a prediction of the total side-weir discharge.

Case Study:

To the best of CPE's knowledge, no data exist that fully describe the behavior of a long, broad-crested side-weir. However, in an attempt to evaluate the relative accuracy of the side-weir calculation technique used by CPE against classical approaches, several approaches were compared. In particular, flow discharges were calculated for a side-weir placed adjacent to a generically wide, trapezoidal channel. The example selected used the following input parameters:

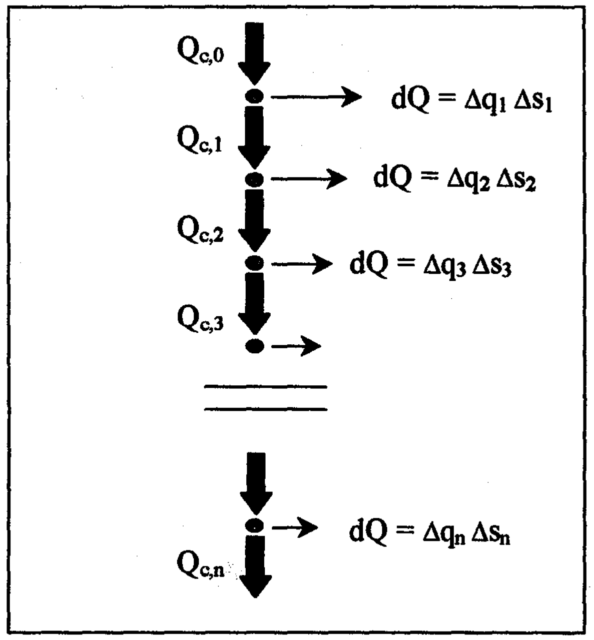


Figure 2 – Schematic showing the numerical scheme used by CPE. The large arrows represent the flow in the main channel and the small arrows represent nodal diversions.

Channel Bottom width (ft)	=	170
Manning's "n"	=	0.025
Side slope H:V	=	4:1
Channel Slope	=	0.0004
Depth of flow (ft)	=	6.7
Froude number	=	0.28
Channel flow rate (cfs)	=	5110
Coefficient, Cd, of Side-Weir	=	2.63
Height of Side-Weir Crest above channel invert (ft)	=	4
Length of Side-Weir (ft)	=	1000
Width of Side-Weir (ft)	=	10
Power-curve coefficient, a	=	200.67
Power-curve coefficient, b	=	1.702

Where the power-curve coefficients "a" and "b" are used in conjunction with Equation 6 to calculate channel flow rate versus depth of flow (i.e., $Q = 200.67y^{1.702}$).

Table 1 presents the results for the example side-weir when using the Engels Equation (1), the Hager approach (3)-(4), and the CPE algorithm described above. Note the relatively wide range of values for side-weir diversion calculated with the three methods. Also note, however, the relatively good agreement between the Hager approach and the CPE algorithm with respect to unit discharge at the most upstream location of the side-weir. As can be deduced, along the first few feet of the example side-weir, the Hager approach and the CPE algorithm predict similar discharges. By recalculating depth of flow in the channel throughout the side-weir section, the CPE algorithm predicts an asymptotically decreasing unit discharge. In contrast to classic side-weir equations, in which side-weir discharge linearly increases with length, the CPE approach adjusts unit discharge with increasing length. Moreover, the CPE approach will not predict a diversion larger than the incremental flow in the channel above the crest of the side-weir. It should also be noted, however, that in order to achieve a target channel flow rate downstream of a side-weir with a given side-weir crest, an engineer using the CPE approach would predict a much longer side-weir than would the engineer using a classic approach to the side-weir problem.

Table 1 – Calculations of diverted flow from example side-weir using three independent approaches.

	ENGELS	HAGER	CPE
Diversion Per Unit Length at Beginning of the Weir (cfs/ft)	*5.4*	*11.1*	*11.7*
Total Diversion (cfs)	*5,389*	*11,101*	*2,724*

Figure 3, below, shows the predicted diversion of the Hager approach from the CPE approach as the length of the side-weir increases. Consistent with the values in Table 1, Figure 3 shows how the two approaches give similar results for short side-weirs. However, as the length of the side-weir increases past approximately 50 feet, the predicted flow diversions begin to diverge. While the Hager approach continues upward in a linear fashion, the CPE curve levels off, asymptotically approaching a maximum total amount of flow diversion. The maximum value corresponds to the difference between the flow in channel before the diversion and the flow value in the channel after the diversion, assuming the post-diversion depth of flow equals the height of the side-weir crest itself. Physically, this diversion limit represents the maximum diversion from a side-weir of infinite length. In theory, such a side-weir would reduce the depth of flow in the channel to exactly the height of the side-weir crest, though CPE is unaware of any technical literature that illustrates this flow behavior.

Discussion and Conclusions:

In approaching the design problem of appropriately sizing the length of side-weirs over which flow would discharge into offline detention basins, CPE quickly discovered a lack of literature and research that closely represented the design challenge at hand. Though literature on the nature of side-weirs is plentiful, CPE was unable to locate research specifically targeted at discharges from long, broad-crested side-weirs that significantly reduce the energy in the main channel. In order to produce results that gave CPE greater confidence with respect to the prediction of flow diversions from a side-weir, a simple approach was developed. Fortunately, the current investigation is conceptual in nature, primarily aiming to show that certain detention structures could meet the design criteria for the project. Therefore, great precision is not necessary. Instead, CPE developed a side-weir calculation approach and design philosophy that focused upon the characterization of the behavior of long side-weirs that

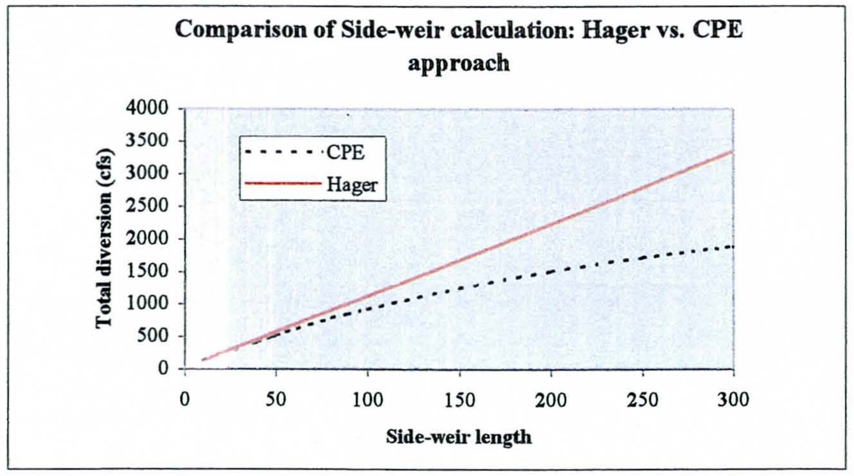


Figure 3 – Illustration of Divergence of Hager and CPE side-weir calculations as side-weir becomes long.

effectively "chop" the flood peak down to the height of the weir crests. Given the nature of the CPE approach, it is possible that the side-weirs, as designed, are longer than they need to be. Nonetheless, CPE feels that the assumption of broad-crested side-weir flow driven by the total head in the channel provides the necessary accuracy to meet the needs of the current conceptual design.

As the project moves into the next stage of detailed design, CPE strongly suggests that the District adopts a rigorous and detailed design approach for all side-weirs in the project scope. This is because different problems can arise in the final design, dependent upon whether the designed side-weir either over predicts or under predicts the volume of diverted flow. For example, over prediction of the diverted flow imperils undersized downstream improvements; while under prediction of the diverted flow leads to greater diversion into the detention basin, and a potentially overwhelming of the basin. For the final engineering design, empirical physical modeling or some form of 2D or 3D numerical modeling is recommended in order to fully validate the chosen side-weir design approach.

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